INTERNATIONAL CODE COUNCIL 2012/2013 CODE DEVELOPMENT CYCLE Group A (2012)

PROPOSED CHANGES TO THE 2012 EDITIONS OF THE

INTERNATIONAL BUILDING CODE[®]

INTERNATIONAL FUEL GAS CODE®

INTERNATIONAL MECHANICAL CODE®

INTERNATIONAL PLUMBING CODE®

INTERNATIONAL PRIVATE SEWAGE DISPOSAL CODE[®]



April 29th – May 8th, 2012 Sheraton Dallas Hotel Dallas, TX

First Printing

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TABLE OF CONTENTS (Group A)

Introductionii
2012 ICC Code Development Hearingsii
Registration and Votingii
Advanced Registrationiii
Code Development Process Changesiii
Proceduresiii
Assembly Actioniv
Multiple Part Code Change Proposalsiv
Group A and B Code Changes by Committeev
Group A Code Development Committee Responsibilitiesvi
Analysis Statementsvi
Reference Standardsvi
Referenced Standards Updatesvi
Nodifications vii
Code Correlation Committee viii
ICC Website viii
2012/2013 Code Development Scheduleix
2012/2013 Staff Secretariesx
Assignment Crossover — Within the IBCxi
CP #28-05 Code Development xii
Cross Index of Proposed Changesxxiv
Hearing Schedulexxix
2012 Proposed Changes xxx

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.

2012 ICC CODE DEVELOPMENT HEARINGS

These proposed changes will be discussed in public hearings to be held on April 29th, 2012 through May 8th, 2012 at the Sheraton Dallas Hotel, Dallas, Texas. The code committees will conduct their public hearings in accordance with the schedule shown on page xxix.

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:

Saturday, April 28th Sunday, April 29th through Tuesday, May 8th

4:00 pm to 6:00 pm 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. Each member is entitled to one vote, except that each Governmental Member Voting Representative in attendance may vote on behalf of its Governmental Member. 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.

As such, new membership applications as well as renewal applications must be received by ICC's Member Services Department by April 18th, 2012. 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 April 18th, 2012 deadline. For information on application for new membership and membership renewal, please go to <u>www.iccsafe.org/membership/join.html</u> 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 <u>www.iccsafe.org/springhearings</u>.

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 2012/2013 Code Development Cycle is the first schedule for the revised code development process (see page ix).

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 1.6: **Recording.** This section was revised to clarify that ICC maintains sole ownership in the content of the hearings and has the right to control its subsequent distribution. In addition, the technology references were updated, using the term "recording" to replace "videotaping".
- Section 2.4 **Emergency Procedures.** This section was revised create a 'metric' to aid in the determination of when an issue rises to the level of concern appropriate to an emergency amendment. Furthermore, it now stipulates a process by which a proposed Emergency Amendment is reviewed by the ICC Codes and Standards Council who is responsible for the implementation and oversight of ICC's Code Development Process.
- Section 3.3.1
- &
- Section 6.4.1 **Proponent.** An e-mail address for each code change/public comment proponent will be published in the monograph, unless the proponent requests otherwise.

Section 3.3.5.3

- & Section 6.4.5 **Substantiation.** ICC evaluates whether substantiating material is germane, but the amendment makes it clear that ICC does not in all circumstances evaluate substantiating material for guality or accuracy.
- Section 3.3.5.6 **Cost Impact.** The proponent should submit information that supports their claim regarding cost impact. Any information submitted will be considered by the code development committee. This language is intended to emphasize the need to provide information on how the proposed change will affect the cost of construction.
- Section 3.6.3.1 If a proposed new standard is not submitted in at least draft form, the corresponding code change proposal shall be considered incomplete and shall not be processed.
- Section 4.5.1 **Standards referenced in the I-Codes.** The deadline for availability of updated referenced standards and receipt by the Secretariat is December 1st of the third year of each code cycle. For the 2012/2013 cycle, the deadline is December 1st, 2014.

- Section 5.2.2 **Conflict of interest.** The original language, "Violation thereof shall result in the immediate removal of the committee member from the committee." was removed because there was no mechanism to enforce it. The recourse for someone who feels this section has been violated is to appeal.
- Section 5.4.2 **Open meetings.** A provision has been added that stipulates that participants shall not advocate a position on specific code changes with Committee Members other than through the methods provided in this policy.

Section 5.4.3

&

- Section 7.3.3 **Presentation of Material at the Public Hearing.** All participants are to make it clear what interests they are representing. This disclosure provides additional information upon which to evaluate the testimony.
- Section 5.7 **Assembly consideration.** A successful assembly action will no longer be the initial motion at the Final Action Consideration.
- Section 5.7.3 **Assembly action.** A successful assembly action shall be a majority vote of the votes cast by eligible voters, rather than a 2/3 majority (see below).
- Section 5.7.4 **Eligible voters.** This section is revised to clarify that each member, including Governmental Member Voting Representatives, gets only one vote.
- Section 7.4 **Eligible voters.** This section requires that all Governmental Membership applications must be received by April 1 of the year of the Final Actions for a Governmental Member to be eligible to vote at the Final Action Hearings.

ASSEMBLY ACTION

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

- A successful assembly action now requires a simple majority rather than a 2/3 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 code change proposal that receives a successful assembly action will be placed on the Final Action Hearing Agenda for individual consideration.

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.

GROUP A AND GROUP B CODE CHANGES

Starting with this 2012/2013 Code Development Cycle, for the development of the 2015 Edition of the I-Codes, there are two groups of code development committees and they will meet in separate years. The groupings are as follows:

(Heard In 2012) (Heard In 2013) International Building Code Committees: Administrative Provisions (Chapter 1 all codes except IRC and IECC, referenced standards administrative updates, and desig definitions) IBC-Fire Safety (Chapters: 7, 8, 9, 14, 26 and App. D) Administrative Provisions (Chapter 1 all codes except IRC and IECC, referenced standards administrative updates, and desig definitions) IBC-General (Chapters: 2-6, 12, 13, 27-34, App. A, B, C, F, H, K) Administrative Code Committee IBC-Means of Egress (Chapters: 10, 11 and App. E) IBC-Structural (Chapters: 15-25 and App. G,I, J, L, M) International Fuel Gas Code International Energy Conservation Code (see note 1) IFGC Committee Commercial Energy Committee	
International Building Code Committees: Administrative Provisions (Chapter 1 all codes except IRC and IECC, referenced standards administrative updates, and desig definitions) IBC-Fire Safety (Chapters: 7, 8, 9, 14, 26 and App. D) IBC-General (Chapters: 2-6, 12, 13, 27-34, App. A, B, C, F, H, K) IBC-Means of Egress (Chapters: 10, 11 and App. E) Administrative Code Committee IBC-Structural (Chapters: 15-25 and App. G,I, J, L, M) International Energy Conservation Code (see note 1) IFGC Committee Commercial Energy Committee	
IBC-Fire Safety (Chapters: 7, 8, 9, 14, 26 and App. D) definitions) IBC-General (Chapters: 2-6, 12, 13, 27-34, App. A, B, C, F, H, K) Administrative Code Committee IBC-Means of Egress (Chapters: 10, 11 and App. E) IBC-Structural (Chapters: 15-25 and App. G,I, J, L, M) International Fuel Gas Code International Energy Conservation Code (see note 1) IFGC Committee Commercial Energy Committee	1 gnated
IBC-General (Chapters: 2-6, 12, 13, 27-34, App. A, B, C, F, H, Administrative Code Committee IBC-Means of Egress (Chapters: 10, 11 and App. E) IBC-Structural (Chapters: 15-25 and App. G,I, J, L, M) IBC-nears of Egress (Chapters: 15-25 and App. G,I, J, L, M) International Energy Conservation Code (see note 1) IFGC Committee Commercial Energy Committee	
IBC-Means of Egress (Chapters: 10, 11 and App. E) IBC-Structural (Chapters: 15-25 and App. G,I, J, L, M) International Fuel Gas Code IFGC Committee IFGC Committee	
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International Fuel Gas Code International Energy Conservation Code (see note 1) IFGC Committee Commercial Energy Committee	
IFGC Committee Commercial Energy Committee	
Residential Energy Committee	
International Mechanical Code International Existing Building Code	
IMC Committee IEBC Committee	
International Plumbing Code International Fire Code	
IPC Committee IFC Committee	
International Private Sewage Disposal Code International Green Construction Code Committees:	
IPC Committee IGCC—Energy/Water Committee (Chapters: 6 and 7)
IGCC—General Committee (Chapters:2-5, 8-11 and Append)	!
International Performance Code (see note 2)	
ICC Performance Code Committee	
International Property Maintenance Code	
IPMC/I2C Committee	
International Wildiand-Orban Internace Code	
IFC Committee	
International Zoning Code	
IPMC/IZC Committee	
International Residential Code Committees:	
IRC-B (Chapters: 1-10 and App. E, F, G,H, J, K, L, M	, O)
IRC-M/P (Chapters: 12-33 and App. I, P)	
International Swimming Pool and Spa Code	
ISPSC Committee	

NOTE:

 Residential Energy Committee is responsible for Chapter 11 of the IRC and the Residential Provisions of the IECC.
 In anticipation of minimal code change activity, a ICC Performance Committee has not been appointed. Any changes will be considered by the IFC Committee.

GROUP A CODE DEVELOPMENT COMMITTEE RESPONSIBILITIES

Some sections of the International Codes have a letter designation in brackets in front of them. For instance, Section 301.1.4 of the IEBC has a [B] in front of it, meaning that this section is the responsibility of one of the IBC Code Development Committees (in this case, IBC-S).

Code change proposals submitted for such code sections that have a bracketed letter designation in front of them will be heard by the respective committee responsible for such code sections. Because different committees will meet in different years, some proposals for a given code will be heard by a committee in a different year than the year in which the primary committee for this code meets.

Note that there are several code change proposals in the IBC-Structural hearing order that are changes to the International Existing Building Code (marked with prefix "EB"). These are proposed changes to sections of the existing building code that are the responsibility of the IBC-Structural Code Development Committee. A complete summary of the Group A and Group B Code Development Committees' responsibilities can be view at the ICC Website: <u>http://www.iccsafe.org/cs/codes/Documents/2012-13cycle/GroupA-B CDC-Responsibilities.pdf</u>.

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. Section 3.6.3.1 of CP #28 now requires that a code change proposal will not be processed unless a consensus draft of the standard has been provided. Proponents of code changes which propose a new standard have been directed to forward copies of the standard to the Code Committee. An analysis statement will be posted on the ICC website providing information regarding standard content, such as enforceable language, references to proprietary products or services, and references to consensus procedure. The analysis statements for referenced standards will be posted on or before March 28th, 2012. This information will also be published and made available at the hearings.

REFERENCED STANDARDS UPDATES

Administrative updates of any standards already referenced in any of the I-Codes will be contained in a code change proposal for consideration by the Administrative Code Development Committee. The Administrative Code Development Committee is a Group B committee which will conduct hearings on the administrative provisions (Chapter 1 and certain definitions) of all I-Codes, and the referenced standards update. Therefore, this committee will conduct its code development hearing during the code development hearings in 2013.

It should be noted that, in accordance with Section 4.5.1 of CP #28 (see page xvi), standards promulgators will have until December 1, 2014 to finalize and publish any updates to standards in the administrative update. If the standard update is not finalized and published by December 1, 2014, the respective I-Codes will be revised to reference the previously listed year edition of the standard.

MODIFICATIONS

Those who are submitting a 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 <u>in advance</u> – the form is at

<u>http://www.iccsafe.org/cs/codes/publicforms.htm</u>. 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, modifications, 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 16 point.

Example:

Original code change proposal.

The original code change proposal requested the following change to Section 305.3 of one of our I-Codes: (Note that the example is fictional.)

G10-12 305.13

Proponent: John West representing self

Revise as follows:

305.3 Interior surfaces. All <u>interior</u> surfaces, including windows and doors, shall be maintained in good <u>and clean</u> condition. Peeling, chipping, flaking or abraded paint shall be repaired, removed or covered. Cracked or loose plaster, decayed wood and other defective surface conditions shall be corrected. <u>Surfaces of porous materials made of or</u> <u>containing organic materials, such as but not limited to wood, textiles, paint, cellulose insulation, and paper, including paper-faced gypsum board, that have visible signs of mold or mildew shall be removed and replaced or remediated in an <u>approved manner.</u></u>

Exception: Porous materials that do not contain organic materials, such as clean unpainted bricks and concrete.

Proposed modification:

A modification to the code change proposal is proposed:

- 1. To add "and sanitary" after "clean" in the first sentence.
- 2. To add "or water permeable" after "porous" in the third sentence.
- 3. Delete "in an approved manner." in the last sentence.
- 4. Delete the proposed new exception.

The modification should read as follows. Note that the font style is Ariel, and the font size is 16 pt. The cross out, <u>underline</u> format is removed from the text of the original proposal and the requested revisions in the original proposal are made and shown as original text. The modification to the original proposal is shown with cross out, <u>underline</u> format applied to the changes proposed in the modification.

Example of proposed modification:

G10-12 305.13

Proponent: Sam Sumter representing self

Modify the proposal as follows:

305.3 Interior surfaces. All interior surfaces, including windows and doors, shall be maintained in good, and clean and sanitary condition. Peeling, chipping, flaking or abraded paint shall be repaired, removed or covered. Cracked or loose plaster and other defective surface conditions shall be corrected. Surfaces of porous or water permeable materials made of or containing organic materials, such as but not limited to wood, textiles, paint, cellulose insulation, and paper, including paper-faced gypsum board, that have visible signs of mold or mildew shall be removed and replaced or remediated in an approved manner.

Exception: Porous materials that do not contain organic materials, such as clean unpainted bricks and concrete.

Note: The modification should be able to be shown on the overhead screen on a single page. Only show the pertinent part of the code change proposal that shows the intended revisions. The entire code change proposal need not be shown.

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 are shown at the end of this document (CCC-1).

ICC WEBSITE – <u>WWW.ICCSAFE.ORG</u>

This document is posted on the 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, will be identified in updates posted prior to the Code Development Hearings on the ICC website at http://www.iccsafe.org. Users are encouraged to periodically review the ICC Website for updates to the 2012/2013 Code Development Cycle-Group A (2012) 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.

PROPONENT CONTACT INFORMATION

For most of the code change proposals, an e-mail address for the proponent has been provided. ICC PUBLIC HEARING ::: April – May, 2012

2012/2013 ICC CODE DEVELOPMENT SCHEDULE

	DATE		
STEP IN CODE DEVELOPMENT CYCLE	2012 – Group A Codes IBC, IFGC, IMC, IPC, IPSDC (See Notes)	2013 – Group B Codes Admin, ICCPC, IEBC, IECC, IFC, IgCC, IPMC, ISPSC, IRC, IWUIC, IZC (See Notes)	
2012 EDITION OF I-CODES PUBLISHED	April :	30, 2011	
DEADLINE FOR RECEIPT OF APPLICATIONS FOR ALL CODE COMMITTEES	June 1, 2011 (updated to July 1 for IECC and IRC – Energy; Aug 1 for IgCC and ISPSC)		
DEADLINE FOR RECEIPT OF CODE CHANGE PROPOSALS	January 3, 2012	January 3, 2013	
WEB POSTING OF "PROPOSED CHANGES TO THE I- CODES"	March 12, 2012	March 11, 2013	
DISTRIBUTION DATE OF "PROPOSED CHANGES TO THE I-CODES" (CD only)	April 2, 2012	April 1, 2013	
CODE DEVELOPMENT HEARING (CDH)	April 29 – May 6, 2012 Sheraton Dallas Hotel Dallas, TX	April 21 – 28, 2013 Sheraton Dallas Hotel Dallas, TX	
WEB POSTING OF "REPORT OF THE PUBLIC HEARING"	June 8, 2012	May 31, 2013	
DISTRIBUTION DATE OF "REPORT OF THE PUBLIC HEARING" (CD only)	June 29, 2012	June 21, 2013	
DEADLINE FOR RECEIPT OF PUBLIC COMMENTS	August 1, 2012	July 15, 2013	
WEB POSTING OF PUBLIC COMMENTS "FINAL ACTION AGENDA"	September 10, 2012	August 28, 2013	
DISTRIBUTION DATE OF PUBLIC COMMENTS "FINAL ACTION AGENDA" (CD only)	October 1, 2012	September 16, 2013	
FINAL ACTION HEARING (FAH)	October 24 – 28, 2012 Oregon Convention Center Portland, OR	October 2 – 9, 2013 Atlantic City Convention Center Atlantic City, NJ	
ANNUAL CONFERENCES	October 21 – 24, 2012 Oregon Convention Center Portland, OR	September 29 – October 2, 2013 Atlantic City Convention Center Atlantic City, NJ	

Notes:

- Be sure to review the "Group A and Group B Code Development Committee Responsibilities" posted at <u>www.iccsafe.org/responsibilities</u> which identifies committee responsibilities which are different than Group A and Group B codes which may impact the applicable code change cycle and resulting code change deadline.
- The International Green Construction Code (IgCC) and International Swimming Pool and Spa Code (ISPSC) to undergo a full cycle of code development in 2011 resulting in 2012 editions published in March/2012
- Group B "Admin" includes code change proposals submitted to Chapter 1 of all the I-Codes except the ICCPC, IECC and IRC and the
 administrative update of referenced standards in the 2012 I-Codes
- Start 2015/2016 Code Development Cycle with Group A code change proposals due January 5, 2015

2012/2013 STAFF SECRETARIES

GROUP A (2012)

IBC-Fire Safety	IBC-General	IBC-Means of Egress	IBC-Structural
Chapters 7, 8, 9, 14, 26	Chapters 1-6, 12, 13, 27-34	Chapters 10, 11	Chapters 15-25
Ed Wirtschoreck	BethTubbs	Kim Paarlberg	Alan Carr
ICC Chicago District Office	ICC Northbridge Field Office	ICC Indianapolis Field Office	ICC NW Resource Center
1-888-ICC-SAFE, ext 4317	1-888-ICC-SAFE, ext 7708	1-888-ICC-SAFE, ext 4306	1-888-ICC-SAFE, ext 7601
FAX: 708/799-0320	FAX: 419/ 730-6531	FAX: 708/799-0320	FAX: 425/637-8939
ewirtschoreck@iccsafe.org	btubbs@iccsafe.org	kpaarlberg@iccsafe.org	acarr@iccsafe.org
IFGC	ІМС	IPC/IPSDC	
Gregg Gress	Gregg Gress	Fred Grable	
ICC Chicago District Office	ICC Chicago District Office	ICC Chicago District Office	
1-888-ICC-SAFE, ext 4343	1-888-ICC-SAFE, ext 4343	1-888-ICC-SAFE, ext 4359	
FAX: 708/799-0320	FAX: 708/799-0320	FAX: 708/799-0320	
ggress@iccsafe.org	ggress@iccsafe.org	fgrable@iccsafe.org	

GROUP B (2013)

ADMINISTRATIVE Chapter 1 All Codes Except IRC	IEBC	IECC-Commercial	IECC-Residential
Kim Paarlberg ICC Indianapolis Field Office 1-888-ICC-SAFE, ext 4306 FAX: 708/799-0320 kpaarlberg@iccsafe.org	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 dbowman@iccsafe.org	Dave Bowman ICC Chicago District Office 1-888-ICC-SAFE, ext 4323 FAX: 708/799-0320 dbowman@iccsafe.org
IFC	lgCC-General	IgCC-Energy/Water	ICC PC
Bill Rehr/ Beth Tubbs ICC Chicago District Office 1-888-ICC-SAFE, ext 4342 FAX: 708/799-0320 <u>brehr@iccsafe.org</u> <u>btubbs@iccsafe.org</u>	Allan Bilka ICC Chicago District Office 1-888-ICC-SAFE, ext 4326 FAX: 708/799-0320 abilka@iccsafe.org	Fred Grable ICC Chicago District Office 1-888-ICC-SAFE, ext 4359 FAX: 708/799-0320 fgrable@iccsafe.org	BethTubbs ICC Northbridge Field Office 1-888-ICC-SAFE, ext 7708 FAX: 419/ 730-6531 <u>btubbs@iccsafe.org</u>
IPMC	IRC-Building	IRC Mechanical	IRC Plumbing
Ed Wirtschoreck ICC Chicago District Office 1-888-ICC-SAFE, ext 4317 FAX: 708/799-0320 ewirtschoreck@iccsafe.org	Larry Franks/ Dave Bowman ICC Birmingham District Office 1-888-ICC-SAFE, ext 5279 FAX: 205/592-7001 Ifranks@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
ISPSC	IWUIC	IZC	
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	Ed Wirtschoreck ICC Chicago District Office 1-888-ICC-SAFE, ext 4317 FAX: 708/799-0320 ewirtschoreck@iccsafe.org	

COMMITTEE A ASSIGNMENT CROSSOVER LIST—WITHIN THE IBC

The 2012/2013 Staff Secretaries assignments on page x 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 403.5 deal with means of egress from high-rise buildings. Therefore, even though Chapter 4 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 2012 portion (Group A) of the 2012/2013 Cycle.

SECTION	CHAPTER	SECTION	CODE CHANGE
	MAINTAINED BY	MAINTAINED BY	PROPOSALS
403.5	IBC-General	IBC-Means of Egress	E4, E7
405.7.1	IBC-General	IBC-Means of Egress	E3
411.7	IBC-General	IBC-Means of Egress	E3
1508.1	IBC-Structural	IBC-Fire Safety	FS178
3401.2	IBC-General	IBC-Structural	S90
3406.1.3	IBC-General	IBC-Means of Egress	E4
3406.4	IBC-General	IBC-Means of Egress	E4
3411.8.4	IBC-General	IBC-Means of Egress	E4
3411.8.15	IBC-General	IBC-Means of Egress	E211



CP# 28-05 CODE DEVELOPMENT

Approved:	9/24/05
Revised:	10/29/11

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 public officials actively engaged in the administration, formulation or enforcement of laws, ordinances, rules or regulations relating to the public health, safety and welfare 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 Recording:** Individuals requesting permission to record any meeting or hearing, or portion thereof, shall be required to provide the ICC with a release of responsibility disclaimer and shall acknowledge that ICC shall retain sole ownership of the recording, and that they have insurance coverage for liability and misuse of recording materials. Equipment and the process used to record 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 recording. An unedited copy of the recording shall be forwarded to ICC within 30 days of the meeting. Recordings shall not otherwise be copied, reproduced or distributed in any manner. Recordings shall be returned to

ICC or destroyed upon the request of ICC.

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:

- **2.4.1 Scope:** Emergency actions are limited to those issues representing an immediate threat to health and safety that warrant a more timely response than allowed by the Code Development Process schedule.
- **2.4.2** Initial Request: A request for an emergency action shall be based upon perceived threats to health and safety and shall be reviewed by the ICC Codes and Standards Council for referral to the Board of Directors for action with their analysis and recommendation.
- **2.4.3 Board and Member Action:** 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. Email addresses shall be published with the code change proposals unless the proponent otherwise requests on the submittal form.
 - **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 may 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. All substantiating material published by ICC is material that has been provided by the proponent and in so publishing ICC makes no representations or warranties about its quality or accuracy.
 - **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. The proponent should submit information that supports their claim. Any information submitted will be considered by the code development committee. This information will be included in the bibliography of 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. If a new standard is not submitted in at least draft form, the code change shall be considered incomplete and shall not be processed. 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 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 of the third year of each code cycle. The published version of the new edition of the 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.
- **4.6 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 Board of Directors.
 - **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. A committee member who is a proponent of a proposal shall not participate in any committee discussion on the matter or any committee discussion on the matter or 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). Participants shall not advocate a position on specific code changes with Committee Members other than through the methods provided in this policy.
 - **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. Each individual presenting information at the hearing shall state their name and affiliation, and shall identify any entities or individuals they are representing in connection with their testimony. 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. Re-rebuttal 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.
 - **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: A successful assembly action shall be a majority vote of the votes cast by eligible voters (See 5.7.4).
 - **5.7.4 Eligible Voters:** All members of ICC in attendance at the public hearing shall be eligible to vote on floor motions. Each member is entitled to one vote, except that each Governmental Member Voting Representative in attendance may vote on behalf of its Governmental Member. 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. Email addresses shall be published with the public comments unless the commenter otherwise requests on submittal form. 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 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 may 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. All substantiating material published by ICC is material that has been provided by the proponent and in so publishing ICC makes no representations or warranties about its quality or accuracy.

- **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. Each individual presenting information at the hearing shall state their name and affiliation, and shall identify any entities or individuals they are representing in connection with their testimony. 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.
 - **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 for Governmental Membership must be received by the ICC by April 1st of the applicable year in order for its designated representatives to be eligible to vote at the Final Action Hearing. Applications, whether new or updated, for governmental member voting representative status must be received by the Code Council thirty (30) 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. An individual designated as a Governmental Member Voting Representative shall provide sufficient information to establish eligibility as defined in the ICC Bylaws. The Executive Committee of the ICC Board, in its discretion, shall have the authority to address questions related to eligibility. Decisions of the Executive Committee shall be final and not appealable pursuant to CP 1, other than claims of fraud or misrepresentation, supported by reasonably credible evidence, that were material to the outcome of the Final Action Hearing.

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:

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

- **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.

2012 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 2012/2013 Staff Secretaries on page x. 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 703.2.3 is proposed for revision in code change S70-12, which is to be heard by the IBC Structural Committee. This section of the IBC is typically the responsibility of the IBC Fire Safety Committee as listed in the table of 2012/2013 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 E4-12 which will be placed on the IBC Means of Egress 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 portion of the 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
CE	International Energy Conservation Code – Commercial
RE	International Energy Conservation Code – Energy
F	International Fire Code
FG	International Fuel Gas Code
FS	International Building Code - Fire Safety
G	International Building Code – General
GEW	International Green Construction Code – Energy/Water
GG	International Green Construction Code – General
Μ	International Mechanical Code
PC	ICC Performance Code
Р	International Plumbing Code
PSD	International Private Sewage Disposal Code
PM	International Property Maintenance Code
RE	International Residential Code - Building
RM	International Residential Code - Mechanical
RP	International Residential Code - Plumbing
S	International Building Code – Structural
SP	International Swimming Pool and Spa Code
WUIC	International Wildland-Urban Interface Code
Z	International Zoning Code

Internetional Building Co	da	907.2.10.1	G71
International Building Co	de	907.2.13.2	E4
101.4	G201	907.5.2.2	E4
101.4.7 (New)	G201	909.4.6	G32 Part II
104.11.3 (New)	FS73	909.9	S70
107.2.6	G198	909.18	S113, S117
110.3.5	S304	909.20	E4, E5
116.5	G201	909.21.7	S113
202	P27, P29	911.1.5	E4
403.5	E4, E7	1003.2	G62
404.6	FS41, FS99	Table 1004.1.2	G193
405.7.1	E3	1004.3	S90
410.6.1	E3	1005.7.2	G73
411.7	E3	1007.1	G237
414.7.2	E3	1007.6	G57
505.2.3	E7	1009.3	FS51, FS99
505.3	E101	1015.2.1	G85
703.2.3	S70	1015.4	G57
706.1	G103	1015.5	G57
707.5.1	E7	Table 1016.2	G32 Part I, G87
707.6	E4	1018.1	G31 Part I
707.7.1	E4	Table 1018.1	G32 Part I
709.5	G31 Part I	Table 1018.2	G32 part I
710.8	G32 Part I	1018.4	G32 Part I
711.4	E7	Table 1021.2(2)	G57
712.1.8	G32 Part I, G54, E7	1022.7	G85
712.1.12	E7	1027.1	G175
713.1	E4, E7	1203.1	M36, M37, M38, M39
713.14.1	G32 Part I, E110	1205.4	E4
713.14.1.2 (new)	G174 Part III	1207.1	E4
Table 716.5	G51, E4	1403.7	S102, S103
716.5.3	E3	1404.13 (New)	S309
717.5.5	G32 Part I	1507.16	G98
718.2.4	E4	1507.16.1	G98
722.5	S238	1508.1	FS178
Table 803.9	E4	1609.1.2	G199
901.5	S90	1808.7.3	G193
903.2.6	G31 Part II, G32 Part II	2103.15(New)	FS177
903.2.8	G31 Part II	2110.1.1	E4
903.2.8.1	G31 Part II	2303.1.4 (new)	G142 Part II
903.2.8.2 (new)	G31 Part II	2308.12.7	E4
903.2.8.2	G31 Part II	2405.3	G199
903.2.8.3 (new)	G31 Part II	2406.4	G193
903.3.1.3	G31 Part II	2406.4.5	G193
903.3.2	G32 Part II	2406.4.6	E4
905.3.3	E4	2406.4.7	E4
905.4	E4	2607.4	G199
906.2	G71	2609.4	G193, G199
Table 906.3(1)	G71	Table 2902.1.2 (New)	P27
Table 906.3(2)	G71	2902.2	P34
907.2.6	G32 Part II, G71	2902.3	P35
907.2.6.1	G31 Part II	2902.3.1	P36
907.2.6.4 (new)	G32 Part II	2902.3.5	P37

International Building Code (continued)		1003.2	G62
		Table 1004.1.2	G193
2902.4.1	P39	1005.7.2	G73
2902.6 (New)	P30	1007.1	G237
Table 2902.1.2 (New)	P27	1007.6	G57
3007.7	E110	1015.2.1	G85
3007.9	FS138	1015.4	G57
3008.7	E110	1015.5	G57
3008.9	FS138	Table 1016.2	G32 Part I, G87
3111.1	S3	1018.1	G31 Part I
3306.8	S90	Table 1018.1	G32 Part I
3311.1	E4	Table 1018.2	G32 part I
3401.2	S90	1018.4	G32 Part I
3406.1.3	E4	Table 1021.2(2)	G57
3406.4	E4	1022.7	G85
3411.8.4	E4	1027.1	G175
3411.8.15	E211	1104.6.1	E4
International Fire Code		1104.9	E4
International Fire Code		1104.10	E4
202	G1, G2, G11, G13, G31	1104.12	E4
	Part I, G32 Part I, G43,		
	G70		
Definition of Group A	G27	1104.16	E4
Definition of Group B	G28, G29, G30	1104.20	E4
Definition of Group E	G27	1104.21	E4
Definition of Group I	G31 Part I, G32 Part I,	1104.23	E4
	G33, G34, G35, G36,		
	G37		
Definition of Group R	G31 Part I, G34, G36,	3313.1	E4
	G38, G39, G40, G41		
Definition of Group S	G42	5005.4.4	E3
508.1.5	E4	5704.2.9.4	E4
604.2.16 (new)	G77	5706.5.1.12	E4
903.2.6	G31 Part II, G32 Part II	INTERNATIONAL PLUME	BING CODE
903.2.8	G31 Part II		
903.2.8.1	G31 Part II	202	G8, G193 Part IV,
			P3(HEARD BY IBC-S)
903.2.8.2 (new)	G31 Part II	309.2	P20 (HEARD BY IBC-S)
903.2.8.2	G31 Part II	403.3.3	G/1
905.3.3	E4	403.3.4	G/1
905.4		403.5	G/1
903.2.8.3 (new)	G31 Part II	423.1	G193 Part IV
903.3.1.3	G31 Part II	612.1	G193 Part IV
903.3.2	G32 Part II	801.1	G193 Part IV
906.2	G71	802.1.4	G193 Part IV
Table 906.3(1)	G71	INTERNATIONAL MECHA	ANICAL CODE
1 able 906.3(2)	G71		
907.2.6	G32 Part II, G71	202	G8
907.2.6.1	G31 Part II	304.11	E108
907.2.6.4 (new)	G32 Part II	306.5.1	
907.2.10.1	G71	403.2.1	G193 Part II
907.2.13.2		Table 403.3	G193 Part II
907.5.2.2	E4	601.3	E228, E229
909.4.6	G32 Part II	901.5	FG3

		406.1	G225, G226
		410.5.1(new)	G235
901.6	FG3	410.6	G235, G236, G237
926.2	FG38	410.7	G237, G238, G240
926.3	FG38	410.7.1	G240
1107.2	E4	410.8	G239, E211
1401.1	G193 Part II	410.8 (new)	G237
		410.8.1 (new)	G237
INTERNATIONAL FUEL G	JAS CODE	410.8.4	G241
202	G8	410.8.6	G242
306.5.1	E4	410.8.9	G235, G236
614.6	M71	410.8.11	G243
Section 617	G193 Part III	606.2.2	G221 Part II
617.1	G193 Part III	606.2.3.1	G224 Part II
629.1	M169	907.2	G213 Part II
		907.2.1	G213 Part II
	E SEWAGE DISPOSAL	907.2.2	G213 Part II
CODE		1401.2	G244
202	G8, P228 (HEARD BY IBC-S)	1401.2.5	G245
401.3.2	G193 Part IV	1401.3.2	G246
Table 406.1	G193 Part IV	Table 1401.3.2(new)	G246
Table 604.1(2)	G193 Part IV	1401.6	G244
Table 802.7.2	G193 Part IV	1401.6.1	G101
Table 802.8	G193 Part IV	1401.6.1.1	G101
		1401.6.2	G101, G244
	NG BUILDING CODE	1401.6.2.1	G101
Chapters 3 through 14	G205	1401.6.4	G244
202	G23, G24	Table 1401.6.4	G244
402.1	G210	1401.6.5	G244
402.4	G211	Table 1401.6.5	G57, G244
403.1	G210, G212	1401.6.6	G51
403.3 (new)	G213 Part I	1401.6.7	G244
403.3.1 (new)	G213 Part I	1401.6.8	G244
403.3.2 (new)	G213 Part I	Table 1401.6.8	G244
403.3.3 (new)	G213 Part I	1401.6.8.1	G244
403.4	G211	1401.6.9	G244
403.4.1(new)	G214	Table 1401.6.9	G244
403.4.5 (new)	G215, G216, G217	1401.6.10	G244
403.5 (new)	G218	Table 1401.6.10	G244
403.7 (new)	G219	1401.6.11	G244
403.7.1(new)	G219	Table 1401.6.11	G244
403.7.2(new)	G219	1401.6.12	G244
403.7.3(new)	G219	Table 1401.6.12	G244
404.1	G212	1401.6.12.1	G244
404.2 (new)	G220	1401.6.16	G244
404.2	G221 Part I	1401.6.16.1	G244
404.2.1	G211	1401.6.17	G244
404.2.2	G222	Table 1401.6.17	G244
404.3	G223	1401.6.18	G244
404.3.1	G224 Part I	Table 1401.6.18	G244
404.4	G222	1401.6.20 (new)	G244
404.2.3	G211, G212	Table 1401.6.20 (new)	G244
404.5	G212	1401.6.21 (new)	G244

INTERNATIONAL EXISTING BUILDING CODE (continued)

Table 1401.6.21.1 (new)	G244
1401.6.21.1.1(new)	G244
1401.6.21.2(new)	G244
Table 1401.6.21.2(new)	G244
1401.6.21.2.1(new)	G244
1401.6.21.3(new)	G244
Table 1401.6.21.3	G244
1401.6.21.3.1(new)	G244
Table 1401.7	G244
1401.8	G244
Table 1401.8	G244

2012 GROUP A CODE DEVELOPMENT HEARING SCHEDULE April 29 – May 8, 2012 Sheraton Dallas Hotel

Unless noted by "Start no earlier than X am," 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.

Please note that the hearing start on Sunday, April 29th has been revised from 10:00 am to 12:00 pm from the originally posted version. Prior to the hearings starting at noon on Sunday, the following is also scheduled:

- Membership Councils: 8:00 am 10:00 am
- CDP ACCESS update (Expanding code development participation): 10:15 am 11:15 am

For more information on the scheduling of these two activities, be sure to check the link to the Member Committees page on the ICC Website: <u>http://www.iccsafe.org/membership/pages/committees.aspx</u>

	Sunday April 29	Monday April 30	Tuesday May 1	Wednesday May 2	Thursday May 3
	Start 12 pm	Start 8 am	Start 8 am	Start 8 am	Start 8 am
÷	IBC - FS	IBC - FS	IBC - FS	IBC - G	IBC – G
TRACK			IBC – G (Start no earlier than 8 am)		IBC - E (Start no earlier than 8 am)
	End 9 pm	End 9 pm	End 9 pm	End 9 pm	End 9 pm
	Start 12 pm	Start 8 am	Start 8 am	Start 8 am	Start 8 am
	IFGC	IPC/IPSDC	IPC/IPSDC	IMC	ІМС
TRACK 2	IPC/IPSDC		IMC (Start no earlier than 8 am)		IEBC – S (Start no earlier than 8 am) IBC – S
	End 9 pm	End 9 pm	End 9 pm	End 9 pm	End 9 pm

	Friday May 4	Saturday May 5	Sunday May 6	Monday May 7	Tuesday May 8
	Start 8 am	Start 8 am	Start 8 am		
TRACK 1	IBC - E	IBC – E	IBC – E		
	End 9 pm	End 9 pm	Finish 12 pm		
	Start 8 am	Start 8 am	Start 8 am	Start 8 am	Start 8 am
TRACK 2	IBC - S	IBC - S	IBC - S	IBC - S	IBC – S
	End 9 pm	End 9 pm	End 9 pm	End 9 pm	Finish 12 pm

Notes:

1. IEBC – S: Structural provisions in the IEBC to be heard by the IBC – Structural Code Committee.

2. Hearing times may be modified at the discretion of the Chairman.

3. Breaks will be announced. Lunch and dinner breaks planned for each track. There will not be a lunch break on Sunday, April 29th.

2012 PROPOSED CHANGES TO THE INTERNATIONAL CODES

CODE

<u>PAGE</u>

International Building Code	
Fire Safety	FS1
General	G1
Means of Egress	E1
Structural (Including portions of International Existing Building Code)	S1
International Fuel Gas Code	FG1
International Plumbing Code	P1
International Mechanical Code	M1
Code Correlation Committee	CCC1

2012 PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE – STRUCTURAL

(Portions of the International Existing Building Code will be heard by the Structural Committee)

STRUCTURAL CODE COMMITTEE

Marcelino Iglesias - Chair Code Specialist State of New Jersey – DCA – Div. of Codes & Standards Trenton, NJ

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Staff Secretariat:

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TENTATIVE ORDER OF DISCUSSION 2012 PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE

STRUCTURAL

(Including portions of the International Existing Building Code)

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. Note that some IBC-S code change proposals may not be included on this list, as they are being heard by other committees. Please consult the Cross Index of Proposed Changes.

IEBC	G213-12, Part I	G228-12	S29-12
EB15-12	G213-12, Part II	EB11-12	S30-12
EB16-12	EB1-12		S31-12
EB17-12	EB2-12	IBC-S	S32-12
EB18-12	EB3-12	S1-12	S37-12
EB19-12	EB12-12	S2-12	S33-12
EB20-12	EB13-12	S3-12	S34-12
EB21-12	EB14-12	S5-12	S35-12
EB22-12	G211-12	S46-12	S36-12
EB23-12	G212-12	G20-12	S38-12
EB24-12	G214-12	G21-12	S39-12
EB25-12	EB4-12	S4-12	S40-12
EB26-12	EB5-12	S68-12	S41-12
EB27-12	G218-12	S6-12	S28-12
EB28-12	EB6-12, Part I	S7-12	S42-12
EB29-12	EB6-12, Part II	S8-12	S43-12
EB30-12	EB7-12	S9-12	S44-12
EB31-12	EB8-12	S10-12	S45-12
EB32-12	EB9-12	S11-12	S47-12
EB33-12	G216-12	S12-12	S48-12
EB34-12	EB10-12	S13-12	S53-12
EB35-12	G215-12	S14-12	S59-12
EB36-12	G217-12	S15-12	S60-12
EB37-12	G221-12, Part I	S16-12	S61-12
EB38-12	G221-12, Part II	S17-12	S62-12
EB39-12	G222-12	S18-12	S63-12
EB40-12	G223-12	S25-12	S64-12
EB41-12	G224-12, Part I	S26-12	S66-12
EB42-12	G224-12, Part II	S27-12	S67-12

S70-12 S324-12 S140-12 S189-12 S71-12 S325-12 S141-12 S190-12 S72-12 S326-12 S142-12 S191-12 S73-12 S327-12 S143-12 S192-12 S74-12 S328-12 S144-12 S193-12 S74-12 S328-12 S144-12 S193-12 S75-12 S329-12 S145-12 S194-12 S76-12 S330-12 S146-12 S196-12 S77-12 S331-12 S148-12 S197-12 S78-12 S333-12 S148-12 S197-12 S78-12 S333-12 S148-12 S199-12 S81-12 S334-12 S150-12 S200-12 S82-12 S336-12 S152-12 S201-12 S82-12 S107-12 S153-12 S202-12 S83-12 S108-12 S105-12 S204-12 S84-12 S109-12 S156-12 S205-12 S87-12 S108-12 S204-12 S206-12 S89-12	S69-12	S323-12	S139-12	S188-12
S71-12 S325-12 S141-12 S190-12 S72-12 S326-12 S142-12 S191-12 S73-12 S327-12 S143-12 S192-12 S74-12 S328-12 S144-12 S193-12 S75-12 S329-12 S145-12 S194-12 S76-12 S330-12 S146-12 S195-12 S77-12 S331-12 S147-12 S196-12 S78-12 S333-12 S144-12 S196-12 S78-12 S333-12 S144-12 S196-12 S78-12 S333-12 S144-12 S198-12 S78-12 S333-12 S144-12 S198-12 S80-12 S338-12 S150-12 S199-12 S81-12 S336-12 S152-12 S202-12 S84-12 S107-12 S154-12 S203-12 S86-12 S109-12 S155-12 S204-12 S86-12 S109-12 S155-12 S206-12 S88-12 S111-12 S158-12 S207-12 S88-12 S111-12 S156-12 S206-12 S90-12 S335-12	S70-12	S324-12	S140-12	S189-12
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S79-12 S333-12 S149-12 S198-12 S80-12 S318-12 S150-12 S199-12 S81-12 S334-12 S151-12 S200-12 S82-12 S336-12 S152-12 S201-12 S82-12 S336-12 S152-12 S201-12 S83-12 S337-12 S153-12 S202-12 S84-12 S107-12 S153-12 S202-12 S84-12 S107-12 S155-12 S204-12 S85-12 S108-12 S155-12 S204-12 S86-12 S109-12 S156-12 S205-12 S87-12 S110-12 S157-12 S206-12 S87-12 S111-12 S158-12 S207-12 S88-12 S111-12 S158-12 S207-12 S90-12 S335-12 S166-12 S209-12 S90-12 S112-12 S166-12 S209-12 S91-12 S113-12 S160-12 S211-12 S92-12 S113-12 S162-12 S212-12 S94-12 S114-12 S163-12 S213-12 S97-12 S115-12	S78-12	S332-12	S148-12	S197-12
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S85-12 S108-12 S155-12 S204-12 S86-12 S109-12 S156-12 S205-12 S87-12 S110-12 S157-12 S206-12 S88-12 S111-12 S158-12 S207-12 S89-12 S335-12 S159-12 S208-12 S90-12 S338-12 S166-12 S209-12 S91-12 S112-12 S160-12 S210-12 S92-12 G10-12 S161-12 S211-12 S93-12 S113-12 S162-12 S212-12 S94-12 S114-12 S163-12 S213-12 S95-12 S115-12 S164-12 S214-12 S96-12 S116-12 S165-12 S215-12 S97-12 S117-12 S169-12 S216-12 S98-12 S118-12 S167-12 S217-12 S98-12 S118-12 S167-12 S216-12 S98-12 S118-12 S167-12 S216-12 S98-12 S118-12 S167-12 S216-12 S98-12 S118-12 S167-12 S216-12 S98-12 S118-12	S84-12	S107-12	S154-12	S203-12
S86-12 S109-12 S156-12 S205-12 S87-12 S110-12 S157-12 S206-12 S88-12 S111-12 S158-12 S207-12 S89-12 S335-12 S159-12 S208-12 S90-12 S338-12 S166-12 S209-12 S91-12 S112-12 S160-12 S210-12 S92-12 G10-12 S161-12 S211-12 S93-12 S113-12 S162-12 S212-12 S94-12 S114-12 S162-12 S212-12 S94-12 S114-12 S163-12 S214-12 S95-12 S115-12 S164-12 S214-12 S96-12 S116-12 S165-12 S215-12 S97-12 S117-12 S169-12 S216-12 S98-12 S118-12 S167-12 S217-12 S98-12 S118-12 S167-12 S216-12 S98-12 S118-12 S167-12 S216-12 S98-12 S118-12 S168-12 S218-12 S100-12 S120-12 S170-12 S218-12 S100-12 S120-12	S85-12	S108-12	S155-12	S204-12
S87-12 S110-12 S157-12 S206-12 S88-12 S111-12 S158-12 S207-12 S89-12 S335-12 S159-12 S208-12 S90-12 S338-12 S166-12 S209-12 S91-12 S112-12 S160-12 S210-12 S92-12 G10-12 S161-12 S211-12 S93-12 S113-12 S162-12 S212-12 S93-12 S113-12 S162-12 S212-12 S94-12 S114-12 S163-12 S213-12 S95-12 S115-12 S164-12 S214-12 S96-12 S116-12 S165-12 S215-12 S97-12 S117-12 S169-12 S216-12 S98-12 S118-12 S167-12 S217-12 S98-12 S118-12 S168-12 S218-12 S100-12 S120-12 S168-12 S218-12 S100-12 S120-12 S170-12 S219-12 S100-12 S120-12 S170-12 G3-12 FS146-12 S122-12 S172-12 G7-12 S102-12 S123-12	S86-12	S109-12	S156-12	S205-12
S88-12 S111-12 S158-12 S207-12 S89-12 S335-12 S159-12 S208-12 S90-12 S338-12 S166-12 S209-12 S91-12 S112-12 S160-12 S210-12 S92-12 G10-12 S161-12 S211-12 S93-12 S113-12 S162-12 S212-12 S93-12 S114-12 S162-12 S212-12 S94-12 S114-12 S163-12 S213-12 S95-12 S115-12 S164-12 S214-12 S96-12 S116-12 S165-12 S216-12 S97-12 S117-12 S169-12 S216-12 S97-12 S118-12 S167-12 S216-12 S98-12 S118-12 S167-12 S216-12 S98-12 S119-12 S168-12 S218-12 S100-12 S120-12 S170-12 S219-12 S100-12 S121-12 S170-12 S219-12 S100-12 S122-12 S172-12 G7-12 S102-12 S123-12 S173-12 G12-12 S103-12 S124-12	S87-12	S110-12	S157-12	S206-12
S89-12 S335-12 S159-12 S208-12 S90-12 S338-12 S166-12 S209-12 S91-12 S112-12 S160-12 S210-12 S92-12 G10-12 S161-12 S211-12 S93-12 S113-12 S162-12 S211-12 S93-12 S113-12 S162-12 S212-12 S94-12 S114-12 S163-12 S213-12 S94-12 S114-12 S163-12 S213-12 S94-12 S114-12 S163-12 S213-12 S94-12 S114-12 S163-12 S214-12 S95-12 S115-12 S164-12 S214-12 S96-12 S116-12 S165-12 S215-12 S97-12 S117-12 S169-12 S216-12 S98-12 S118-12 S167-12 S216-12 S98-12 S119-12 S168-12 S216-12 S98-12 S119-12 S168-12 S216-12 S100-12 S120-12 S170-12 S219-12 S100-12 S120-12 S170-12 S219-12 I00-12 S122-12	S88-12	S111-12	S158-12	S207-12
S90-12 S338-12 S166-12 S209-12 S91-12 S112-12 S160-12 S210-12 S92-12 G10-12 S161-12 S211-12 S93-12 S113-12 S162-12 S212-12 S94-12 S113-12 S162-12 S212-12 S93-12 S113-12 S162-12 S212-12 S94-12 S114-12 S163-12 S213-12 S95-12 S115-12 S164-12 S214-12 S96-12 S116-12 S165-12 S215-12 S97-12 S117-12 S169-12 S216-12 S98-12 S118-12 S167-12 S217-12 S98-12 S119-12 S168-12 S218-12 S100-12 S120-12 S170-12 S218-12 S100-12 S120-12 S170-12 S219-12 S101-12 S121-12 S171-12 G3-12 FS146-12 S122-12 S172-12 G7-12 S102-12 S123-12 S173-12 G12-12 S102-12 S123-12 S174-12 S220-12 I02-12 S124-12	S89-12	S335-12	S159-12	S208-12
S91-12 S112-12 S160-12 S210-12 S92-12 G10-12 S161-12 S211-12 S93-12 S113-12 S162-12 S212-12 S94-12 S114-12 S163-12 S213-12 S95-12 S115-12 S164-12 S214-12 S96-12 S116-12 S165-12 S215-12 S97-12 S117-12 S169-12 S216-12 S97-12 S117-12 S169-12 S216-12 S98-12 S118-12 S167-12 S217-12 S99-12 S119-12 S168-12 S218-12 S100-12 S120-12 S170-12 S219-12 S101-12 S121-12 S177-12 G3-12 FS146-12 S122-12 S172-12 G7-12 S102-12 S123-12 S173-12 G12-12 S103-12 S124-12 S174-12 S220-12 S104-12 S125-12 S175-12 S222-12 S104-12 S125-12 S175-12 S222-12	S90-12	S338-12	S166-12	S209-12
S92-12 G10-12 S161-12 S211-12 S93-12 S113-12 S162-12 S212-12 S94-12 S114-12 S163-12 S213-12 S95-12 S115-12 S164-12 S214-12 S96-12 S116-12 S165-12 S215-12 S97-12 S116-12 S165-12 S216-12 S97-12 S117-12 S169-12 S216-12 S97-12 S118-12 S167-12 S216-12 S98-12 S118-12 S167-12 S217-12 S99-12 S119-12 S168-12 S218-12 S100-12 S120-12 S170-12 S219-12 S101-12 S121-12 S171-12 G3-12 FS146-12 S122-12 S172-12 G7-12 S102-12 S123-12 S173-12 G12-12 S103-12 S124-12 S174-12 S220-12 S104-12 S125-12 S175-12 S222-12 S104-12 S125-12 S175-12 S222-12	S91-12	S112-12	S160-12	S210-12
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P3-12 S127-12 S177-12 S224-12	P3-12	S127-12	S177-12	S224-12
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M31-12 S129-12 S294-12 S226-12	M31-12	S129-12	S294-12	S226-12
P20-12 S130-12 S179-12 S227-12	P20-12	S130-12	S179-12	S227-12
G23-12 S138-12 S180-12 S228-12	G23-12	S138-12	S180-12	S228-12
G24-12 S131-12 S181-12 S229-12	G24-12	S131-12	S181-12	S229-12
S105-12 S132-12 S182-12 S230-12	S105-12	S132-12	S182-12	S230-12
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S321-12 S136-12 S186-12 S234-12	S321-12	S136-12	S186-12	S234-12
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S236-12	S281-12
S237-12	S282-12
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S242-12	S287-12
S243-12	S288-12
S244-12	S289-12
S245-12	S290-12
S246-12	S291-12
G4-12	S292-12
G17-12	S202-12 S203-12
\$2/17_12	S205-12
S247-12	S206 12
S240-12 S240-12	S290-12 S207 12
S249-12 S250 12	S297-12 S209 12
0200-12 C140-10 Dort II	5290-12
9142-12, Fait II	S299-12 S200 12
0201-12	S300-12 S201 12
G20-12 S252 42	SJUI-12 S176 12
G25 12	S170-12 S202 12
920-12 9253-12	S302-12 S202 12
S255-12 S257-12	S303-12 S204 12
S204-12 S255 40	S304-12 S205 12
5200-12 S256 40	5305-1Z 5221-12
5250-12	5221-12
5257-12	5256-12
5259-12	5306-12
5200-12	5307-12
5261-12	5308-12
5262-12	S310-12
5263-12	S311-12
S264-12	5312-12
S265-12	FS168-12
S266-12	FS192-12
S267-12	FS193-12
S268-12	FS194-12
S269-12	FS195-12
S270-12	FS198-12
S271-12	S317-12
G16-12	S313-12
S272-12	S314-12
S273-12	S315-12, Part I
S274-12	S316-12, Part I
S275-12	G186-12
S276-12	G187-12
S277-12	G188-12
S278-12	G189-12
S279-12	
S280-12	
EB1–12 [B]301.1.4, [B]301.1.4.1, [B]Table 301.1.4.1, [B]301.1.4.2, [B]Table 301.1.4.2

Proponent: Jennifer Goupil, The Structural Engineering Institute of ASCE (jgoupil@asce.org)

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

Revise as follows:

[B] 301.1.4 Evaluation and design procedures. The seismic evaluation and design shall be based on the procedures specified in the *International Building Code*, ASCE 31 or ASCE 41. The procedures contained in Appendix A of this code shall be permitted to be used as specified in Section 301.1.4.2.

[B] 301.1.4.1 Compliance with IBC level seismic forces. Where compliance with the seismic design provisions of the *International Building Code* is required, the procedures shall be in accordance with one of the following:

- One-hundred percent of the values in the *International Building Code*. Where the existing seismic force-resisting system is a type that can be designated as "Ordinary," values of *R*, Ω₀ and C₀ used for analysis in accordance with Chapter 16 of the *International Building Code* shall be those specified for structural systems classified as "Ordinary" in accordance with Table 12.2-1 of ASCE 7, unless it can be demonstrated that the structural system will provide performance equivalent to that of a "Detailed," "Intermediate" or "Special" system.
- Compliance with the performance objectives in ASCE 41 using both the BSE-1 and BSE-2 earthquake hazard levels and the corresponding performance levels shown in Table 301.1.4.1 Section 2.2.4 based on the assigned Risk Category for the building.

[B] TABLE 301.1.4.1 PERFORMANCE CRITERIA FOR IBC—LEVEL SEISMIC FORCES OCCUPANCY

[B] 301.1.4.2 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 301.1.4.1 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 and subject to the limitations of the respective Appendix Chapters shall be deemed to comply with this section.
 - 2.1. The seismic evaluation and design of unreinforced masonry bearing wall buildings in 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 Risk Category I or II are permitted to be based on the procedures specified in Chapter A2.
 - 2.3. Seismic evaluation and design of cripple walls and sill plate anchorage in residential buildings of light-frame wood construction in 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 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 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 301.1.4.2. 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. <u>3.</u> Compliance with <u>the performance objectives in</u> ASCE 41 using the BSE-1 Earthquake Hazard Level and the performance level shown in Table 301.1.4.2. The design spectral response acceleration parameters S_{xx} and S_{xx} specified in ASCE 41 shall not be taken less than 75 percent of the respective design spectral response acceleration parameters S_{DS} and S_{D1} defined by the *International Building Code* Section 2.2.1 based on the assigned Risk Category for the building.

[B] TABLE 301.1.4.2

PERFORMANCE CRITERIA FOR REDUCED IBC-LEVEL SEISMIC FORCES RISK CATEGORY

Reason: This proposal has two primary purposes:

- 1. Replace references to ASCE 31-03 and 41-06 with the updated standard ASCE 41-13, which combined 31 and 41 and contains numerous technical updates, representing the state of the practice for seismic evaluation and rehabilitation of existing buildings.
- Remove IEBC Tables 301.1.4.1 and 301.1.4.2 and replace with a reference to the related sections of ASCE 41-13. The update standard contains performance objective criteria for both a new building standard equivalent level ("IBC-level seismic forces" in the IEBC), and a basic retrofit level ("reduced IBC-level seismic forces" in the IEBC).

Both of these purposes and a general summary of the changes associated with the new standard are presented below:

ASCE 41-13 Summary

ASCE 41-13 is the culmination of a multi-year, ANSI approved update process for the two seismic evaluation and rehabilitation standards promulgated by ASCE. There are several significant updates to the standards:

- ASCE 31-03 and 41-06 have been combined into one standard for improved consistency and usability. The primary
 features of the two standards have been maintained, including a three-tiered analysis approach; the use of simplified,
 experience-based approach for common building types; the use of advance analytical techniques for more complex or
 unusual buildings.
- Updated seismic hazard and performance objectives, including the addition of a "new building standard equivalent" performance and a change in the seismic hazard determination of the basic performance objective for existing buildings. The new building equivalent utilizes the same seismic hazards as ASCE 7-10. The existing building performance has removed the 0.75 factors on demands that has traditionally been used and instead uses reduced seismic hazards (see below for more detail). This approach is currently used for existing buildings in the 2007 California Building Code.
- Updated and revised checklists for the Tier 1 screening procedure that was in ASCE 31-03.
- Updated provisions for analysis, foundations, and the major materials chapters in ASCE 41-06 based on incorporation of research and practice since ASCE 41-06 was developed.

A public ballot version of the new standard will be available from ASCE in the spring of 2012 and it is expected that it a prepublication (white cover) version will be available prior to the ICC Final Action Hearings in October of 2012. Any person interested in obtaining a public comment copy of ASCE 41-13 may do so by contacting the proponent at jgoupil@asce.org. **Referencing ASCE 41-13 for Seismic Performance**

It is our opinion that the table describing the ASCE 41 performance levels is best kept within the standard rather than defining force levels, performance objectives, and interpolation of acceptance criteria in the IEBC. This is consistent with how ASCE 7 works with the IBC. Namely, a building is assigned a Risk Category by the IBC, and then ASCE 7 defines the performance objective for that Risk Category. In ASCE 7 this is done via the seismic importance factor and other limitations contained in the standard. We propose the same method for the IEBC: Risk Category is assigned by the Code (in this case the IEBC), and associated seismic performance is specified by the referenced standard (ASCE 41-13).

Section 301.1.4.1 IBC Level Seismic Forces

This proposal removes the ASCE 41-06 performance levels from the IEBC and instead references a new section in ASCE 41-13 that contains criteria for "New Building Standards Equivalent Performance Objective." The objectives are similar to Table 301.1.4.1 in the 2012 IEBC and are intended to be generally consistent with the IBC and ASCE 7 as referenced in IEBC Section 301.1.4.1 Item 1.

Since ASCE 41-13 Section 2.2.4 addresses both structural and nonstructural items, the revised text references only the structural performance criteria consistent with Table 301.1.4.1 in the IEBC.

If kept within the IEBC, an updated version of Table 301.1.4.1 would be as follows:

TABLE 301.1.4.1

PERFORMANCE CRITERIA FOR IBC-LEVEL SEISMIC FORCES

RISK CATEGORY (BASED ON IBC TABLE 1604.5)	PERFORMANCE LEVEL FOR USE WITH ASCE 41 BSE-1 <u>N</u> EARTHQUAKE HAZARD LEVEL	PERFORMANCE LEVEL FOR USE WITH ASCE 41 BSE-2 <u>N</u> EARTHQUAKE HAZARD LEVEL
1	Life Safety (LS)	Collapse Prevention (CP)
II	Life Safety (LS)	Collapse Prevention (CP)

	Damage Control Note a	Limited Safety Note a
IV	Immediate Occupancy (IO)	Life Safety (LS)

a. Acceptance criteria for Risk Category III shall be taken as 80 percent of the acceptance criteria specified for Risk Category II performance, but need not be less than the acceptance criteria specified for Risk Category IV performance levels.

Therefore, this part of the proposal effectively has two substantive revisions to the 2012 version of Table 301.1.4.1 based on the updates in ASCE 41-13:

- BSE-1N and BSE-2N in ASCE 41-13 are similar to the BSE-1 and BSE-2 in ASCE 41-06 except that they are based on the MCE_R ground motions consistent with ASCE 7-10. In addition whereas the BSE-1 in ASCE 41-06 was taken as the lesser of 2/3MCE and earthquake exceedance probability of 10% in 50 years, the BSE-1N is defined as MCE_R without considering the earthquake exceedance probability of 10% in 50 years.
- 2. The interpolation for Risk Category III has been changed from 80% of Risk Category IV to halfway between Risk Category II and Risk Category IV based on the definitions of "Damage Control" and "Limited Safety" in ASCE 41-13. Based on review and modifications to the acceptance criteria during the development of ASCE 41-06, the halfway interpolation better reflects the intent of the ASCE 7-10 Importance Factors for Risk Category III. Note also that the halfway interpolation is consistent with how the IEBC treated Risk Category III prior to 2009.

Section 301.1.4.2 Reduced IBC Level Seismic Forces

This proposal removes the ASCE 41-06 performance levels from the IEBC and instead references the section in ASCE 41-13 that contains criteria for "Basic Performance Objective for Existing Buildings." The objectives are similar to Table 301.1.4.2 in the 2012 IEBC and are intended to be generally consistent with the traditional approach for reduced seismic forces (75% of new code).

Since ASCE 41-13 Section 2.2.1 addresses both structural and nonstructural items, the revised text references only the structural performance criteria consistent with Table 301.1.4.1 in the IEBC.

ASCE 41-13 contains a three-tiered approach with Tiers 1 and 2 taken from ASCE 31-03 and Tier 3 being the Systematic Method from ASCE 41-06. Therefore, effectively the methods in ASCE 41-13 as referenced in new Item 3 and the same as those referenced in 2012 IEBC Items 3 and 4.

If kept within the IEBC, an updated version of Table 301.1.4.1 would be as follows:

TABLE 301.1.4.2

PERFORMANCE CRITERIA FOR REDUCED IBC-LEVEL SEISMIC FORCES

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
-	Life Safety (LS)	Life Safety (LS)
=	Life Safety (LS)	Life Safety (LS)
	Note a	Damage Control
		Note a
IV	Immediate Occupancy (IO)	Immediate Occupancy (IO)

a. For Risk Category III, the ASCE 41 Tier 1 Screening checklists shall be based on the Life Safety Performance Level, except that checklists statements using the Quick Check procedures of ASCE 41 Section 4.5.3 shall be to a demand to capacity ratio based on the average of the demand to applicable capacity ratio for Life Safety and Immediate Occupancy.

a. Acceptance criteria for Risk Category III shall be taken as 80 percent of the acceptance criteria specified for Risk Category II performance, but need not be less than the acceptance criteria specified for Risk Category IV performance levels.

b. For Risk Category III, the ASCE 31 screening phase checklists shall be based on the life safety performance level.

Therefore, this part of the proposal effectively has four substantive revisions:

- The BSE-1E is a newly defined seismic hazard in ASCE 41-13 intended for the Basic Performance Objective for existing buildings. The hazard level is defined as an earthquake with a 20% exceedance probability in 50 years, which is generally consistent with a 10% in 50 year earthquake with the 0.75 factor that was built into the ASCE 31-03 methodology for seismic evaluation.
- 2. The interpolation for Risk Category III has been changed from 80% of Risk Category IV to halfway between Risk Category II and Risk Category IV based on the definitions of "Damage Control" in ASCE 41-13. Based on review and modifications to the acceptance criteria during the development of ASCE 41-06, the halfway interpolation better reflects the intent of the ASCE 7-10 Importance Factors for Risk Category III. Note also that the halfway interpolation is consistent with how the IEBC treated Risk Category III prior to 2009.
- 3. The performance objectives for the Tier 1 and Tier 2 procedures in ASCE 41-13 consists of a single check (one performance level and seismic hazard combination), consistent with ASCE 31-03 as referenced in the 2012 IEBC. Due to seismic hazard reduction (from 2/3 MCE to 20% in 50 year) combined with the elimination of the ASCE 31-03 0.75 factor, the effective performance objective for Tier 1 and Tier 2 is similar to what the 2012 IEBC Table 301.4.2 specifies for ASCE 31-03.
- 4. The performance objective for the Tier 3 procedure in ASCE 41-13 consists of a dual check (two performance level and seismic hazard combination), which differs from how the 2012 IEBC references ASCE 41-06. The inclusion of the second seismic hazard (BSE-2E defined as 5% in 50 year) is intended to offset the effect of the hazard reduction from the ASCE 41-06 BSE-1 (10% in 50 year) to the ASCE 41-13 BSE-1E (20% in 50 year). Therefore, the dual level check proposed is intended to be generally consistent with the single level check in 2012 IEBC Table 301.1.4.2.

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

Analysis: This code change proposal references <u>ASCE</u> standard <u>41</u>, which is already referenced in <u>this code</u>. However, the proposed change to code text is written to correlate with a new edition of the standard <u>ASCE 41-13</u>, rather than the edition presently referenced in the code, which is <u>the 06</u> edition. The <u>13</u> edition of this standard is not yet completed, published and available. The update to this standard will be considered by the Administrative Code Committee during the 2013 Code Development Cycle. Should this code change proposal be approved, but the update to the standard not be approved by the Administrative Code Committee, the code text will revert to the text as it appears in the 2012 Edition of the code. Additionally, if the standard update is approved but the document is not published and available by December 1, 2014, an errata will be issued to the code text to the text as it appears in the 2012 edition of the code.

EB1-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				301.1.4-EB-GOUPIL.doc

EB2–12 [B]301.1.4, [B]301.1.5 (NEW), Chapter 16 (NEW)

Proponent: Matthew Senecal, P.E., American Concrete Institute

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

Revise as follows:

[B] 301.1.4 <u>Seismic</u> evaluation and design procedures. The seismic evaluation and design shall be based on the procedures specified in the *International Building Code*, ASCE 31 or ASCE 41. The procedures contained in Appendix A of this code shall be permitted to be used as specified in Section 301.1.4.2.

[B] 301.1.5 Concrete evaluation and design procedures. Non-seismic evaluation and design of structural concrete shall be in accordance with the requirements of ACI 562.

Add new standard to Chapter 16 as follows:

ACI

562-12 - Code Requirements for Evaluation, Repair, and Rehabilitation of Concrete Buildings

Reason: There are no general evaluation and design criteria for concrete structures in the IEBC. ASCE 31, ASCE 41, and Appendix A of this code provide direction for particular structural systems in high seismic areas. ACI 562 is a new referenced standard addressing non-seismic evaluation and design of concrete structures. ACI 562 is compatible with the principles of this code, ASCE 31, and ASCE 41.

Cost Impact: The code change proposal will set a minimum standard for the repair or rehabilitation of concrete structures; therefore, the cost of construction may increase or decrease depending on the standard of practice of the local jurisdiction.

Analysis: A review of the standard proposed for inclusion in the code ACI 562-12 with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

EB2-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				301.1.4-EB-SENECAL.doc

EB3–12 [B]301.1.4.2, [B]A502.1

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B] 301.1.4.2 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_a used for analysis shall be as specified in Section 301.1.4.1 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 and subject to the limitations of the respective Appendix A Chapters shall be deemed to comply with this section.
 - 2.1. The seismic evaluation and design of unreinforced masonry bearing wall buildings in 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 Risk Category I or II are permitted to be based on the procedures specified in Chapter A2.
 - 2.3. Seismic evaluation and design of cripple walls and sill plate anchorage in residential buildings of light-frame wood construction in 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 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 risk categories are assigned to risk category I, II or III is permitted to be based on the procedures specified in Chapter A5.
- Compliance with ASCE 31 based on the applicable performance level as shown in Table 301.1.4.2. 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 301.1.4.2. The design spectral response acceleration parameters S_{xs} and S_{xr} specified in ASCE 41 shall not be taken less than 75 percent of the respective design spectral response acceleration parameters S_{zs} and S_{zr} defined by the *International Building Code*.

Revise as follows:

[B] A502.1 Scope. The provisions of this chapter shall apply to all buildings having concrete floors or roofs supported by reinforced concrete walls or by concrete frames and columns. This chapter shall not apply to buildings with roof diaphragms that are defined as flexible diaphragms by the building code, and shall not apply to concrete frame buildings with masonry infilled walls. Buildings that were designed and constructed in accordance with the seismic provisions of the 1993 *BOCA National Building Code*, the 1994 *Standard Building Code*, the 1976 *Uniform Building Code*, the 2000 *International Building Code* or later editions of these codes shall be deemed to comply with these provisions, unless the seismicity of the region has increased since the design of the building.

Exception: This chapter shall not apply to concrete buildings where Seismic Design Category A is permitted assigned to risk category IV.

Reason: This proposal clarifies the eligibility of buildings to use Appendix Chapter A5, with coordinated revisions to Chapter 3 and Chapter A5. Two changes are proposed:

- Chapter A5 is intended to improve a building's performance with respect to safety but not necessarily with respect to postearthquake functionality or recovery. As such, it is not appropriate for buildings assigned to risk category IV. The proposal makes appropriate revisions to Chapter 3 and Chapter A5.
- The current Chapter A5 text says the chapter does not "apply" to SDC A; commentary explains that this is based on the low seismicity associated with SDC A. There is no technical reason why the chapter's provisions cannot be used for these buildings, however, so that confusing "limitation" is removed.

EB3-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
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EB4-12 [B] 706.3.2

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B] 706.3.2 Roof diaphragms resisting wind loads in high-wind regions. Where roofing materials are removed from more than 50 percent of the roof diaphragm <u>of a building</u> or section of a building located where the basic wind speed is greater than 90 mph <u>ultimate design wind speed is greater than 155 mph</u> or in a special wind region, as defined in Section 1609 of the *International Building Code*, roof diaphragms, connections of the roof diaphragm to roof framing members, and roof-to-wall connections shall be evaluated for the wind loads specified in the *International*

Building Code, including wind uplift. If the diaphragms and connections in their current condition are not capable of resisting at least 75 percent of those wind loads, they shall be replaced or strengthened in accordance with the loads specified in the *International Building Code*.

Exception: One-and two-family dwellings need not be evaluated or strengthened.

Reason: This proposal corrects a printing error makes the following three changes:

- It makes the wind speed trigger less conservative, raising it from a BWS or nominal value of 90 mph to 120 mph. The current value (BWS = 90) is too low and has the effect of triggering retrofit work in many inland areas unnecessarily and without historical basis. BWS of 120 mph, or UDWS of 155 mph, is thought to be adequate, as it covers the critical coastal areas.
- It converts from the old Basic Wind Speed of 120 mph to the new mapped Ultimate Design Wind Speed of 155 mph, based on IBC Table 1609.3.1. This change is essentially administrative, for purposes of consistent terminology.
- It exempts houses. Many jurisdictions already cover houses with the IRC and exempt them entirely from IBC and IEBC provisions. In these cases the proposed exception makes no difference. Where the IBC or IEBC applies, this exception is considered prudent so as not to discourage very common and beneficial reroofing projects.

Note that by using a single wind speed value, the provision will now automatically cover different areas for buildings in different risk categories (see IBC Figures 1609A through 1609C). This is appropriate.

Finally, addition of the words "of a building" in the first sentence corrects what appears to be a printing error in the first printing of the 2012 IEBC. Those words were present in the 2009 edition and were not removed by any approved changes (though they were missing in the monographs from the last cycle). Ideally, this correction should be made through published errata.

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

EB4-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	2				706.3.2-EB-BONOWITZ.doc

EB5–12 [B] 706.3.2

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB) (gehrlich@nahb.org)

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

Revise as follows:

[B] 706.3.2 Roof diaphragms resisting wind loads in high-wind regions. Where roofing materials are removed from more than 50 percent of the roof diaphragm or section of a building located where the basic ultimate design wind speed V_{ult} , determined in accordance with Figure 1609A of the International Building Code is greater than 90 115 mph or in a special wind region, as defined in Section 1609 of the International Building Code, roof diaphragms, connections of the roof diaphragm to roof framing members, and roof-to-wall connections shall be evaluated for the wind loads specified in the International Building Code, including wind uplift. If the diaphragms and connections in their current condition are not capable of resisting at least 75 percent of those wind loads, they shall be replaced or strengthened in accordance with the loads specified in the International Building Code.

Reason: The purpose of this proposal is to correlate basic wind speed triggers in the IEBC with the IBC. The 2012 IBC adopted new ultimate-strength basis wind speed maps from ASCE 7-10. A conversion factor from the ultimate wind speed selected from the new maps (V_{ult}) down to the old allowable-stress level wind speed (V_{asd}) was introduced into the IBC to accommodate triggers for special requirements in high-wind regions, tables limiting the use of ballasted roofs at certain heights and wind speeds, and tables for proper selection of shingles and other roofing materials for wind resistance. Unfortunately, this conversion was not introduced into the IEBC, with the result that provisions which were supposed to apply only in high-wind regions now appear to apply across the entire United States. This proposal not only corrects this oversight, it fully updates the IEBC provisions to match the 2012 IBC and ASCE 7-10.

EB5-12					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
	-				706.3.2-EB-EHRLICH.doc

EB6–12 [B] 807.5, [IBC] 3404.4

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

THIS IS A TWO PART CODE CHANGE. BOTH PARTS WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE AS TWO SEPARATE CODE CHANGES. SEE TENTATIVE HEARING ORDER FOR THIS COMMITTEE

PART I - IEBC

Revise as follows:

[B] 807.5 Existing structural elements resisting lateral loads. *Alterations* affecting the demands or capacities of existing elements of the lateral load-resisting system shall be evaluated using the wind provisions of the *International Building Code* and the reduced IBC-level seismic forces. Any existing lateral load-resisting structural elements whose demand-capacity ratio with the *alteration* considered is more than 10 percent greater than its demand-capacity ratio with the alteration shall be brought into compliance with those wind and seismic provisions. In addition, the *alteration* shall not create a structural irregularity prohibited by ASCE 7 unless the entire structure complies with Section 301.1.4.2. For the purposes of this section, comparisons of demand-capacity ratios and calculation of design lateral loads, forces and capacity shall account for the cumulative effects of *additions* and *alterations* since the original construction. Except as permitted by Section 807.6, where the alteration increases design lateral loads, or where the alteration results in prohibited structural irregularity as defined in ASCE 7, or where the alteration decreases the capacity of any existing lateral load-carrying structural element, the structure of the altered building or structure shall be shown to meet the wind and seismic provisions of the *International Building Code*. Reduced IBC-level seismic forces shall be permitted.

Exception: Any existing lateral load-carrying structural element whose demand-capacity ratio with the alteration considered is no more than 10 percent greater than its demand-capacity ratio with the alteration ignored shall be permitted to remain unaltered. For purposes of calculating demand-capacity ratios, the demand shall consider applicable load combinations with design lateral loads or forces per IBC Sections 1609 and 1613. Reduced IBC-level seismic forces shall be permitted. For purposes of this exception, comparisons of demand-capacity ratios and calculation of design lateral loads, forces, and capacities shall account for the cumulative effects of additions and alterations since original construction.

PART II – IBC STRUCTURAL

Revise as follows:

3404.4 Existing structural elements carrying lateral load. Except as permitted by Section 3404.5, where the *alteration* increases design lateral loads in accordance with Section 1609 or 1613, or where the *alteration* results in a <u>prohibited</u> structural irregularity as defined in ASCE 7, or where the *alteration* decreases the capacity of any existing lateral load-carrying structural element, the structure of the altered building or structure shall be shown to meet the requirements of Sections 1609 and 1613.

Exception: Any existing lateral load-carrying structural element whose demand-capacity ratio with the *alteration* considered is no more than 10 percent greater than its demand-capacity ratio with the *alteration* ignored shall be permitted to remain unaltered. For purposes of calculating demand-capacity ratios, the demand shall consider applicable load combinations with design lateral loads or forces per Sections 1609 and 1613. For purposes of this exception, comparisons of demand-capacity ratios and calculation of design lateral loads, forces, and capacities shall account for the cumulative effects of *additions* and *alterations* since original construction.

Reason: The proposal rewrites IEBC Section 807.5 using the clearer logic of IBC Section 3404.4. No change in scope or effect is intended. In applying the clearer wording, however, the scope of triggered work associated with the creation of a prohibited irregularity is slightly changed, from full compliance without exception to the usual compliance eligible for the 10 percent DCR exception. This is appropriate, and the resulting IEBC provision will be consistent with the corresponding IBC provision, except that the IEBC criteria will continue to allow the use of reduced seismic forces.

The proposal also modifies IBC Section 3404.4 for consistency by inserting the word "prohibited" in one place.

EB6-12 PART I - IEBC					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
PART II - IBC					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					807.5-EB-BONOWITZ.doc

EB7-12 [B] 907.4.2

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] 907.4.2 Substantial structural alteration. Where more than 30 percent of the total floor and roof areas of the building or structure have been or are proposed to be involved in structural *alteration* within a five-year period, the evaluation and analysis shall demonstrate that the <u>lateral load resisting system of the</u> altered building or structure complies with the *International Building Code* for wind loading and with reduced IBC-level seismic forces. The areas to be counted toward the 30 percent shall be those areas tributary to the vertical load-carrying components, such as joists, beams, columns, walls and other structural components that have been or will be removed, added or altered, as well as areas such as mezzanines, penthouses, roof structures and in-filled courts and shafts.

Reason: This proposal clarifies the long-standing intent of the IEBC that alteration-triggered structural upgrade applies to the (designated or *de facto*) lateral system only, and not to the gravity system or to nonstructural components.

EB7-12					
Public Hearing:	Committee:	AS	AM	D	
•	Assembly:	ASF	AMF	DF	
	,				907.4.2-EB-BONOWITZ.doc

EB8–12 [B] 907.4.2, [B] 907.4.3 (NEW), [B] 907.4.4

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] 907.4.2 Substantial structural alteration. Where more than 30 percent of the total floor and roof areas of the building or structure have been or are proposed to be involved in structural *alteration* within a five-year period, the evaluation and analysis shall demonstrate that the <u>lateral load-resisting system of the</u> altered building or structure complies with the *International Building Code* for wind loading and with reduced IBC-level seismic forces. The areas to be counted toward the 30 percent shall be those areas tributary to the vertical load-carrying components, such as joists, beams, columns, walls and other structural components that have been or will be removed, added or altered, as well as areas such as mezzanines, penthouses, roof structures and in-filled courts and shafts.

[B]907.4.3 Seismic Design Category F. Where the building is assigned to seismic design category F. the evaluation and analysis shall demonstrate that the lateral load-resisting system of the altered building or structure complies with reduced IBC-level seismic forces and with the wind provisions applicable to a limited structural alteration.

[B] 907.4.3 <u>907.4.4</u> Limited structural alteration. Where the work does not involve a substantial structural *alteration* and the building is not assigned to seismic design category F, the existing elements of the lateral load-resisting system shall comply with Section 807.5.

Reason: This proposal adds a new category of triggered seismic upgrade for the most vulnerable buildings undergoing Level 3 Alteration. Currently, alteration triggers seismic upgrade only when the alteration project makes intentional structural changes that add up to a "substantial structural alteration" (Section 907.4.2). A top-to-bottom architectural and mechanical renovation, however, triggers no seismic mitigation. This proposal fills some of that mitigation gap.

The proposal covers only buildings assigned to Seismic Design Category F. SDC F buildings are those in the highest seismicity and of the greatest importance to post-earthquake response and recovery (risk category IV). If any buildings are deserving of triggered upgrades when their lives are significantly extended through major alterations, these are. Many such buildings (California hospitals, for example) are already addressed by targeted legislation, so will not be affected by the proposed trigger. Yet many jurisdictions with substantial seismic risks do not have histories of proactive mitigation and lack the code mechanism to enforce these common-sense improvements to essential facilities. These jurisdictions look to the model codes for best practices.

The proposal borrows language and concepts, specifically the use of reduced loads, from the current trigger in Section 907.4.2. By limiting the scope and criteria, the proposal properly balances regulatory benefits with potential owner costs. (See also the Cost Impact statement below for mitigating factors.)

The proposal makes two associated revisions in addition to adding new Section 907.4.3:

- In Section 907.4.2, the long-standing intent that triggered upgrades address only structural systems and do not require nonstructural compliance is clarified by adding a few words.
- In current Section 907.4.3 (to be renumbered 907.4.4), reference to the proposed SDC F trigger is added to maintain the logical flow.

Cost Impact: Undetermined: Buildings assigned to SDC F that undergo Level 3 Alteration will be subject to seismic upgrade. However, 1) it is not known how many such buildings exist, 2) many such buildings already have made or would make seismic improvements voluntarily, especially as part of a major alteration, 3) many such buildings would pass the triggered evaluation anyway and would not entail any additional cost, and 4) owners can avoid the triggered work by limiting their scope of alteration.

EB8-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				907.4.3-EB-BONOWITZ.doc

EB9-12 [B] 907.4.4

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] 907.4.4 Wall anchors for concrete and masonry buildings. For any building assigned to Seismic Design Category D, E or F with a structural system consisting of concrete or reinforced masonry walls with a flexible roof diaphragm or <u>and any building assigned to Seismic Design Category C, D, E, or F with a structural system consisting of</u> unreinforced masonry walls with any type of roof diaphragm, the alteration work shall include installation of wall anchors at the roof line to resist the reduced IBC-level seismic forces, unless an evaluation demonstrates compliance of existing wall anchorage.

Reason: This proposal extends a common-sense seismic mitigation provision from SDC D-F into SDC C.

The proposal is motivated by damage patterns observed throughout the east coast from the 2011 Virginia earthquake and by the recognition that most jurisdictions where SDC C is prevalent do not have histories of proactive mitigation. Rather, they look to the model codes for best practices. This proposal is modeled on successful practice in Massachusetts, an SDC C jurisdiction that has been proactive regarding mitigation and adaptive reuse of unreinforced masonry buildings.

The proposal does represent an increase in potentially triggered work, but the increase is measured and prudent. The proposal only applies to URM bearing walls. A lack of roof-to-wall anchors, especially when paired with unbraced URM parapets, poses a remaining risk throughout areas of moderate and high seismicity. Also the proposal is only triggered by Level 3 Alterations where the intended work area already exceeds 50 percent of the building. The triggered wall anchorage represents a small additional cost by comparison, and one that makes sense where significant resources are being spent to modernize a URM building.

Cost Impact: URM buildings assigned to SDC C that undergo Level 3 Alteration will require wall anchors. The cost is considered small compared with the typical cost of a Level 3 Alteration.

EB9-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				907.4.4-EB-BONOWITZ.doc

EB10–12 [B] 907.4.5

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] 907.4.5 Bracing for unreinforced masonry parapets. Parapets constructed of unreinforced masonry in buildings assigned to Seismic Design Category <u>C</u>, D, E or F shall have bracing installed as needed to resist the reduced IBC-level seismic forces, unless an evaluation demonstrates compliance of such items.

Reason: This proposal extends a common-sense seismic mitigation provision from SDC D-F into SDC C.

The proposal is motivated by damage patterns observed throughout the east coast from the 2011 Virginia earthquake and by the recognition that most jurisdictions where SDC C is prevalent do not have histories of proactive mitigation. Rather, they look to the model codes for best practices. This proposal is modeled on successful practice in Massachusetts, an SDC C jurisdiction that has been proactive regarding mitigation and adaptive reuse of unreinforced masonry buildings.

- The proposal does represent an increase in potentially triggered work, but the increase is measured, prudent, and cost-effective:
 The proposal only applies to URM parapets. Unbraced URM parapets remain the most widespread, vulnerable, and dangerous structural elements in earthquakes, as we have seen in several recent non-California events, including Virginia, Wells, NV, and Christchurch, NZ.
 - Parapet bracing has a long history and is effective. Los Angeles required URM parapet bracing in 1949.
 - Parapet bracing is not intrusive, as it can be done from outside the building.
 - The proposal is only triggered by Level 3 Alterations where the intended work area already exceeds 50 percent of the building. The triggered parapet bracing represents a small additional cost by comparison, and one that makes sense where significant resources are being spent to modernize a URM building.

Cost Impact: Minor: URM buildings assigned to SDC C that undergo Level 3 Alteration will become subject to parapet bracing. The cost of parapet bracing is small compared with the typical cost of a Level 3 Alteration.

EB10-12				
Public Hearing: Committee:	AS	AM AME	D	
Assembly.	AU		Ы	907.4.5-EB-BONOWITZ.doc

EB11–12 [B] 1007.3.1

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] 1007.3.1 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 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 1012.4; 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 301.1.4.1 for the new risk category.

Exceptions:

- Group M Any occupancies being changed to Group A, E, I-1, M, R-1, R-2 or R-4 occupancies without an increase in risk category, for buildings less than six stories in height and in assigned to 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 risk category based on the provision for reduced *International Building Code* level seismic forces as specified in Section 301.1.4.2.
- 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 Risk Category IV. For the purposes of this exception, buildings occupied by two or more occupancies not included in the same 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 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)].

Reason: This proposal extends the seismic upgrade waiver currently provided in Exception 1.

Currently, Section 1007.3.1 triggers seismic upgrade for certain changes of occupancy from one "hazard category" to another, defined by Table 1012.4. It makes special provisions, both in the triggers and the exceptions, for Group M buildings. In particular, Exception 1 waives the upgrade requirement for certain changes from Group M within hazard category 3, presumably based on the relative seismic risk of the different HC 3 occupancies. But the hazard categories are defined in terms of egress, and there really is no rational basis in seismic terms for singling out Mercantile occupancies. Any seismic risk posed (or avoided) by a Group M building is certainly also posed (or avoided) by many Group B, F, S, U, or R-3 buildings, but the latter group are all assigned to HC 4 and are therefore targeted for seismic upgrades in ways that Group M buildings are not. This does not make sense, and it has the effect of discouraging beneficial adaptive reuse projects for existing Group B and F buildings.

The proposal therefore extends the Exception 1 waiver to other occupancies regardless of their hazard category. The provisos regarding building height and SDC remain, so only relatively low risk buildings are getting a new waiver. Also, if the Risk Category changes, the waiver does not apply.

Note that even under this proposal, Section 1007.3.1 will remain more conservative with respect to seismic upgrade triggers than IBC Section 3408, which triggers seismic upgrade only for a change in risk category, regardless of occupancy group.

EB11-12					
Public Hearing: Com	mittee: AS	A	И [)	
Asse	mbly: ASI	F AN	MF D	DF	
	,			1	007.3.1-EB-BONOWITZ.doc

EB12–12 [B]1103.3

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] 1103.3 Lateral force-resisting system. The lateral force-resisting system of *existing buildings* to which additions are made shall comply with Sections 1103.3.1, 1103.3.2 and 1103.3.3.

Exceptions:

- 1. Buildings of Group R occupancy with no more than five dwelling or sleeping units used solely for residential purposes where the *existing building* and the *addition* comply with the conventional light-frame construction methods of the *International Building Code* or the provisions of the *International Residential Code*.
- 2. In other existing buildings where the lateral-force story shear in any story is not increased by more than 10 percent cumulative.

2. Any existing lateral load-carrying structural element whose demand-capacity ratio with the *addition* considered is no more than 10 percent greater than its demand-capacity ratio with the *addition* ignored shall be permitted to remain unaltered. For purposes of this exception, comparisons of demand-capacity ratios and calculation of design lateral loads, forces, and capacities shall account for the cumulative effects of additions and alterations since original construction.

Reason: The proposal follows the precedent set in the 2006 IBC, making the exception to lateral system upgrade element-based, as opposed to story-based. The intent is that elements triggered for lateral upgrade by Section 1103.3.1 or 1103.3.2 should be exempt based on their individual demand-capacity ratios, not on the overall story shear. A focus on story shear can miss critical individual elements in vertical additions and can be difficult to define in the case of horizontal additions. The language of the proposed exception is taken from IBC Section 3403.4.

EB12-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	2				1103.3-EB-BONOWITZ.doc

EB13–12 [B] 1103.5

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net).

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

Revise as follows:

[B] 1103.5 Flood hazard areas. Additions and foundations in *flood hazard areas* shall comply with the following requirements:

- 1. For horizontal additions that are structurally interconnected to the existing building:
 - 1.1. If the *addition* and all other proposed work, when combined, constitute *substantial improvement*, the *existing building* and the *addition* shall comply with Section 1612 of the *International Building Code*.
 - 1.2. If the *addition* constitutes *substantial improvement*, the *existing building* and the *addition* shall comply with Section 1612 of the *International Building Code*.
- 2. For horizontal additions that are not structurally interconnected to the existing building:
 - 2.1. The addition shall comply with Section 1612 of the International Building Code.
 - 2.2. If the *addition* and all other proposed work, when combined, constitute *substantial improvement*, the *existing building* and the *addition* shall comply with Section 1612 of the *International Building Code*.
- 3. For vertical additions and all other proposed work that, when combined, constitute *substantial improvement*, the *existing building* shall comply with Section 1612 of the *International Building Code*.
- 4. For a new, replacement, raised, or extended foundation, if the foundation work and all other proposed work, when combined, constitute *substantial improvement*, the *existing building* shall comply with Section 1612 of the *International Building Code*.
- 5. For a new foundation or replacement foundation, the foundation shall comply with Section 1612 the International Building Code.

Reason: New foundations and replacement foundations are new structures and should comply with the code requirements for new structures rather than be treated the same as raised/extended foundations. The situation with a new or replacement foundation is similar to relocated or moved buildings which are covered by Chapter 13. Section 1302.6 requires the foundations for moved or relocated buildings to comply with the requirements for new structures.

Cost Impact: This provision applies to projects that already propose to build a new foundation or a replacement foundation. Because new and replacement foundations should already be considered new structures, there shouldn't be any increase in cost. However, given how the existing language is written, there will be a cost increase only for those foundations that would not have been determined to be substantial improvement.

EB13-12			
Public Hearing: Committee:	AS	AM	D
Assembly:	ASF	AMF	DF
			1103.5-EB-INGARGIOLA-WILSON-QUINN.doc

EB14–12 [B]1302.6

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net).

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

Revise as follows:

[B]1302.6 Flood hazard areas. If relocated or moved into a flood hazard area, structures shall comply with Section 1612 of the *International Building Code* or Section R322 of the *International Residential Code*, as applicable.

Reason: Section 1302.2 already specifies that the foundation system of relocated buildings shall comply with the IBC or IRC, as applicable. As currently written, Section 1302.6 does not allow use of the flood resistant requirements of the IRC. This proposal clarifies that the provisions of the International Residential Code may be used, if applicable to the occupancy.

Cost Impact: The cost for some residential foundations may be lower because the prescriptive provisions of the IRC can be used, rather than requiring a registered design professional for all foundation system for relocated homes.

EB14-12				
Public Hearing: C	Committee:	AS	AM	D
A	Assembly:	ASF	AMF	DF
	2			1302.6-EB-INGARGIOLA-WILSON-QUINN.doc

EB15–12 [B]A103

Proponent: Marko Schotanus, Chair, Existing Buildings Committee, Structural Engineers Association of California (mschotanus@ruthchek.com)

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

Revise as follows:

SECTION A103 DEFINITIONS

For the purpose of this chapter, the applicable definitions in the building code shall also apply.

[B] POINTING. The partial reconstruction of the bed joints of an unreinforced masonry wall as defined in UBC Standard 21-8.

Reason: Pointing is not limited to bed joints. The chapter provisions also intend that deterioration in head joints should be considered.

EB15-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	A103-POINTING-EB-SCHOTANUS.doc

EB16–12 [B] A105.3, [B] A107.3, [B] A107.4, [B] Table A1-E, [B]A107.5 (NEW), [B]A107.5.1 (NEW), [B]A107.5.2 (NEW), [B]A107.5.3 (NEW), [B]A107.5.4, [B]Chapter A6 (New)

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] A105.3 Requirements for plans. The following construction information shall be included in the plans required by this chapter:

- 1. Dimensioned floor and roof plans showing existing walls and the size and spacing of floor and roof-framing members and sheathing materials. The plans shall indicate all existing and new crosswalls and shear walls and their materials of construction. The location of these walls and their openings shall be fully dimensioned and drawn to scale on the plans.
- 2. Dimensioned wall elevations showing openings, piers, wall classes as defined in Section A106.3.3.8, thickness, heights, wall shear test locations, cracks or damaged portions requiring repairs, the general condition of the mortar joints, and if and where pointing is required. Where the exterior face is veneer, the type of veneer, its thickness and its bonding and/or ties to the structural wall masonry shall also be noted.
- 3. The type of interior wall and ceiling materials, and framing.
- 4. The extent and type of existing wall anchorage to floors and roof when used in the design.
- 5. The extent and type of parapet corrections that were previously performed, if any.
- 6. *Repair* details, if any, of cracked or damaged unreinforced masonry walls required to resist forces specified in this chapter.
- 7. All other plans, sections and details necessary to delineate required retrofit construction.
- 8. The design procedure used shall be stated on both the plans and the permit application.
- Details of the anchor prequalification program required by UBC Standard 21-7 Section A107.5.3, I f used, including location and results of all tests.

[B] A107.3 Existing wall anchors. Existing wall anchors used as all or part of the required tension anchors shall be tested in pullout according to UBC Standard 21-7 Section A107.5.1. The minimum number of anchors tested shall be four per floor, with two tests at walls with joists framing into the wall and two tests at walls with joists parallel to the wall, but not less than 10 percent of the total number of existing tension anchors at each level.

[B] A107.4 New bolts. All new embedded bolts shall be subject to periodic special inspection in accordance with the building code, prior to placement of the bolt and grout or adhesive in the drilled hole. Five percent of all bolts that do not extend through the wall shall be subject to a direct-tension test, and an additional 20 percent shall be tested using a calibrated torque wrench. Testing shall be performed in accordance with UBC Standard 21-7 Section A107.5. New bolts that extend through the wall with steel plates on the far side of the wall need not be tested.

Exception: Special inspection in accordance with the building code may be provided during installation of new anchors in lieu of testing.

All new embedded bolts resisting tension forces or a combination of tension and shear forces shall be subject to periodic special inspection in accordance with the building code, prior to placement of the bolt and grout or adhesive in the drilled hole. Five percent of all bolts resisting tension forces shall be subject to a direct-tension test, and an additional 20 percent shall be tested using a calibrated torque wrench.

Testing shall be performed in accordance with UBC Standard 21-7 Section A107.5. New through-bolts need not be tested.

[B] TABLE A1-E STRENGTH VALUES OF NEW MATERIALS USED IN CONJUNCTION WITH EXISTING CONSTRUCTION

e. Other bolt sizes, values and installation methods may be used, provided a testing program is conducted in accordance with UBC Standard 21-7 Section A107.5.3. The useable strength value shall be determined by multiplying the calculated allowable value, as determined by UBC Standard 21-7 in accordance with Section A107.5.3, by 3.0, and the useable usable value shall be limited to a maximum of 1.5 times the value given in the table. Bolt spacing shall not exceed 6 feet (1829 mm) on center and shall not be less than 12 inches (305 mm) on center.

(Portions of Table not shown remain unchanged)

[B]A107.5 Tests of anchors in unreinforced masonry walls.

[B]A107.5.1 Direct tension testing of existing anchors and new bolts. The test apparatus shall be supported by the masonry wall. The distance between the anchor and the test apparatus support shall not be less than one half the wall thickness for existing anchors and 75 percent of the embedment for new embedded bolts. Existing wall anchors shall be given a preload of 300 pounds (1335 N) prior to establishing a datum for recording elongation. The tension test load reported shall be recorded at 1/8 inch (3.2 mm) relative movement between the existing anchor and the adjacent masonry surface. New embedded tension bolts shall be subject to a direct tension load of not less than 2.5 times the design load but not less than 1,500 pounds (6672 N) for five minutes (10 percent deviation).

[B]A107.5.2 Torque testing of new bolts. Bolts embedded in unreinforced masonry walls shall be tested using a torque-calibrated wrench to the following minimum torques:

1/2-inch-diamter (13 mm) bolts: 40 foot pounds (54.2 N-m) 5/8-inch-diamter (16 mm) bolts: 50 foot pounds (67.8 N-m) 3/4-inch-diamter (19 mm) bolts: 60 foot pounds (81.3 N-m)

[B]A107.5.3 Prequalification test for bolts and other types of anchors. This section is applicable when it is desired to use tension or shear values for anchors greater than those permitted by Table A1-E. The direct-tension test procedure set forth in Section A107.5.1 for existing anchors shall be used to determine the allowable tension values for new embedded through bolts, except that no preload is required. Bolts shall be installed in the same manner and using the same materials as will be used in the actual construction. A minimum of five tests for each bolt size and type shall be performed for each class of masonry in which they are proposed to be used. The allowable tension values for such anchors shall be the lesser of the average ultimate load divided by a factor of safety of 5.0 or the average load at which 1/8 inch (3.2 mm) elongation occurs for each size and type of bolt and class of masonry.

The test procedure for prequalification of shear bolts shall comply with ASTM E 488 or another approved procedure.

The allowable values determined in this manner shall be permitted to exceed those set forth in Table A1-E.

[B]A107.5.4 Reports. Results of all tests shall be reported. The report shall include the test results as related to anchor size and type, orientation of loading, details of the anchor installation and embedment, wall thickness, and joist orientation.

Add new standard to Chapter A6 as follows:

ASTM

E 488-10 Test Method for Strength of Anchors in Concrete and Masonry Elements

Reason: This proposal solves a problem caused by reference in the current provisions to an unavailable standard. Several sections and tables in Chapter A1 reference UBC Standard 21-7, but UBC Standards are no longer maintained and are not readily available. We know of no ICC-compliant standard for testing of existing and new wall anchors as needed by Appendix A1. Therefore, this proposal inserts the provisions from 1997 UBC Standard 21-7 in their entirety (with minor editorial changes) into a new Section A107.5.

The proposal also adds ASTM E 488 to IEBC Chapter A6. The 1990 edition of this standard was referenced in 1997 UBC Standard 21-7. This proposal updates that to the 2010 edition, as cited in proposed Section A107.5.3. A copy of the 2003 is being submitted separately for reference; the 2010 version is little-changed, and a copy will be provided prior to the hearings.

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

Analysis: A review of the standard proposed for inclusion in the code, ASTM E488-10 with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

EB16-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	A105.3-EB-BONOWITZ.doc

EB17–12 [B] A106.2

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] A106.2 Existing materials. Existing materials used as part of the required vertical load-carrying or lateral force-resisting system shall be in sound condition, or shall be repaired or removed and replaced with new materials. All other unreinforced masonry materials shall comply with the following requirements:

- 1. The lay-up of the masonry units shall comply with Section A106.3.2, and the quality of bond between the units has been verified to the satisfaction of the building official;
- 2. Concrete masonry units are verified to be load-bearing units complying with UBC Standard 21-4 ASTM C90 or such other standard as is acceptable to the building official; and
- 3. The compressive strength of plain concrete walls shall be determined based on cores taken from each class of concrete wall. The location and number of tests shall be the same as those prescribed for tensile-splitting strength tests in Sections A106.3.3.3 and A106.3.3.4, or in Section A108.1.

The use of materials not specified herein or in Section A108.1 shall be based on substantiating research data or engineering judgment, with the approval of the building official.

Reason: This proposal solves a problem caused by reference in the current provisions to an unavailable standard. Current Section A106.2 references UBC Standard 21-4, but UBC Standards are no longer maintained and are not readily available. 1997 UBC Standard 21-4 was already based on ASTM Standard Specification C90-95 with respect to hollow load-bearing concrete block. The latest version of C90 provides the data needed to determine what Appendix A1 requires: the net mortared area of hollow concrete block and the thickness of face shells of nominal widths. The proposal therefore references ASTM C90 in place of UBC Standard 21-4.

ASTM C90 is not a new IEBC reference standard, as it is already referenced in IEBC Section A505.2.3.

EB17-12					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
					A106.2-EB-BONOWITZ.doc

EB18–12 [B] A106.3.2.1

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] A106.3.2.1 Multiwythe solid brick. The facing and backing shall be bonded so that not less than 10 percent of the exposed face area is composed of solid headers extending not less than 4 inches (102 mm) into the backing. The clear distance between adjacent full length headers shall not exceed 24 inches (610 mm) vertically or horizontally. Where the backing consists of two or more wythes, the headers shall extend not less than 4 inches (102 mm) into the most distant wythe, or the backing wythes shall be bonded together with separate headers with their area and spacing conforming to the foregoing. Wythes of walls not bonded as described above shall be considered veneer. Veneer wythes shall not be included in the effective thickness used in calculating the height-to-thickness ratio and the shear capacity of the wall.

Exception: <u>Where S_{D1} is not more than 0.3</u>, veneer wythes anchored as specified in the building code and made composite with backup masonry may be used for calculation of the effective thickness, where *SD1* exceeds 0.3.

Reason: This proposal corrects a mistake made when references to Seismic Zones were removed in the 2006 I-codes. In the 2003 IEBC, this exception read, "In other than Seismic Zone 4, or where S_{D1} exceeds 0.3g, veneer wythes anchored as specified in the Building Code and made composite with backup masonry may be used for calculation of the effective thickness." The revision for 2006 intended to delete the reference Seismic Zone 4, but by striking only "In other than Seismic Zone 4," it changed the meaning to suggest that veneer may be counted as part of the masonry *only* in regions of high seismicity, when just the opposite is intended. This proposal corrects the provision and restores the intended meaning.

E	B18	-12		
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Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				A106.3.2.1-EB-BONOWITZ.doc

EB19–12 [B]A104, [B]A106.3.3.1

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B]SECTION A104 SYMBOLS AND NOTATIONS

For the purpose of this chapter, the following notations supplement the applicable symbols and notations in the building code.

V_{test} = Load at incipient cracking for each in-place shear test per UBC Standard 21-6 performed in accordance with Section A106.3.3.1, pounds (kN).

(No change to notations not shown)

[B] A106.3.3.1 Mortar tests. The quality of mortar in all masonry walls shall be determined by performing in-place shear tests in accordance with the following:

- 1. The bed joints of the outer wythe of the masonry should shall be tested in shear by laterally displacing a single brick relative to the adjacent bricks in the same wythe. The head joint opposite the loaded end of the test brick should shall be carefully excavated and cleared. The brick adjacent to the loaded end of the test brick should shall be carefully removed by sawing or drilling and excavating to provide space for a hydraulic ram and steel loading blocks. Steel blocks, the size of the end of the brick, should shall be used on each end of the ram to distribute the load to the brick. The blocks should shall not contact the mortar joints. The load should shall be applied horizontally, in the plane of the wythe. The load recorded at first movement of the test brick as indicated by spalling of the face of the mortar bed joints is V_{test} in Equation A1-3.
- 2. Alternative procedures for testing shall be used where in-place testing is not practical because of crushing or other failure mode of the masonry unit (see Section A106.3.3.2).

Reason: This proposal is effectively editorial. It removes duplication and solves the problem caused by reference to an unavailable standard. UBC Standard 21-6 is no longer maintained and is not readily available. In any case, the information contained in UBC Standard 21-6 (a two-paragraph long standard) already appears verbatim in Section A106.3.3.1 item 1. The only differences are:

- Current A106.3.3.1 item 1 uses "should" in several places. The proposal changes these to "shall."
- UBC Standard 21-6 describes briefly how to calculate the mortar strength from the test. The last sentence of current Section A106.3.3.1 item 1 already replaces that instruction with a more specific reference to Equation A1-3.

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

EB19-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	

A106.3.3.1-EB-BONOWITZ

EB20–12 [B]A103, [B]A106.3.3.9

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B] A106.3.3.9 Pointing. Deteriorated mortar joints in unreinforced masonry walls shall be pointed according to UBC Standard 21-8. in accordance with the following requirements:

- Joint preparation. The deteriorated mortar shall be cut out by means of a toothing chisel or nonimpact power tool to a depth at which sound mortar is reached but not less than 3/4-inch (19 mm). Care shall be taken not to damage the brick edges. After cutting is complete, all loose material shall be removed with a brush, air stream, or water stream.
- 2. Mortar preparation. The mortar mix shall be proportioned as required by the registered design professional. The pointing mortar shall be prehydrated by first thoroughly mixing all ingredients dry and then mixing again, adding only enough water to produce a damp workable mix which will retain its form when pressed into a ball. The mortar shall be kept in a damp condition for one and one-half hours; then sufficient water shall be added to bring it to a consistency that is somewhat drier than conventional masonry mortar.
- 3. **Packing.** The joint into which the mortar is to be packed shall be damp but without freestanding water. The mortar shall be tightly packed into the joint in layers not exceeding 1/4-inch (6.4 mm) in depth until it is filled; then it shall be tooled to a smooth surface to match the original profile.

Nothing shall prevent pointing of any deteriorated masonry wall joints before the tests are made testing in accordance with Section A106.3.3 is performed, except as required in Section A107.1.

Revise as follows:

SECTION A103 DEFINITIONS

POINTING. The partial reconstruction of the bed joints of an unreinforced masonry wall as defined in UBC Standard 21-8. The process of removal of deteriorated mortar from between masonry units and placement of new mortar. Also known as repointing or tuckpointing for purposes of this chapter.

REPOINTING. See Pointing.

TUCKPOINTING. See Pointing.

Reason: This proposal solves a problem caused by reference in the current provisions to an unavailable standard. Current Section A106.3.3.9 references UBC Standard 21-8, but UBC Standards are no longer maintained and are not readily available. However, while various references exist, we know of no ICC-compliant standard for pointing. Therefore, this proposal inserts the relevant and necessary wording from UBC 21-8 (a short document less than a half-page long) into the provisions.

Specifically, the proposal:

• Clarifies that "pointing," the term used in this chapter, also means "repointing" or "Tuckpointing," terms used in some locales to mean the same thing. (For examples, see ASTM E2260-03, "Standard Guide for Repointing (Tuckpointing) Historic Masonry;" National Park Service Preservation Brief 2, "Repointing Mortar Joints in Historic Masonry Buildings;" and Brick

Industry Association Technical Note 46, "Maintenance of Brick Masonry."). Note that despite the current text of section A103, UBC Standard 21-8 did not actually define pointing, so this definition is new, but consistent with that old standard.

- Adds the terms Repointing and Tuckpointing to the Definitions as a guide for those using other terms.
- Adds provisions describing the pointing process, using language taken directly from 1997 UBC Standard 21-8, with a few minor editorial changes. The only substantive change is the removal of a requirement in UBC Standard 21-8 for Type N or Type S pointing mortar. Selection of the mortar can be left to the registered design professional.
- Makes a more specific reference to the tests of interest with respect to pointing.

EB20-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					A106.3.3.9-EB-BONOWITZ

EB21-12 [B]A108.2

Proponent: Gary R. Searer, Wiss, Janney, Elstner Associates, Inc., representing self

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

Revise as follows:

[B] A108.2 Masonry shear strength. The unreinforced masonry shear strength, v_m , shall be determined for each masonry class from one of the following equations:

1. The unreinforced masonry shear strength, v_m , shall be determined by Equation A1-4 when the mortar shear strength has been determined by Section A106.3.3.1.

 $v_m = 0.56v_t + \frac{0.75P_D}{A}$

The mortar shear strength values, v_t , shall be determined in accordance with Section A106.3.3.5 and shall not exceed 100 pounds per square inch (689.5 kPa) for the determination of v_m .

(Portions of text not shown remain unchanged)

Reason: There is no technical justification for limiting mortar shear strength values to an arbitrary value of 100 psi. While many structures have mortar strengths less than 100 psi, many other structures have mortar strengths greater than 100 psi. There is no need for extra conservatism for stronger, better built, or more robust structures.

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

EB21-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	•				A108.2-EB-SEARER.doc

(Equation A1-4)

(Equation

EB22-12 [B]A206.6

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Delete without substitution:

[B]A206.6 Minimum member size. Wood members used to develop anchorage forces to the diaphragm must be at least 3-inch (76 mm) nominal members for new construction and replacement. All such members must be checked for gravity and earthquake loading as part of the wall-anchorage system.

Exception: Existing 2-inch (51 mm) nominal members may be doubled and internailed to meet the strength requirement.

Reason: Minimum member size is no longer a requirement of the code for new construction. It is more rational to determine member size by calculation than by arbitrary limits, so smaller members should be acceptable if justified by calculation.

EB22-12				
Public Hearing: Committee	: AS	AM	D	
Assembly:	ASF	AMF	DF	
				A206.6-EB-BONOWITZ

EB23–12 [B] A301.3

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] A301.3 Alternative design procedures. The details and prescriptive provisions herein are not intended to be the only acceptable strengthening methods permitted. Alternative details and methods may be used where designed by a registered design professional and <u>or</u> approved by the *code official*. Approval of alternatives shall be based on a demonstration that the method or material used is at least equivalent in terms of strength, deflection and capacity to that provided by the prescriptive methods and materials.

Where analysis by a registered design professional is required, such analysis shall be in accordance with all requirements of the building code, except that the seismic forces may be taken as 75 percent of those specified in the building code.

Reason: This proposal provides flexibility to local jurisdictions to use alternative prescriptive solutions without the need for engineered solutions. This is consistent with the intent of the chapter and represents a practice already successfully in place in Berkeley and other California jurisdictions. Since the final sentence of the section already requires a demonstration of equivalence, code official approval is sufficient and there should be no need for both special approval *and* engineered design.

EB23-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	2				A301.3-EB-BONOWITZ.doc

EB24–12 [B] A302

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

SECTION A302 DEFINITIONS

For the purpose of this chapter, in addition to the applicable definitions in the building code, certain additional terms are defined as follows:

[B] ADHESIVE ANCHOR. An assembly consisting of a threaded rod, washer, nut, and chemical adhesive approved by the *code official* for installation in existing concrete or masonry.

[B] COMPOSITE PANEL. A wood structural panel product composed of a combination of wood veneer and wood-based material, and bonded with waterproof adhesive.

[B] CRIPPLE WALL. A wood-frame stud wall extending from the top of the foundation to the underside of the lowest floor framing.

[B] EXPANSION ANCHOR. An approved post-installed anchor, inserted into a pre-drilled hole in existing concrete or masonry, that transfers loads to or from the concrete or masonry by direct bearing or friction or both.

[B] ORIENTED STRAND BOARD (OSB). A mat-formed wood structural panel product composed of thin rectangular wood strands or wafers arranged in oriented layers and bonded with waterproof adhesive. **[B] PERIMETER FOUNDATION.** A foundation system that is located under the exterior walls of a building.

[B] PLYWOOD. A wood structural panel product composed of sheets of wood veneer bonded together with the grain of adjacent layers oriented at right angles to one another.

[B] SNUG-TIGHT. As tight as an individual can torque a nut on a bolt by hand, using a wrench with a 10-inch-long (254 mm) handle, and the point at which the full surface of the plate washer is contacting the wood member and slightly indenting the wood surface.

[B] WAFERBOARD. A mat-formed wood structural panel product composed of thin rectangular wood wafers arranged in random layers and bonded with waterproof adhesive.

[B] WOOD STRUCTURAL PANEL. A structural panel product composed primarily of wood and meeting the requirements of United States Voluntary Product Standard PS 1 and United States Voluntary Product Standard PS 2. Wood structural panels include all-veneer plywood, composite panels containing a combination of veneer and wood-based material, and mat-formed panels such as oriented strand board and waferboard.

WOOD STRUCTURAL PANEL. A panel manufactured from veneers, wood strands or wafers or a combination of veneer and wood strands or wafers bonded together with waterproof synthetic resins or other suitable bonding systems. Examples of wood structural panels are:

Composite panels. A wood structural panel that is comprised of wood veneer and reconstituted woodbased material and bonded together with waterproof adhesive;

Oriented strand board (OSB). A mat-formed wood structural panel comprised of thin rectangular wood strands arranged in cross-aligned layers with surface layers normally arranged in the long panel direction and bonded with waterproof adhesive; or

Plywood. A wood structural panel comprised of plies of wood veneer arranged in cross-aligned layers. The plies are bonded with waterproof adhesive that cures on application of heat and pressure. **Reason:** This proposal updates Chapter A3 and provides consistency of definitions between the IEBC and the IBC. The proposal replaces definitions in current IEBC Chapter A3 with the definition of Wood Structural Panel (and the three example types) verbatim from 2012 IBC Chapter 2.

In addition, the definition of Waferboard is proposed to be deleted, as waferboard is no longer used for this application or widely produced.

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

EB24-12

Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
	-				A302-EB-BONOWITZ.doc

EB25–12 [B] A303.1

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B] A303.1 General. For the purposes of this chapter, <u>any of the following conditions shall be deemed a</u> <u>structural weakness:</u> structural weaknesses shall be as specified below.

1. Sill plates or floor framing that are supported directly on the ground without a foundation system that conforms to the building code.

(Portions of text not shown remains unchanged)

Reason: This proposal is an editorial improvement and clarification.

EB25-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				A303.1-EB-BONOWITZ

EB26–12 [B] A304.2.6, Chapter A6 (NEW)

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] A304.2.6 New sill plates. Where new sill plates are used in conjunction with new foundations, they shall be minimum 2x nominal thickness and shall be preservative-treated wood or naturally durable wood permitted by the building code for similar applications, and shall be marked or branded by an approved agency. Nails Fasteners in contact with preservative-treated wood shall be hot-dip galvanized or other material permitted by the building code for similar applications. Fasteners, whether cast-in-place or post-installed, that anchor a preservative-treated sill plate to the foundation shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum. Metal framing anchors in contact with preservative treated wood shall be galvanized in accordance with ASTM A 653 with a G 185 coating.

Add new standard to Chapter A6 as follows:

ASTM

B695-04 Standard Specification for Coating of Zinc Mechanically Deposited on Iron and Steel

Reason: This proposal makes two improvements related to metal hardware in contact with treated wood:

- In the second sentence, it replaces "nails" with "fasteners" to clarify that the provision is general.
- It inserts a sentence addressing allowable compliance for anchor bolts. The compliance details match those in 2012 IBC Section 2304.9.5.3.

Since ASTM B 695 is not yet used in the IEBC, the proposal adds it to Chapter A6. However, B 695 is already used in the IBC, so a copy is not provided with the proposal.

EB26-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				A304.2.6-EB-BONOWITZ.doc
EB27–12 [B] A304.3.1, [B] A304.3.2, [B] Table A3-A, [B] Table A3-B, [B]Figure A3-3

Proponent: David Bonowitz, Chair, Existing Buildings Subcommittee, Code Advisory Committee, National Council of Structural Engineers Associations (dbonowitz@att.net)

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

Revise as follows:

[B] A304.3.1 Existing perimeter foundations. Where the building has an existing continuous perimeter foundation, all perimeter wall sill plates shall be anchored to the foundation with adhesive anchors or expansion anchors in accordance with Table A3-A. Anchors shall be installed in accordance with Figure A3-3, with the plate washer installed between the nut and the sill plate. The nut shall be tightened to a snug-tight condition after curing is complete for adhesive anchors and after expansion wedge engagement for expansion anchors.

All anchors shall be installed in accordance with manufacturer's recommendations. Where existing conditions prevent anchor installations through the sill plate, this connection may shall be made in accordance with Figure A3-4A, A3-4B, or A3-4C. The spacing of these alternate connections shall comply with the maximum spacing requirements of Table A3-A. Expansion anchors shall not be used where the installation causes surface cracking of the foundation wall at the locations of the bolt anchor.

[B] A304.3.2 Placement of anchors. Anchors shall be placed within 12 inches (305 mm), but not less than 9 inches (229 mm), from the ends of sill plates and shall be placed in the center of the stud space closest to the required spacing. New sill plates may be installed in pieces where necessary because of existing conditions. For lengths of sill plates greater than 12 feet (3658 mm) <u>12 feet (3658 mm) or greater</u>, anchors or bolts shall be spaced along the sill plate as specified in Table A3-A. For other lengths of sill plate, anchor placement shall be in accordance with Table A3-B.

Exception: Where physical obstructions such as fireplaces, plumbing or heating ducts interfere with the placement of an anchor, the anchor shall be placed as close to the obstruction as possible, but not less than 9 inches (229 mm) from the end of the plate. Center-to-center spacing of the anchors shall be reduced as necessary to provide the minimum total number of anchors required based on the full length of the wall. Center-to-center spacing shall not be less than 12 inches (305 mm).

[B] TABLE A3-A SILL PLATE ANCHORAGE AND CRIPPLE WALL BRACING

a. Sill plate anchors shall be chemical adhesive anchors or expansion bolts anchors in accordance with Section A304.3.1.

(Portions of Table not shown remain unchanged)

[B] TABLE A3-B

SILL PLATE ANCHORAGE FOR VARIOUS LENGTHS OF SILL PLATE^{a,b}

a. Connections shall be either chemical adhesive anchors or expansion bolts anchors

(Portions of Table not shown remain unchanged)

[B] FIGURE A3-3 SILL PLATE BOLTING ANCHORING TO EXISTING FOUNDATION

(No change to figure)

Reason: The proposal makes terminology changes for consistency. The proposed wording change to Section A304.3.2 provides consistency with current Table A3-B.

EB27-12					
Public Hearing:	Committee:	AS	AM AME	D DE	
	Assembly.				A304.3.1-EB-BONOWITZ.doc

EB28–12 [B] A304.4.1.1

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B] A304.4.1.1 Sheathing installation requirements. Wood structural panel sheathing shall not be less than ${}^{15}/{}_{32}$ -inch (12 mm) thick and shall be installed in accordance with Figure A3-5 or A3-6. All individual pieces of wood structural panels shall be nailed with 8d common nails spaced 4 inches (102 mm) on center at all edges and 12 inches (305 mm) on center at each intermediate support with not less than two nails for each stud. Nails shall be driven so that their heads are flush with the surface of the sheathing and shall penetrate the supporting member a minimum of $1^{1}/_{2}$ inches (38 mm). When a nail fractures the surface, it shall be left in place and not counted as part of the required nailing. A new 8d nail shall be located within 2 inches (51 mm) of the discounted nail and be hand-driven flush with the sheathing surface. Where the installation involves horizontal joints, those joints shall occur over nominal 2-inch by 4-inch (51 mm by 102 mm) blocking installed with the nominal 4-inch (102 mm) dimension against the face of the plywood.

Vertical joints at adjoining pieces of wood structural panels shall be centered on studs such that there is a minimum 1/8 inch (3.2 mm) between the panels., and such that the nails are placed a minimum of ½ inch (12.7 mm) from the edges of the existing stud. Where such required edge distances cannot be maintained because of the width of the existing stud, a new stud shall be added adjacent to the existing studs and connected in accordance with Figure A3-7.



Reason: This proposal revises the edge distance requirement to avoid a potential problem nailing into narrow existing studs. The current requirement, shown in Figure A3-7, puts the nail 1/2 inch from the plywood edge; with a 2x stud, this leaves too little edge distance into the stud. A 3/8 inch edge distance in the plywood is considered adequate for this application.

In addition to changing the edge distance in Figure A3-7, the proposal makes the following improvements:

- Removes the duplicative edge distance requirement from Section A304.4.1.1, deferring to Figure A3-7.
- Revises wording in Figure A3-7, from "sheet metal connectors" to "framing clips" for consistency.
- Defines the height of the cripple wall, H, in Figure A3-7.

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

EB28-12

Public Hearing: Committee:	AS	AM	D
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Assembly:	ASF	AMF	DF	
,				A304.4.1.1-EB-BONOWITZ

EB29–12 [B]A403.5

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B]A403.5. <u>Deformation Compatibility and PA effects</u>. The requirements of the building code shall apply, except as modified herein. All structural framing elements and their connections not required by design to be part of the lateral force-resisting system shall be designed and/or detailed to be adequate to maintain support of design dead plus live expected gravity loads when subjected to the expected deformations caused by seismic forces. The stress analysis of cantilever columns shall use a buckling factor of 2.1 for the direction normal to the axis of the beam. Increased demand due to P Δ effects and story sidesway stability shall be considered in retrofit stories that rely on the strength and stiffness of cantilever columns for lateral resistance.

Reason:

The proposal makes a number of revisions related to the performance of gravity load-carrying columns subjected to lateral deformations within the retrofitted story:

- The title of the section is changed to reflect its actual concerns, which are greater than just P-delta effects.
- "Design dead plus live" loads represent an over-conservative requirement for existing elements that are not part of the lateral system, so only "expected gravity" are required.
- The current sentence about "stress analysis of cantilever columns" is unclear as to whether it is concerned with columns that are part of the lateral system (which would likely be columns added as part of the retrofit) or existing columns carrying only gravity loads. The proposed revision handles both situations:
 - For existing gravity columns, the current sentence is unnecessary. The first two sentences establish the general requirements. A specific effective length factor need not be given here, especially since it might be overconservative for the actual condition.
 - For columns that do resist lateral loads, the proposed new sentence clarifies that increased demands must be considered. Specific criteria are, appropriately, left to the engineer of record, subject to the general requirements in the first part of the section.

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

EB29-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	2				A403.5-EB-BONOWITZ

EB30–12 [B]A403.8

Proponent: Gary Searer, Wis, Janney, Elstner Associates, Inc, representing self

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

Revise as follows:

[B]A403.8: Horizontal diaphragms. The strength of an existing horizontal diaphragm sheathed with wood structural panels or diagonal sheathing need not be investigated unless the diaphragm is required to transfer lateral forces from vertical elements of the seismic force resisting system above the diaphragm to elements below the diaphragm because of an offset in placement of the elements.

Wood diaphragms with stories above shall not be allowed to transmit lateral forces by rotation or cantilever except as allowed by the building code; however, r-Rotational effects shall be accounted for when <u>asymmetric</u> unsymmetric wall stiffness increases shear demands.

Exception: Diaphragms that cantilever 25 percent or less of the distance between lines of lateralload-resisting elements from which the diaphragm cantilevers may transmit their shears by cantilever, provided that rotational effects on shear walls parallel and perpendicular to the load are taken into account.

Reason: None of these requirements is particularly clear, and none of these requirements is required or assists the engineer in understanding how the SWOF structure will behave. Specifically, by definition, all SWOF structures already use the diaphragm to transfer lateral forces (including by rotation or cantilever), but the intent of the deleted portions was not to trigger investigation of the floor diaphragm; indeed, no soft/weak/open front wood-framed structures have ever been identified where a structural wood panel diaphragm or diagonally sheathed diaphragm failed, resulting in a collapse in a prior earthquake (where the current, unclear requirement would have "caught" and prevented the failure.

"Unsymmetric" is not a word.

The exception is so unclear as to be useless (Is it an exception to the first paragraph of this section, the second paragraph, or both?), and even has the potential to make proper strengthening more difficult or less economical than required. For example, consider a 90-foot long by 25-foot wide structure, with a solid back wall and two end transverse walls. Assuming that this poorly worded exception is intended to take the distance between transverse walls times 25 percent (25 percent of 90 feet is 22.5 feet), this structure would not be allowed without adding strength and stiffness along the open front, although there is nothing wrong with having a robust lateral force resisting system that consists of the back wall and the two end transverse walls. If one then adds interior transverse walls at the third points, then the maximum cantilever counterintuitively drops to 25 percent of 30 feet or 7.5 feet, and you would <u>still</u> have to add strength and stiffness along the open front. Conversely, if the structure were 110 feet long by 25 feet wide, the structure would qualify for this exception <u>unless</u> the designer tried to add interior transverse walls at the third points -- again a counterintuitive result. Finally, rotational effects are already taken into account in the second paragraph, so how this is an exception is unclear.

EB30-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	2				A403.8-EB-SEARER

EB31–12 [B]A404.2.4

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B]A404.2.4 Shear wall hold-downs. Shear walls shall be provided with hold-down anchors at each end. Two hold-down anchors are required at intersecting corners. Hold-downs shall be approved connectors with a minimum ⁵/₈-inch-diameter (15.9 mm) threaded rod or other approved anchor with a minimum allowable load of 4,000 pounds (17.8 kN). Anchor embedment in concrete shall not be less than 5 inches (127 mm). Tie-rod systems shall not be less than ⁵/₈ inch (15.9 mm) in diameter unless using high strength cable. Threaded rod or high <u>High</u> strength cable elongation shall not exceed 5/8 inch (15.9 mm) using design forces under a 4,000 pound (17.8 kN) axial load.

Reason: This proposal clarifies the current requirement, acknowledging that Section A404 is a prescriptive approach, so there are no "design forces" to be applied. Instead, the required allowable strength from earlier in the section is used to gauge the cable axial stiffness. This is consistent with the 2009 IEBC commentary. Threaded rods are excluded from the elongation requirement because they have a minimum diameter given in the previous sentence (and because a 5/8" steel rod would easily meet the deflection limit).

EB31-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					4405 2 4-FB-BONOWITZ

EB32–12 [B]A404.2.4

Proponent: Marko Schotanus, Chair, Existing Buildings Committee, Structural Engineers Association of California (MSchotanus@ruthchek.com)

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

Revise as follows:

[B]A404.2.4 Shear wall hold-downs. Shear walls shall be provided with hold-down anchors at each end. Two hold-down anchors are required at intersecting corners. Hold-downs shall be approved connectors with a minimum ⁵/₈-inch-diameter (15.9 mm) threaded rod or other approved anchor with a minimum allowable load of 4,000 pounds (17.8 kN). Anchor embedment in concrete shall not be less than 5 inches (127 mm). Tie-rod systems shall not be less than ⁵/₈ inch (15.9 mm) in diameter unless using high strength cable. Threaded rod or high strength cable elongation shall not exceed 5/8 inch (15.9 mm) using design forces.

Reason: This proposal removes the unnecessary final sentence regarding hold-down stiffness. First, the current provision is impossible to implement because Section A405 is a prescriptive approach with no "design forces" and because the provision does not specify a length over which to measure or calculate the elongation. Second, if the 4000 pound allowable load from earlier in the provision is used to gauge the stiffness, the minimum diameter 5/8 inch rod would have to be over 100 ft long to see a 5/8 inch elongation. Typical cable systems, while less stiff than rods, are adequate as well. Finally, the 5/8 inch elongation limit means little in terms of performance, because different shear wall lengths and story heights will experience different drifts for the same hold down elongation.

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

EB32-12 Public Hearing: Committee: AS AM D Assembly: ASF AMF DF

EB33–12 [B]A503.2

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[A]A503.2 Properties of cast-in-place materials. Except where specifically permitted herein, the stress-strain relationship of concrete and reinforcement shall be determined from published data or by testing. All available information, including building plans, original calculations and design criteria, site observations, testing and records of typical materials and construction practices prevalent at the time of construction, shall be considered when determining material properties. For Tier 3 analysis, <u>nominal and</u> expected material properties shall <u>be established in accordance with Section 6.2 of ASCE 41.</u> be used in lieu of nominal properties in the calculation of strength, stiffness and deformability of building omponents. The procedure for testing and determination of material properties shall be from Section 6.2 of ASCE 41-06.

Reason: This proposal intends to update a reference standard and makes appropriate corresponding revisions. ASCE 41-06 Supplement No. 1 has been available to and in use by engineers for several years. It is available online, free, at http://content.seinstitute.org/publications/ASCE41supplement.html. A pdf version is being submitted with this proposal.

Supplement No. 1 modified the ASCE 41 modeling parameters and acceptance criteria for concrete elements of interest in IEBC Chapter A5. The modifications reflect recent testing and represent more rational and appropriately less conservative criteria than were in ASCE 41 previously. They should be used. The current criteria of Chapter A5 use expected material properties as a way of compensating for the previous conservatism of ASCE 41. Now that Supplement No. 1 is available, that compensation is no longer needed, and ASCE 41, with Supplement No. 1, may be referenced directly, as proposed.

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

Analysis: This change proposal references ASCE standard 41, which is already referenced in this code. However, the proposed change to code text is written to correlate with supplement 1 of the 2006 edition of the standard rather than the simply the 2006 edition presently referenced in the code. The update to this standard will be considered by the Administrative Code Committee during the 2013 Code Development Cycle. Should this code change proposal be approved, but the update to the standard not be approved by the Administrative Code Committee, the code text will revert to the text as it appears in the 2012 edition of the code.

EB33-12

Public Hearing: C	Committee:	AS	AM	D	
Ā	Assembly:	ASF	AMF	DF	
	-				A503.2-EB-BONOWITZ

EB34–12 [B] A503.2, [B] A504.1, [B] A505.1, [B]A506.3.2, [B] A507.1, [B]Chapter A6

Proponent: Jennifer Goupil, The Structural Engineering Institute of ASCE (jgoupil@asce.org)

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

Revise as follows:

[B] A503.2 Properties of cast-in-place materials.

Except where specifically permitted herein, the stress-strain relationship of concrete and reinforcement shall be determined from published data or by testing. All available information, including building plans, original calculations and design criteria, site observations, testing and records of typical materials and construction practices prevalent at the time of construction, shall be considered when determining material properties.

For Tier 3 analysis, expected material properties shall be used in lieu of nominal properties in the calculation of strength, stiffness and deformability of building components.

The procedure for testing and determination of material properties shall be from <u>ASCE 41 Section 10.2.</u> 6.2 of ASCE 41-06.

[B] A504.1 Site ground motion for Tier 1 analysis.

The earthquake loading used for the determination of demand on elements of the structure shall correspond to that required by <u>ASCE 41 Chapter 4.ASCE 31 Tier 1</u>.

[B] A505.1 General.

Structures conforming to the requirements of the ASCE <u>41 Chapter 431</u> Tier 1, Screening Phase, are permitted to be shown to be in conformance to this chapter by submission of a report to the building official as described in this section.

[B] A506.3.2 Component stiffness.

Component stiffness shall be calculated based on the approximate values shown in <u>ASCE 41</u> Table <u>10-5</u> 6-5 of ASCE 41.

[B] A507.1 General.

A Tier 3 evaluation shall be performed using the nonlinear procedures of <u>ASCE 41</u> Section <u>10.3.1.2.2</u> 6.3.1.2.2. of <u>ASCE 41</u>. The general assumptions and requirements of <u>ASCE 41 Section 10.3</u> Section 6.0, excluding concrete frames with in-fills shall be used in the evaluation. Site-ground motions in accordance with Section A504.3 are permitted for this evaluation.

Reason: The purpose of this proposal is update Appendix A5 to the recently updated ASCE 41-13, which is a combination of the two standards referenced in the 2012 IEBC (ASCE 31-03 and 41-06). The updated and combined standard follows the same three-tiered approach ASCE 31/41 so this proposal is simply an update of section references. The concrete provisions of ASCE 41-13 Chapter 4 (Tier 1 in A5) and Chapter 10 (Tier 3 in A5) have been updated based on recent research and also incorporate provisions adopted by the ACI 369 Committee as representative of the state of the practice for the seismic evaluation and retrofit of existing concrete buildings.

A public ballot version of the new standard will be available from ASCE in the spring of 2012 and it is expected that it a prepublication (white cover) version will be available prior to the ICC Final Action Hearings in October of 2012. Any person interested in obtaining a public comment copy of ASCE 41-13 may do so by contacting the proponent at jgoupil@asce.org.

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

Analysis: This change proposal references ASCE standard 41, which is already referenced in this code. However, the proposed change to code text is written to correlate with a new edition of this standard ASCE 41-13, rather than the edition presently referenced in the code, which is the 2006 edition. The 2013 edition of this standard is not yet completed, published and available.

The update to this standard will be considered by the Administrative Code Committee during the 2013 Code Development Cycle. Should this code change proposal be approved, but the update to the standard not be approved by the Administrative Code Committee, the code text will revert to the text as it appears in the 2012 edition of the code. Additionally, if the standard update is approved but the document is not published and available by December 1, 2014, an errata will be issued to the code that will return the affected code text to the text as it appears in the 2012 edition of the code.

EB34-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				A503.2-EB-GOUPIL

EB35–12 [B]A507.1

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B]A507.1 General. A Tier 3 evaluation shall be performed using the <u>nonlinear procedures Nonlinear</u> <u>Static Procedure or Nonlinear Dynamic Procedure</u> of Section <u>6.3.1.2.2.3</u> of ASCE 41. The general assumptions and requirements of Sections <u>2.0, 3.0 and</u> 6.0, excluding <u>those for</u> concrete frames with <u>infills infills</u>, shall be used in the evaluation. <u>Reduced IBC level Site site</u>-ground motions in accordance with Section A504.3 are permitted for this evaluation. <u>Structures meeting the ASCE 41 Life Safety (LS)</u> <u>acceptance criteria shall be deemed to comply with this chapter. If a Tier 3 analysis identifies</u> nonconforming conditions, such conditions shall be modified to conform to the acceptance criteria.

Reason: This proposal corrects and revises Chapter A5's references to ASCE 41. The proposed references to ASCE 41 Sections 2.0, 3.0, and 6.0, as opposed to just Section 6.0, give a more complete understanding of the various ASCE 41 provisions that Chapter A5 expects to be followed.

The proposed added sentence at the end of the section clarifies the Performance Level to be used with ASCE 41 in order to match the general intent of Chapter A5. This was always the intent of this section; it had just not been stated clearly before.

EB35-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				A507.1-EB-BONOWITZ

EB36–12 [B]C101.1, [B]C101.2, [B]C101.3 (New)

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B]C101.1 Intent and purpose. The provisions of this chapter provide prescriptive methods for selected structural retrofitting of existing buildings to increase their resistance to wind loads. Except as provided herein, other structural provisions of the *International Building Code* or the *International Residential Code* shall apply, as required.

[B]C101.2 Scope. The following prescriptive methods are intended for applications where the gable end wall framing is provided by a metal-plate-connected gable end frame or a conventionally framed gable end. The retrofits are appropriate for wall studs or webs spaced 24 inches (610 mm) on center maximum and oriented with the wide face either parallel or perpendicular to the surface of the gable end. Gable ends to be strengthened shall be permitted to be retrofitted using methods prescribed by this chapter.

[B]C101.1 Purpose. This chapter provides prescriptive methods for partial structural retrofit of an existing building to increase its resistance to out-of-plane wind loads. It is intended for voluntary use and for reference by mitigation programs. The provisions of this chapter do not necessarily satisfy requirements for new construction. Unless specifically cited, the provisions of this chapter do not necessarily satisfy requirements for structural improvements triggered by addition, alteration, repair, change of occupancy, building relocation or other circumstances.

[B]C101.2 Eligible buildings and gable end walls. The provisions of this chapter are applicable only to buildings that meet the following eligibility requirements:

- 1. <u>The building is not more than three stories tall, from adjacent grade to the bottom plate of each gable end wall being retrofitted with this chapter.</u>
- 2. The building is classified as Occupancy Group R3 (1-2 family dwellings)
- 3. <u>The structure includes one or more wood-framed gable end walls, either conventionally framed or metal-plate-connected.</u>

In addition, the provisions of this chapter are applicable only to gable end walls that meet the following eligibility requirements:

- <u>4.</u> Each gable end wall has or shall be provided with studs or vertical webs spaced 24 inches (610 mm) on center maximum.
- 5. Each gable end wall has a maximum height of 16 ft.

[B]C101.3 Compliance. Eligible gable end walls in eligible buildings may be retrofitted with this chapter. Eligible buildings with one or more ineligible gable end walls may be retrofitted with this chapter, provided all ineligible gable end walls are retrofitted with alternative criteria approved by the building official as equivalent. All other modifications required for conformance with this chapter shall be designed and constructed in accordance with the *International Building Code* or *International Residential Code* provisions for new construction except as specifically provided for by this chapter.

Reason: This proposal reorganizes, clarifies, and supplements the Chapter's provisions regarding intent, scope, eligibility, and compliance.

Proposed section C101.1 restates the first sentence of current section C101.1 and adds two clarifying sentences that confirm the relationship of this chapter to the rest of the IEBC and to other I-codes (similar to the current text of Section C201.1). Chapter C1 was added to the 2012 IEBC as a good idea suitable for voluntary use but not benchmarked in terms of performance. Because other IEBC provisions at times call for structural evaluation or retrofit to resist wind loads, it is important to be clear that Chapter C1 does not necessarily satisfy those requirements.

Proposed section C101.2 lays out the eligibility requirements in a more direct and specific way:

- Item 1: The proposed three-story limit is new, but it reflects our understanding (based on review of the supporting
 calculations and Chapter history) of the intent of Chapter C1 to apply to typical 1-2 unit dwellings of conventional wood
 framing. Given the limits of the Chapter's supporting studies and past applications, it would be wrong to encourage this
 retrofit scheme for taller or more complex structures that happen to have wood framed gable end walls.
- Item 2: The proposed occupancy eligibility rule is new, but it again reflects our understanding of the intent of Chapter 1 to apply to typical 1-2 unit dwellings. Given the limits of the Chapter's supporting studies, past applications, and lack of benchmarking by risk category, it would be wrong to encourage this retrofit scheme for multi-unit complexes or for assisted living, commercial, educational, or other occupancies simply because the building looks like a house. (For ease of use by homeowners and residential contractors, we have proposed this eligibility limit in terms of occupancy. Alternatively, because the governing load is extreme wind, eligibility could be written in terms of risk category with reference to IBC Table 1604.5.)
- Item 3: This is a simple provision that merely confirms the presence of the structural elements of interest.
- Item 4: The 24 inch spacing requirement matches the current provision in C101.2. The proposed rule adds an allowance that a non-conforming structure may be made to conform through the retrofit.
- Item 5: The 16 ft height limit comes from current Table C104.2. It is useful to have such eligibility rules in one place near the top of the chapter.

Proposed section C101.3 implements the eligibility rules of proposed section C101.2 and explicitly addresses the case of buildings where some gable end walls are eligible and others are not. The final sentence restates the provision from current section C101.1, but in an appropriate place. The text is borrowed from IEBC A403.1, which has the same intent.

In summary, the proposal is measured and fair, and it respects the intention of the Chapter and its proponents. We have limited the proposal to basic issues, leaving aside remaining questions regarding, for example, maximum spans, suitable roof sheathing, suitable ceiling construction, and suitable exterior wall sheathing or siding.

EB36-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				C101.1-EB-BONOWITZ

EB37–12 [B]C201.1, [B]C201.2

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B]C201.1 Intent and purpose. The provisions of this chapter provide prescriptive methods for selected structural retrofitting of existing buildings. Compliance with these provisions will not always meet the requirements for new construction in the *International Building Code* or the *International Residential Code*. The provisions of this chapter are intended to provide methods for strengthening existing buildings to increase resistance to wind loads.

[B]C201.2 Scope. The provisions of this chapter are a prescriptive alternative for one- and two-family dwellings located where the wind speed according to Section 1609 of the *International Building Code* exceeds 100 mph (44.7 m/s) to achieve compliance with Section 706.3 of the *International Existing Building Code*.

[B]C201.1 Purpose. This chapter provides prescriptive methods for partial structural retrofit of an existing building to increase its resistance to wind loads. It is intended for voluntary use and for reference by mitigation programs. The provisions of this chapter do not necessarily satisfy requirements for new construction. Unless specifically cited, the provisions of this chapter do not necessarily satisfy requirements for structural improvements triggered by addition, alteration, repair, change of occupancy, building relocation or other circumstances.

[B]C201.2 Eligible conditions. The provisions of this chapter are applicable only to buildings that meet the following eligibility requirements:

1. Buildings assigned to risk category I or II per International Building Code Table 1604.5.

Reason: This proposal clarifies and corrects the Chapter's provisions regarding intent, scope, and eligibility.

Proposed section C201.1 restates current section C201.1 and adds a clarifying sentence that confirms the relationship of this chapter to the rest of the IEBC and to other I-codes. Chapter C2 was added to the 2012 IEBC as a good idea suitable for voluntary use but not benchmarked in terms of performance. Because other IEBC provisions at times call for structural evaluation or retrofit to resist wind loads, it is important to be clear that Chapter C2 does not necessarily satisfy those requirements. In particular, the statement in current section C201.2 regarding compliance with Section 706.3 is for that reason proposed for deletion.

Proposed section C201.2 expands the current reference to "one- and two-family dwellings." Since nothing in Chapter C2 presumes a building use or a construction type specific to R3 occupancy, the Chapter actually has broader applicability than is currently stated. The appropriate limit is to risk category I and II buildings, as proposed. Also, there is no need to state a minimum wind speed in the provision; if the criteria are good for wind speeds over 100 mph, they are also good for lower demands.

EB37-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					C201.1-EB-BONOWITZ

EB38–12 [B] C201.2, [B] Table C202.1.2

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB) (gehrlich@nahb.org)

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

Revise as follows:

[B] C201.2 Scope. The provisions of this chapter are a prescriptive alternative for one- and two-family dwellings located where the <u>ultimate design</u> wind speed V_{ult} determined in accordance with Figure 1609A according to Section 1609 of the International Building Code exceeds 130 mph (58 m/s) 100 mph (44.7 m/s) to achieve compliance with Section 706.3 of the International Existing Building Code.

[B] TABLE C202.1.2 SUPPLEMENTAL FASTENERS AT PANEL EDGES AND INTERMEDIATE FRAMING

			MAXIMUM	
			SUPPLEMENTAL	EDGE ZONE ^d FOR
		MAXIMUM	FASTENER SPACING	WIND SPEED GREATER
	EXISTING	SUPPLEMENTAL	FOR INTERIOR ZONE ^C	THAN V _{ULT} > 160 MPH
EVICTING	FASTENER	FASTENER SPACING	LOCATIONS FOR WIND	120 MPH AND
EAISTING	SPACING (EDGE	FOR WIND SPEEDS	SPEEDS EXCEEDING	EXPOSURE C, OR
FASIENERS	OR INTERMEDIATE	GREATER THAN 100	<u>V_{ULT} > 140 MPH 110</u>	WIND SPEED GREATER
	SUPPORTS)	MPH 130 MPH < V _{ULT}	MPH AND EDGE ZONES	ТНАН <u>V_{ULT} > 180 МРН</u>
		<u>≤ 140</u> MPH	NOT COVERED BY THE	140 MPH AND
			COLUMN TO THE	EXPOSURE B
			RIGHT	

(Portions of table not shown remain unchanged)

Reason: The purpose of this proposal is to correlate basic wind speed triggers in the IEBC with the IBC. The 2012 IBC adopted new ultimate-strength basis wind speed maps from ASCE 7-10. A conversion factor from the ultimate wind speed selected from the new maps (V_{ult}) down to the old allowable-stress level wind speed (V_{asd}) was introduced into the IBC to accommodate triggers for special requirements in high-wind regions, tables limiting the use of ballasted roofs at certain heights and wind speeds, and tables for proper selection of shingles and other roofing materials for wind resistance. Unfortunately, this conversion was not introduced into the IEBC, with the result that provisions which were supposed to apply only in high-wind regions now appear to apply across the entire United States. This proposal not only corrects this oversight, it fully updates the IEBC provisions to match the 2012 IBC and ASCE 7-10.

EB38-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				C201.2-EB-EHRLICH

EB39–12 [B] Figure A3-1, [B] Figure A3-2

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B] FIGURE A3-1 NEW REINFORCED CONCRETE FOUNDATION SYSTEM

- a. Where frost conditions occur, the minimum depth shall extend below the frost line.
- b. The ground surface along the interior side of the foundation may be excavated to the elevation of the top of the footing.
- c. When expansive soil is encountered Where the code official has designated the soil as expansive, the foundation depth and reinforcement shall be as directed approved by the building code official.

(Portions of figure not shown remain unchanged)

[B] FIGURE A3-2 NEW MASONRY CONCRETE FOUNDATION

- a. Where frost conditions occur, the minimum depth shall extend below the frost line.
- b. The ground surface along the interior side of the foundation may be excavated to the elevation of the top of the footing.
- c. When expansive soil is encountered Where the code official has designated the soil as expansive, the foundation depth and reinforcement shall be as directed approved by the building code official.

(Portions of figure not shown remain unchanged)

Reason: This proposal clarifies the intended applicability and alternative criteria for expansive soil conditions. The intent of these notes is simply that the default, tabulated values might not be appropriate for highly expansive soil. Since most building departments are aware of local expansive soil conditions (and might even have their own prescriptive pre-approved details), the intent is to call attention to those known cases. Thus, the current wording about "when expansive soil is encountered" gives the wrong impression. Instead, since this chapter presumes no engineered design, there should be no burden on the builder to know or discover the soil conditions. Rather, the burden should merely be to check if the code official has made a designation, and if so, to get appropriate plan check approval for the footing details.

EB39-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					F A3-1-EB-BONOWITZ

EB40–12 [B] Figure A3-4A

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise Figure A3-4A as follows:

1. Revise note at top left:

Existing 2x BLOCKING OR RIM JOIST WITH EXISTING TOENAILS. SEE SECTION A304.1.4 A304.1.3

2. Revise long note at right side: 7" x 3/16" x 9" LONG PLATE WITH (2) – ½" DIAMETER ADHESIVE ANCHORS OR EXPANSION BOLTS ANCHORS TO FOUNDATION WALL ...

3. Correct note 1 [preferably through errata to the 2012 edition]:

1. If shim space exceeds $\frac{2 \frac{1}{2} \text{ in.}}{1 \frac{1}{2} \text{ in.}}$, alternate details will be required.

4. Revise note 2:

Where required, single piece shim shall be foundation grade redwood <u>naturally durable wood</u> or preservative-treated wood. If preservative-treated wood is used, it shall be isolated from the foundation system with a moisture barrier.

5. Correct [preferably through errata to the 2012 edition] and revise title: FIGURE A3-4A: SILL PLATE BOLTING IN EXISTING FOUNDATION—ALTERNATE ALTERNATE SILL PLATE ANCHORING IN EXISTING FOUNDATION WITHOUT CRIPPLE WALLS AND FLOOR FRAMING NOT PARALLEL TO FOUNDATIONS



Reason: The proposal makes five editorial changes, two of which reflect errors in production of the 2012 edition (first printing, April 2011) that should preferably be corrected through errata. Bases for the five proposed changes are:

- 1. Correction of cited code section.
- 2. Editorial revision for consistent terminology ("anchor," not "bolt")
- 3. Errata. The correct value of 1 ½ in. was in approved proposal EB54-09/10 but did not make it into print.
- 4. Editorial revision for consistency with current Section A304.2.6, which was revised for 2012.
- 5. Errata, with one editorial change for 2015. The correct title, "Alternate sill plate ...", was in approved proposal EB54-09/10 but did not make it into print. That approved title actually read "Alternate sill plate bolting in existing foundation ...". For terminology consistency, it should now read as proposed here: "Alternate sill plate anchoring in existing foundation ...".

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

EB40-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				F A3-4A-EB-BONOWITZ

EB41–12 [B] Figure A3-10

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise Figure A3-10 as follows:



For SI: 1 toch = 25.4 mm, 1 foot = 304.8 mm.

FIGURE A3-10-FLOOR PLAN-CRIPPLE WALL BRACING LAYOUT

Reason: The proposal corrects the dimensions shown at the top of Figure A3-10 for 1-story buildings. The calculations at the bottom of the figure are correct, so the figure should be revised in three ways:

- Delete the dimension strings showing 11'-10" spacing between panel centers.
- Change the end panel lengths from 5'-4" to 4'-0" in two places.
- Redraw the end panel lengths to approximate scale as 4-ft long sections.

EB41-12			
Public Hearing: Committee:	AS	AM	D

Assembly: ASF AMF DF	F A3-10-EB-BONOWITZ
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EB42–12 [B] Table A3-A, [B] Figure A3-3

Proponent: David Bonowitz, S.E., representing NCSEA Code Advisory Committee, Existing Buildings Subcommittee (dbonowitz@att.net)

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

Revise as follows:

[B] TABLE A3-A SILL PLATE ANCHORAGE AND CRIPPLE WALL BRACING

- a. Sill plate anchors shall be chemical anchors or expansion bolts in accordance with Section A304.3.1.
- b. All washer plates shall be <u>3 inches by 3 inches by .229 inch (76 mm x 76 mm x 5.8 mm)</u> <u>2 inches by 2 inches by ³/₁₆ inch (51 mm by 51 mm by 4.8 mm)</u> minimum.
- c. See Figure A3-10 for braced panel layout.
- d. Braced panels at ends of walls shall be located as near to the end as possible.
- e. All panels along a wall shall be nearly equal in length and shall be nearly equal in spacing along the length of the wall.
- f. The minimum required underfloor ventilation openings are permitted in accordance with Section A304.4.4.

(Portions of Table not shown remain unchanged)

[B] FIGURE A3-3 SILL PLATE BOLTING TO EXISTING FOUNDATION

For SI: 1 inch = 25.4 mm.

NOTES:

1. Plate washers shall comply with the following:

1/2 in. anchor or bolt – 2 in. x 2 in. x 3/16 in. 3 in x 3 in x 0.229 in (76 mm x 76 mm x 5.8 mm) minimum

- ⁵/₈ in. anchor or bolt 2 in. x 2 in. x 3/16 in. <u>3 in x 3 in x 0.229</u> in (76 mm x 76 mm x 5.8 mm) minimum
- 2. See Figure A3-5 or A3-6 for cripple wall bracing.

(Portion of Figure not shown remains unchanged)

Reason: This proposal coordinates the minimum washer size with provisions in IRC Section R602.11. The change is made to both Table A3-A (note b) and Figure A3-3 (note 1).

Note to ICC: The washer size listed in 2012 Figure A3-3 note 1 should already be 3" x 3" x 1/4" per EB54-09/10, but that approved change was apparently not picked up in publication. This should be corrected through IEBC errata

EB42-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	,				T A3-A-EB-BONOWITZ

S1–12 202 (NEW), 1504.4, 1504.6, 1504.7, 1507.12.3, 1507.13.3

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Add new text as follows:

SECTION 202 DEFINITIONS

LOW SLOPE (For application to Chapter 15 only). 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).

Revise as follows:

1504.4 Ballasted low-slope roof systems. Ballasted lowslope (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 lowslope 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.

1507.12.3 Ballasted thermoset low-slope roofs. Ballasted thermoset low-slope roofs (roof slope < 2:12) shall be installed in accordance with this section and Section 1504.4. Stone used as ballast shall comply with ASTM D 448.

1507.13.3 Ballasted thermoplastic low-slope roofs. Ballasted thermoplastic low-slope roofs (roof slope < 2:12) shall be installed in accordance with this section and Section 1504.4. Stone used as ballast shall comply with ASTM D448.

Reason: This proposed code change is intended to add clarity to the code by providing a specific definition in Chapter 2— Definitions for the term "low slope", which 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 Section 1504.5, the term is not specifically defined. Adding a specific definition for the term in Section 202—Definitions provides for consistent interpretation throughout the Chapter 15 and allows removal of the parenthetical definition "...(roof slope < 2:12)..."

The addition of the notation to the term 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 202—Definitions for the term "Roof assembly."

S1-12					
Public Hearing: Co	ommittee:	AS	AM	D	
As	sembly:	ASF	AMF	DF	
	•				1504.4-S-GRAHAM

S2–12 202, 1505.8, 1507.17, 1507.17.1, 1507.17.2, 1507.17.3

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

SECTION 202 DEFINITIONS

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

Revise as follows:

1505.8 Photovoltaic systems. Rooftop installed photovoltaic systems that are adhered or attached to the roof covering or photovoltaic modules/shingles installed as roof coverings shall be labeled to identify their fire classification in accordance with the testing required in Section 1505.1.

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 and acceptance criteria in ASTM D 3161. Photovoltaic modules/shingles shall comply with the classification requirements of Table 1507.2.7.1(2) for the appropriate maximum nominal design wind speed. Photovoltaic modules/shingle packaging shall bear a label to indicate compliance with the procedures in ASTM D 3161 and the required classification from Table 1507.2.7.1(2).

Reason: This code change proposal is intended to clarify the term and definition for "Photovoltaic modules/shingles" in Chapter 2-Definitions and carrying this clarification through to the specific requirements for photovoltaic shingles in Section 1507.17

The word "modules" is being deleted from the term and definition because it is not defined in the code in the context of photovoltaic applications and it is not necessary to clearly identify and define the term. Similarly, "/" is being deleted because it is not necessary to identify or define the term; it is not clear whether the "/" is intended to mean "and" or "or". Also, "flat-plate", "three-tab" and "composite" are being deleted because these are not defined in the IBC and these are not necessary to clearly define the term.

The changes in Section 1505.8 and Section 1507.17 are intended to make the terminology consistent with the revised term in Chapter 2-Definitions.

No changes in the current code's technical requirements are intended with this code change proposal.

S2-12					
Public Hearing: Con	nmittee:	AS	AM	D	
Ass	embly:	ASF	AMF	DF	
	-				1505.8 #1-S-GRAHAM

S3–12 202 (NEW), 1505.8, 1509.7, 1509.7.1, 1509.7.2, 1509.7.3, 1511, 1511.1, 3111, 3111.1

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Add new text as follows:

SECTION 202 DEFINITIONS

PHOTOVOLTAIC MODULE. A complete, environmentally protected unit consisting of solar cells, optics, and other components, exclusive of tracker, designed to generate DC power when exposed to sunlight.

PHOTVOLTAIC PANEL. A collection of modules mechanically fastened together, wired, and designed to provide a field-installable unit.

Revise as follows:

1505.8 Photovoltaic systems panels and modules. Rooftop installed photovoltaic systems panels and modules that are adhered or attached to the roof covering or photovoltaic modules/shingles installed as roof coverings shall be labeled to identify their fire classification in accordance with the testing required in Section 1505.1.

1509.7 Photovoltaic systems panels and modules. Rooftop mounted photovoltaic systems panels and modules shall be designed in accordance with this section.

1509.7.1 Wind resistance. Rooftop mounted photovoltaic systems panels and modules shall be designed for wind loads for component and cladding in accordance with Chapter 16 using an effective wind area based on the dimensions of a single unit frame.

1509.7.2 Fire classification. Rooftop mounted photovoltaic systems panels and modules shall have the same fire classification as the roof assembly required by Section 1505.

1509.7.3 Installation. Rooftop mounted photovoltaic systems panels and modules shall be installed in accordance with the manufacturer's installation instructions.

SECTION 1511 SOLAR PHOTOVOLTAIC PANELS AND MODULES

1511.1 Solar photovoltaic panels <u>and</u> modules. Solar photovoltaic panels/modules installed upon a roof or as an integral part of a roof assembly shall comply with the requirements of this code and the *International Fire Code*.

1511.1.1 Structural fire resistance. The structural frame and roof construction supporting the load imposed upon the roof by the photovoltaic panels <u>and</u> modules shall comply with the requirements of Table 601.

Revise as follows:

SECTION 3111 SOLAR PHOTOVOLTAIC PANELS AND MODULES

3111.1 General. Solar photovoltaic panels <u>and</u> modules shall comply with the requirements of this code and the *International Fire Code*.

Reason: This code change proposal is intended to clarify the code by providing specific terms and definitions for photovoltaic devices addressed in the code and then carrying these terms and definitions through to the code's current specific requirements in Section 1505.8, 1509.7, 1511 and 3111.

IBC 2012 currently uses the terminology "photovoltaic systems", which is currently not defined and is not widely recognized in the PV industry. For example, some have questioned whether the term "photovoltaic systems" includes racking and mounting systems, and external wiring. As a result, there appears to be some confusion and possible misinterpretation of the IBC's requirements.

The definitions for the terms "Photovoltaic module" and "Photovoltaic panel" are taken from NFPA 70, "National Electrical Code, 2011 Edition." NFPA is not currently referenced in the IBC; however, it is referenced as a requirement in the *International Fire Code*, Section 605.11.

In Section 1505, the change from "...systems..." to "...panels and modules..." is being made for consistency with the new definitions ion Chapter 2. Also, photovoltaic modules and panels are fire classified according to ASTM E108 or UL790 (and UL1703), which are already included in the IBC. Other photovoltaic system components--such as racking and mounting systems, and external wiring—are not currently fire classified.

and external wiring—are not currently fire classified. In Section 1509.7 and Section 1511, the change from "...systems..." to "...panels and modules..." is being made for consistency with the new definitions in Chapter 2. Also, the terminology "...panels and modules..." already occurs in IBC 2012's Section 1509.7.4.

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

S3-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
, ,				1505.8 #2-S-GRAHAM

S4–12 202 (NEW), 1507.16, 1507.16.1, 1607.12.3, 1607.12.3.1

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Add new text as follows:

SECTION 202 DEFINITIONS

Vegetative roof. An assembly of interacting components designed to waterproof and normally insulate a building's top surface that includes, by design, vegetation and related landscape elements.

Revise as follows:

1507.16 <u>Vegetative roofs</u>, roof gardens and landscaped roofs. <u>Vegetative roofs</u>, roof gardens and landscaped roofs shall comply with the requirements of this chapter and Sections 1607.12.3 and 1607.12.3.1 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 <u>vegetative roof</u>, roof gardens or landscaped roofs shall comply with the requirements of Table 601.

Revise as follows:

1607.12.3 Occupiable roofs. Areas of roofs that are occupiable, such as <u>vegetative roofs</u>, roof gardens, or for assembly or other similar purposes, and marquees are permitted to have their uniformly distributed live loads reduced in accordance with Section 1607.10.

1607.12.3.1 <u>Vegetative and</u> landscaped roofs. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m2). The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

Reason: This code change proposal is intended to use terminology in the IBC that is consistent with that of the *International Green Construction Code* (IgCC). IgCC uses the terminology "vegetative roof" for what is referred to in the IBC as a "roof garden" or "landscaped roof".

This code change proposal adds a definition for the term "vegetative roof" in Section 202. The definition is identical to that in the IgCC and ASTM D1079, "Standard Terminology Relating to Roofing and Waterproofing." The term "vegetative roof" is also added where appropriate in Section 1507.16 and Section 1607.12.3.

S4-12					
Public Hearing: C	committee:	AS	AM	D	
A	ssembly:	ASF	AIVIE	DF	1507 16-S-GRAHAM

S5–12 202 (NEW)

Proponent: Christine Covington, Solar Energy Industries Association

Add new text as follows:

SECTION 202 DEFINITIONS

BUILDING INTEGRATED PHOTOVOLTAIC (BIPV) SYSTEM. A system that incorporates

photovoltaic modules, which covert solar radiation into electricity, as a component of building products that simultaneously provide protection against weather and water entry into the building envelope.

PHOTOVOLTAIC PANEL SYSTEM. A system that incorporates discrete photovoltaic panels, which covert solar radiation into electricity, onto rack support systems which are supported by building structural systems such as roof, floor, or wall assemblies.

Reason: The IBC references different applications of photovoltaic systems in various locations throughout the code without definition. The intent of this change is to provide basic definitions for photovoltaic systems that are embedded in building construction elements (BIPV's) and for systems that are installed extraneous to new or existing building elements (Panel Systems). This is critical in determining the type of testing that will be appropriate for each system. Currently, BIPV's used as roof shingles must pass UL 790 or ASTM E108 to determine fire classification while panel systems used above fire classified roofs must undergo testing in conjunction with UL 1703.

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

S5-12				
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

202-BUILDING INTEGRATED PHOTOVOLTAIC SYSTEM-S-COVINGTON.doc

S6–12 1503.2 (NEW), 1510.1

Proponent: Mark S. Graham/National Roofing Contractors Association/representing National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1503.2 Energy efficiency: Roof assemblies shall be designed and constructed in accordance with Chapter 13 and the *International Energy Conservation Code*.

1510.1 General. Materials and methods of application used for recovering or replacing an existing roof covering shall comply with the requirements of Chapter 15.

Exceptions:

- Reroofing shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 for roofs that provide positive roof drainage.
- 2. Reroofing is permitted without requiring the entire building or structure comply with Section 1503.2 and the energy requirements of the International Energy Efficiency Code. Roof replacement shall conform to the energy requirements for roof assemblies of the International Energy Efficiency Code.

Reason: : This code change proposal is intended to add a direct statement in Chapter 15 indicating roof assemblies are required to comply with Chapter 13-Energy Efficiency and the *International Energy Conservation Code*.

For reroofing, the proposed new language in Section 1510.1 is intended to clarify reroofing does not require upgrading the entire building (or structure) to the current energy code. Roof replacement shall comply with the current energy code. The terms "reroofing" and "roof replacement" are already defined in Chapter 2-Definitions.

S6-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1503.2(NEW)-S-GRAHAM.doc

S7–12 1503.5

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1503.5 Roof <u>Attic</u> ventilation. Intake and exhaust vents shall be provided in accordance with Section 1203.2 and the <u>roof covering</u> manufacturer's installation instructions.

Reason: This code change proposal is intended to clarify the intent of the Code.

While Section 1503.5 is titled "Roof ventilation," the section that is referenced is Section 1203.2-Attic Spaces. On this basis, change the title of Section 1503.5 to "Attic ventilation" appears appropriate.

Also, the code language also makes reference "...the manufacturer's installation instruction." But does not clearly stipulate the manufacturer of which product (roof covering, roof deck, etc.) is intended. "...roof covering..." is added in this proposal to clarify compliance with the roof covering manufacturer's installation instruction are intended to required.

S7-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1503.5-S-GRAHAM

S8–12 1504.1.1, Table 1504.1.1(1), Table 1504.1.1(2), 1507.2.7.1, Table 1507.2.7.1(1), Table 1507.2.7.1(2), 1609.5.2

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1504.1.1 Wind resistance of asphalt shingles. Asphalt shingles shall <u>comply with Section 1507.2.7</u>.<u>be</u> tested in accordance with ASTM D 7158. Asphalt shingles shall meet the classification requirements of Table 1504.1.1(1) for the appropriate maximum basic wind speed. Asphalt shingle packaging shall bear a label to indicate compliance with ASTM D 7158 and the required classification in Table 1504.1.1(1).

Exception: Asphalt shingles not included in the scope of ASTM D 7158 shall be tested and labeled to indicate compliance with ASTM D 3161 and the required classification in Table 1504.1.1(2).

TABLE 1504.1.1(1) CLASSIFICATION OF ASPHALT ROOF SHINGLES IN ACCORDANCE WITH ASTM D 7158^ª

NOMINAL DESIGN WIND SPEED, V _{asd} ^b (mph)	CLASSIFICATION REQUIREMENT
<u>85</u>	<u>D, G or H</u>
<u>90</u>	<u>D, G or H</u>
<u>100</u>	<u>G or H</u>
<u>110</u>	<u>G or H</u>
<u>120</u>	<u>G or H</u>
<u>130</u>	H
<u>140</u>	H
<u>150</u>	<u>H</u>

For SI: 1 foot = 304.8 mm; 1 mph = 0.447 m/s.

a. The standard calculations contained in ASTM D 7158 assume exposure category B or C and building height of 60 feet or less. Additional calculations are required for conditions outside of these assumptions.

b. V_{asd} shall be determined in accordance with Section 1609.3.1.

TABLE 1504.1.1(2)

NOMINAL DESIGN WIND SPEED, V _{asd} ^a	CLASSIFICATION REQUIREMENT		
<u>(mph)</u>			
85	<u>A, D or F</u>		
90	<u>A, D or F</u>		
<u>100</u>	<u>A, D or F</u>		
<u>110</u>	E.		
<u>120</u>	E.		
<u>130</u>	E		
<u>140</u>	E		
<u>150</u>	Ē		

For SI: 1 mph = 0.447 m/s.

a. V_{asd} shall be determined in accordance with Section 1609.3.1.

1507.2.7.1 Wind resistance. Asphalt shingles shall be tested in accordance with ASTM D 7158. Asphalt shingles shall meet the classification requirements of Table 1507.2.7.1(1) for the appropriate maximum basic wind speed. Asphalt shingle packaging shall bear a label to indicate compliance with ASTM D 7158 and the required classification in Table 1507.2.7.1(1).

Exception: Asphalt shingles not included in the scope of ASTM D 7158 shall be tested and labeled to indicate compliance with ASTM D 3161 and the required classification in Table 1507.2.7.1(2).

TABLE 1507.2.7.1(1) CLASSIFICATION OF ASPHALT ROOF SHINGLES PER ASTM D 7158^a

NOMINAL DESIGN WIND SPEED, Vasd ^B	CLASSIFICATION REQUIREMENT		
(mph)			
85	D, G or H		
90	D, G or H		
100	G or H		
110	G or H		
120	G or H		
130	H		
140	H		
150	Ħ		

For SI: 1 foot = 304.8 mm; 1 mph = 0.447 m/s.

a. The standard calculations contained in ASTM D 7158 assume exposure category B or C and building height of 60 feet or less. Additional calculations are required for conditions outside of these assumptions.

. V_{asd} shall be determined in accordance with Section 1609.3

TABLE 1507.2.7.1(2) CLASSIFICATION OF ASPHALT SHINGLES PER ASTM D 3161

NOMINAL DESIGN WIND SPEED, V _{asd} ^a (mph)	CLASSIFICATION REQUIREMENT
85	A, D or F
90	A, D or F
100	A, D or F
110	– F
120	Ę.
130	Ę.
140	E.
150	F

For SI: 1 mph = 0.447 m/s.

a. Vasd shall be determined in accordance with Section 1609.3.1.

Revise as follows:

1609.5.2 Roof coverings. Roof coverings shall comply with Section 1609.5.1.

Exception: Rigid tile roof coverings that are air permeable and installed over a roof deck complying with Section 1609.5.1 are permitted to be designed in accordance with Section 1609.5.3.

Asphalt shingles installed over a roof deck complying with Section 1609.5.1 shall comply with the wind resistance requirements of Section 1507.2.7.1 1504.1.1.

Reason: This code change proposal is intended to relocate the Code's wind resistance requirements for asphalt shingles to the same section where similar wind resistance requirements are provided for other roof system types.

Wind resistance requirements (e.g., testing, classification) for all roof system types and components—other than those for asphalt shingles—are provided in Section 1504-Performance Requirements. The wind resistance requirements for asphalt shingles are currently provided in Section 1507-Requirements for Roof Coverings, specifically in Section 1507.2.7.1-Wind Resistance. The placement of the wind resistance requirements in the asphalt shingle section instead of the performance requirements section dates back to the legacy codes era when wind resistance for asphalt shingles was addressed by prescriptive language (e.g., four or six fasteners per strip shingle) instead of performance-based measures. Today, specific test methods (ASTM D7158 and ASTM D3161) and classifications (Class D, Class F, Class G, etc.) exist and are incorporated into the IBC making placement of the requirements for asphalt singles in Section 1504-Performance Requirements appropriate. Section 1504.1.1-Wind Resistance of Asphalt Shingles are already exists in Section 1504-Performance Requirements and currently serves as a pointer to Section 1507.2.7. This code change proposals moves the applicable wind resistance language from Section 1507.2.7 to Section 1504.1.1, replacing the pointer. Also, in

Chapter 16-Structural Design, a pointer to Section 1507.2.7.1 occurs in the Exception to Section 1509.5.2-Roof Coverings; this pointer is redirected to Section 1504.1.1.

This code change proposal does not include any technical changes in the wind resistance requirements for asphalt shingles. This code change proposal is merely a rearrangement into the proper location of the Code's existing requirements for asphalt shingles wind resistances.

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

S8-12

Public Hearing: Con	nmittee:	AS	AM	D	
Ass	embly:	ASF	AMF	DF	
	-				1504.1.1-S-GRAHAM

S9–12 1504.3.1.1 (NEW), Chapter 35 (NEW)

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

Add new text as follows:

1504.3.1.1 Nonballasted low slope roofs. Nonballasted low slope (roof slope < 2:12) roof systems with built-up, modified bitumen, fully adhered or mechanically attached single-ply shall be installed in accordance with ANSI/SPRI WD-1.

Add new standard to Chapter 35 as follows:

ANSI/SPRI

WD-1-XX Wind Design Standard Practice for Roofing Assemblies

Reason: There are two primary reasons that ANSI/SPRI WD-1 should be included as a reference standard in the IBC.

1. 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. This safety factor has historically been used by the roofing industry to account for variables between tested loads and performance in the field. These variables include deviations in installation and the fact that the wind load test procedures used incorporate static applied loads while dynamic, cyclic loads occur in the field. 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 proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S9-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1504.3.1.1 (NEW)-S-ENNIS
S10–12 1504.3.1

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1504.3.1 Other roof systems. Roof systems with built-up, modified bitumen, fully adhered or mechanically attached single-ply-through fastened metal panel roof systems, and other types of membrane roof coverings shall also be tested in accordance with FM 4474, UL 580 or UL 1897.

Reason: This code change proposal is intended to clarify the Code.

This code change proposal intends to remove "...through fastened metal panel..." from Section 1504.3.1 as this specific roof system type is already addressed in Section 1504.3.2-Metal Panel Roof Systems. The inclusion of wind resistance requirements for through fastened metal panel roof systems in Section 1504.3.1 appears to be a misprint as test methods FM 4474 and UL 1897 included in this section do not apply to metal panel roof systems.

Addressing the wind resistance of through fastened metal panel roof system in Section 1504.3.2-Metal Panel Roof Systems is appropriate as the test methods in this particular section are applicable to metal panel roof systems.

S10-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1504.3.1-S-GRAHAM

S11–12 1504.3.1

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

1504.3.1 Other roof systems. Roof systems with built <u>Built</u>-up, modified bitumen, fully adhered or mechanically attached single-ply through fastened metal panel roof systems, and other types of membrane roof coverings shall also be tested in accordance with FM 4474, UL 580 or UL 1897.

Reason: The first change is purely editorial – the sentence doesn't need to reference "roof systems" twice. Also, this section should not include reference to through fastened metal panel roof systems, since they are covered in Section 1504.3.2.

S11-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1504.3.1-S-MANLEY

S12–12 1504.3.2

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1504.3.2 Metal panel roof systems. Metal panel roof systems through fastened or standing seam shall be tested in accordance with UL 580 or ASTM E 1592.

Exceptions:

- 1. Metal roofs constructed of cold-formed steel, where the roof deck acts as the roof covering and provides both weather protection and support for structural loads, shall be permitted to be designed and tested in accordance with the applicable referenced structural design standard in Section 2210.1.
- 2. Metal roofs constructed of aluminum, where the roof deck acts as the roof covering and provides both weather protection and support for structural loads, shall be permitted to be designed and tested in accordance with the applicable referenced structural design standard in Section 2002.1.

Reason: This code change proposal is intended to permit the use of the Aluminum Association's *Aluminum Design Manual* (ADM1), which is already referenced in Section 2002.1, for the design of wind resistance for aluminum structural panel roof systems in lieu of the test methods prescribed in Section 1504.3.2.

A similar exception for structural metal panels fabricated from cold-formed steel already exists in Section 1504.3.2; it allows the use of AISI S100, "North American Specification for the Design of Cold-formed Steel Structural members."

S12-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1504.3.2-S-GRAHAM

S13–12 1504.3.2

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org) and Lee Shoemaker, Metal Building Manufacturer's Association

Revise as follows:

1504.3.2 Metal panel roof systems. Metal <u>Standing seam metal</u> panel roof systems through fastened or standing seam shall be tested in accordance with UL 580 or ASTM E 1592. <u>Through-fastened panel roof</u> systems shall be tested in accordance with UL 580 or ASTM E1592.

Exception: Metal roofs constructed of cold-formed steel, where the roof deck acts as the roof covering and provides both weather protection and support for structural loads, shall be permitted to be designed and tested in accordance with the applicable referenced structural design standard in Section 2210.1.

Reason: The recommended language provides consistency with the uplift test requirements for standing seam roofs systems as specified in AISI S100, Section D6.2.1. AISI S100 requires that standing seam roofs be tested in accordance with ASTM E1592 to determine panel strength and UL580 is not an optional test for this type of roof system. Panel strengths for through fastened roofs, on the other hand, as specified in AISI S100, can be developed either analytically or through testing in accordance with either UL 580 or ASTM E1592.

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

S13-12				
Public Hearing: Committee: Assembly:	AS ASF	AM AMF	D DF	
-				1504.3.2-S-MANLEY

ICC PUBLIC HEARING ::: April - May 2012

S14–12 1504.4

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

Revise as follows:

1504.4 Ballasted low-slope roof systems. Ballasted lowslope (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. <u>Ballasted roof systems shall be subject to the special</u> inspection requirements of Section 1705.10 to verify conformance to ANSI/SPRI RP-4 standard.

Reason: During the 2005/2006-code change cycle a proposal was submitted to prohibit gravel or stone used as ballast on the roof of a building located in a hurricane-prone regions or on any other building with a mean roof height exceeding prescribed limits based on the building height, exposure category and basic wind speed at the site. These requirements are contained in Section 1504.8. These restrictions were imposed due to damage that occurred reportedly due to wind borne roof aggregate during high wind events. The building height restrictions were imposed due to calculated values.

Prior to this code change proposal the design of ballasted roofs were required to meet ANSI/SPRI RP-4 Wind Design Standard For Ballasted Single-ply Roofing Systems. While this is still a requirement, the code change that occurred due to this proposal now requires that both requirements be met, i.e. the requirements included in the proposal and the requirements of RP-4. This leads to conflicting requirements.

The issue with gravel blow-off that was raised by the NCSEA is that non-code compliant ballasted roof systems are being installed, which is particularly problematic in areas with the potential for high wind events. If these roof systems were installed in accordance with ANSI/SPRI RP-4, then this would not be an issue since this standard is specifically designed to prevent gravel blow-off. This statement is based on the fact that the roof systems that were reported by the NCSEA were investigated and found that they did not conform to the design requirements of the code-referenced standard, ANSI/SPRI RP-4.

To address the issue of gravel blow-off, this code change proposal requires special inspection of ballasted roof assemblies to verify conformance with ANSI/SPRI RP-4 if they are being installed in high wind regions as defined in Section 1705.10 Special inspections for wind resistance.

The ANSI/SPRI RP-4 standard was first included in the building code in 1988. It has demonstrated excellent performance, with no reports of gravel or roof blow-off on systems designed in accordance with the standard. 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, as noted in the NCSEA proposal.

The ANSI/SPRI Ballast Design Guide is based on over 200 wind tunnel tests conducted at the National Research Council of Canada (NRCC). This is the largest commercially available wind tunnel in North America. The tunnel and the experts at the NRCC have used this tunnel to design some of the largest suspension bridges in the world. In addition, over 40 years of field experience and observations from hurricane investigation teams from RICOWI and FEMA have been used in the development of the design criteria.

ANSI/SPRI RP-4 was revised and re-approved in 2008 and is currently being balloted for re-approval. The ballot currently out for re-approval updates the standard to ASCE7-10 requirements. One of the design objectives of ANSI/SPRI RP-4 is to prevent gravel blow-off. The above-mentioned wind tunnel testing 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

The basic approach taken in the ANSI/SPRI RP-4 standard is that as the anticipated wind load on the roof increases due to variables such as design wind speed, building height, exposure category and parapet height, the ballast design requirements get more robust by using larger stone, or substituting pavers for stone, and ultimately not allowing for the use of a ballasted roof system.

The ballast designs contained in the national consensus standard provide restrictions on the use of ballasted single ply roof systems that will allow for the responsible use of aggregate surfacing. There is often the potential for building envelope materials, and many other materials, to become windborne debris in hurricane force wind exposures. In these situations, the approach is to learn how to properly use these materials in high wind areas, not ban their use. The ANSI/SPRI RP-4 standard allows for the continued use of ballasted roofing systems, which are a cost effective method to keep the roof system in place and to improve the energy performance of the building. (Reference the SPRI/DOE/ORNL report on energy effectiveness of ballasted roof systems by going to the following web link, http://www.spri.org/publications/policy.htm under Technical Reports. Select the research report entitled: Evaluating the Energy Performance of Ballasted Roof Systems.

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. The reports can be viewed in the entirety at the same web link provided above for the energy study report. The wind tunnel reports are located at the bottom of that page under Miscellaneous.

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: • Th

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.

Cost Impact: This proposal will increase the cost of construction. The cost increase will be due to the cost of doing a special inspection if the system is being installed in a region described in Section 1705.10 Special inspections for wind resistance.

S14-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1504.4-S-ENNIS

S15–12 1504.5.1 (NEW), Chapter 35 (NEW)

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

Add new text as follows:

1504.5.1 Gutter securement for low-slope roofs. Low-slope (roof slope < 2:12) roof system gutter securement shall be designed and installed for wind loads in accordance with Chapter 16 and tested for resistance in accordance with ANSI/SPRI GD-1, except V_{ult} wind speed shall be determined from Figure 1609A, 1609B, or 1609C as applicable.

Add new standard to Chapter 35 as follows:

SPRI

ANSI/SPRI GD-1-2010 Structural Design Standard for Gutter Systems Use with Low-Slope Roofs

Reason: Currently the IBC contains no requirement that gutters be designed and installed to resist wind and static loads. Studies of the aftermaths of hurricanes revealed a need for better gutter system design. Examples of these observations are shown below. SPRI developed this Standard in response to those studies.

The wind resistance tests contained in this standard measure the resistance of the gutter system to wind forces acting outwardly (away from the building.) and to wind forces acting upwardly tending to lift the gutter off the building. The standard also measures the resistance of the gutter system to static forces of water and ice acting downwardly.

Following are observations of results of gutter failures during high wind events. These observations were made during post hurricane investigations conducted by RICOWI (Roofing Industry Committee on Weather Issues).



Figure 1

Figure 1 is a photo was taken of the gutter/cleat attachment after Hurricane Ike, and is a good example of damage progression. This building, located in Anahuac, TX, experienced wind speeds of 110 mph. The inspection team determined that an overhanging gutter and fractured nailer provided a starting point for peel-back of this multi-ply membrane. The roof membrane peeled away from the insulation layer over most of the roof as shown in Figure 2.



Figure 3 is a photo of a building located in Dickinson, TX after Hurricane Ike. This building experienced wind speeds of 100 mph.



Figure 3

In this case the inspection team determined that a cornering wind caused detachment of the gutter and metal edge, allowing wind to infiltrate and pressurize the roof membrane which led to roll-back of the metal roof membrane, exposing the underlying substrate. Figure 4 is of a building located in Lumberton, MS. This photo was taken after Hurricane Katrina. Estimated wind speed at this

location was 110 to 120 mph.



Figure 4 The inspection team noted that approximately two-thirds of the roof membrane was blown off the roof. Initial failure appears to have occurred at the south roof edge where approximately 25 ft of gutter and edge nailer separated from the structure. A vented 3 ft deep soffit may have contributed to the damage by pressurizing the space between deck and roof assembly. However, the roof assembly may have been pressurized by failure of the south roof edge.

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

S15-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1504.5.1 (NEW)-S-ENNIS

S16–12 1504.7, 1504.7.1 (NEW), 1504.7.2 (NEW), Chapter 35 (NEW)

Proponent: Phillip J. Smith, FM Approvals (Phillip.smith@fmapprovals.com)

Revise as follows:

1504.7 Impact resistance. Impact resistance of roof coverings shall be in accordance with Section 1504.7.1 or 1504.7.2, as applicable.

1504.7.1 Low-slope roofs. 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.

1504.7.2 Asphalt shingles. Asphalt shingles shall meet Class 1, 2, 3 or 4 based on the results of tests conducted in accordance with ANSI/FM 4473.

Add new standard to Chapter 35 as follows:

FΜ

4473-11 Impact Resistance Testing of Rigid Roofing Materials by Impacting with Freezer Ice Balls

Reason: Low sloped roofs (< 2:12) are required to meet specific impact resistance. This change addresses impact resistance of shingles applied in steep slope applications.

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

S16-11					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	•				1504.7.1-S-SMITH(NEW)

S17–12 1504.9 (NEW), Chapter 35 (NEW)

Proponent: Mike Ennis (Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

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.

Add new standard to Chapter 35 as follows:

SPRI

ANSI/SPRI RP14-2010 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 in cooperation with Green Roofs for Healthy Cities 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 Ibs/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 may increase the cost of construction.

S17-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					1504.9 (NEW) #1-S-ENNIS

S18–12 1504.9 (NEW), Chapter 35 (NEW)

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

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. Garden and landscaped roof systems shall be subject to the special inspection requirements of Section 1705.10 to verify conformance to ANSI/SPRI RP-14.

Add new standard to Chapter 35 as follows:

SPRI

ANSI/SPRI RP-14-2010 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 in cooperation with Green Roofs for Healthy Cities 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 Ibs/ft2 of growth media or interlocking trays or 22 Ibs/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.

This proposal includes a requirement for special inspection to verify conformance to the ANSI/SPRI RP14 design standard when the system is installed in a high wind region as described in Section 1705.10.

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 may increase the cost of construction.

S18-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1504.9 (NEW) #2-S-ENNIS

S19–12 1505.1, 1509.7.2

Proponent: Christine Covington, Solar Energy Industries Association

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

Revise as follows:

1505.1 General. Roof assemblies shall be divided into the classes defined below. Class A, B and C roof assemblies and roof coverings required to be listed by this section shall be tested in accordance with ASTM E 108 or UL 790. In addition, *fire-retardant-treated wood* roof coverings shall be tested in accordance with ASTM D 2898. The minimum roof coverings installed on buildings shall comply with Table 1505.1 based on the type of construction of the building.

Exceptions:

- 1. Skylights and sloped glazing that comply with Chapter 24 or Section 2610.
- 2. Rooftop mounted photovoltaic panel systems shall be listed and labeled in accordance with UL 1703 for fire classification. The minimum photovoltaic panel system fire classification listing shall be as required by Table 1505.1 or as otherwise required by this code.

1509.7.2 Fire classification. Rooftop mounted photovoltaic <u>panel</u> systems shall have the same <u>a</u> fire classification as the roof assembly required by Section 1505.

Reason: The current IBC requirement to classify photovoltaic systems consistent with the requirement for roof covering materials does not adequately address fire performance evaluation considerations. Fire testing of rooftop mounted (stand-off, rack-mounted) photovoltaic systems was conducted by the Solar America Board for Codes and Standards in conjunction with Underwriter's Laboratories. Their test results did not confirm that a Class A classified roof combined with a Class A classified photovoltaic module would automatically result in an overall Class A assembly. In some cases, systems would perform better, in many worse. This lack of correlation does not address the overall fire performance concern expressed by ICC members at previous hearings.

The intent of this code change is to control roof surface fire propagation and fire spread from the roof surface to a building's interior.

The UL 1703 Standards Committee has been working on revised roofing classification testing employing a complete system comprised of a representative roof covering combined with the photovoltaic panels/modules being evaluated. This will provide assurance that the roof will be rated as the code intends with the specific panel or module system being used.

For further information on Solar ABC's on-going fire testing, visit <u>http://www.solarabcs.org/current-issues/fire_class_rating.html</u> The revisions to 1509.7.2 direct the user to 1505 where the roof covering and PV panel testing is located. A new second exception is added to 1505.1 to require that the panel is to be evaluated to UL1703, not UL790 or ASTM E108. The exception's second sentence intends that the Class A, B, or C fire classification listed PV panel/module system be consistent with any other fire classification requirement for the roof covering contained within the IBC. In some cases, the code may restrict the roof classification to a higher category than what is required simply based on type of construction.

S19-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1505.1-S-COVINGTON

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

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

Revise as follows:

1505.2 Class A roof assemblies. Class A roof assemblies are those that are effective against severe fire test exposure. Class A roof assemblies and roof coverings shall be *listed* and identified as Class A by an *approved* testing agency. Class A roof assemblies shall be permitted for use in buildings or structures of all types of construction.

Exceptions:

- 1. Class A roof assemblies include those with coverings of brick, masonry or an exposed concrete roof deck.
- Class A roof assemblies also include ferrous or copper shingles or sheets, metal sheets and shingles, clay or concrete roof tile or slate installed on noncombustible decks or ferrous, copper or metal sheets installed without a roof deck on noncombustible framing.
- 3. Class A roof assemblies include minimum 16 oz/sq. ft. (0.0416 kg/m2) copper sheets installed over combustible decks.
- 4. Class A roof assemblies include slate installed over ASTM D226, Type II underlayment over combustible decks.

Reason: In IBC 2009, the Exceptions to Section 1505.2 were amended to require ASTM E 108 or UL 790 fire testing to determine the fire classification of certain roof assemblies, including copper sheets and slate, that had historically been exempted for fire testing. At the time, a lack of adequate fire test data was cited as the reason for this change.

In IBC 2012, Exception 3 was added based upon fire testing that was conducted by the Copper Development Association. The National Roofing Contractor Association and the National Slate Association have conducted fire tests at Underwriters Laboratories, Inc. (UL) that documents slate installed over a specific underlayment (ASTM D226, Type II) over a combustible deck meets the requirements of UL 790 Class A. This testing substantiates the addition of Exception 4 as a Class A roof assembly.

A copy of this test report has been submitted with this code change proposal; additional copies are available by contacting the proponent.

S20-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1505.2-S-GRAHAM

S21-12 1505.8

Proponent: Christine Covington, Solar Energy Industries Association

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

Revise as follows:

1505.8 <u>Building integrated</u> photovoltaic systems. Rooftop installed <u>building integrated</u> photovoltaic systems that are adhered or attached to <u>serve as</u> the roof covering or photovoltaic modules/shingles installed as roof coverings shall be listed and labeled to identify their for fire classification in accordance with the testing required in Section 1505.1.

Reason: This section intends to require flush mounted PV roof coverings or PV integrated roof cladding systems to comply with UL790 or ASTM E108. This is appropriate for these types of systems.

The current language used in this section implies that a stand-off rack mounted panel or module system is also required to be evaluated to UL790 or ASTM E108. These types of stand-off systems have differing fire characteristics that are better evaluated using UL1703 method for fire classification. This is currently required under Section 1509.7.2.

The proposed change will clarify which test is appropriate for BIPV systems used in a roofing application.

S21-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1505.8-S-COVINGTON

S22-12 1505.8 (NEW)

Proponent: David Marsili, City of Las Vegas Fire Rescue, representing self (dmarsili@lasvegasnevada.org)

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

Add new text as follows:

1505.8 Roof identification signs. Identification signs shall be placed at approved locations on the roof of buildings that indicate the roof Fire Classification, Roof Type, Truss Type and the Direction of Construction. The signs are to be a minimum of six inches in height with one inch letters on contrasting background.

Reason: For fire fighter safety during fire emergencies in buildings. Over the past 30 years building construction methods have transformed from heavy timber, larger steel and concrete sections, to smaller, lighter, engineered glue-laminated lumbered beams and trusses. It is now common to find this lightweight steel and engineered wood framing in virtually all structures such as nursing homes, hotels, apartments, schools, daycare centers and strip-malls. With this newer light weight construction of roofs and truss support systems, construction failure and collapse is proven to happen faster under fire conditions than traditional materials, thus it is imperative that any firefighting team assigned to roof ventilation gain this type of construction information of these key elements as soon as possible. This information would be available to fire fighters immediately with small signs placed at proper locations on the roof. This idea was actually called for in a NIOSH released document in April, 2005 called, *"Preventing injuries and death of Fire Fighters due to Truss System Failures."* One jurisdiction in the State of Florida has already locally adopted a similar signage system to alert their Fire Department of the type of construction used on roof and truss system within the building.

In July 2006, the Occupational Safety and Health Administration published a document for building designers, owners and managers entitled *Fire Service Features of Buildings and Fire Protection Systems*. On page 26 of this document it describes the *"Hazards to the Fire Service"* including Light Weight Construction. It states that many firefighters have been killed in collapses attributed to truss failure particularly those made of wood. In Table 1 of the report, testing was conducted on the Structural Members of trusses to the point of failure. One commonly used truss system made of 6 x 1 ³/₄ in. C-joist type construction, completely failed in just 3:45 minutes of their testing.

In May 2008, a document published by Fire Engineering entitled *Structural Collapse under Fire Conditions*, goes into get detail of their testing of floor and roof assemblies. They tested commonly used structural assemblies made of wood under actual load bearing circumstances. On page 2 of this document there is a table called "*Lumber Failure Times*". The table shows all I-joist type construction with typical spacing of 24 inches, failed in 4:40 minutes into the test.

Most fire academies across the country teach fire fighters once they are on the roof on buildings, they have five minutes or less to gain the information necessary to ventilate the structure under fire conditions. This typically is done by cutting many small inspection holes into areas of the roof to determine the type of construction used and the direction of construction of the supporting truss system. This code change will give them this information immediately saving them valuable time to do their jobs and possibly save lives.

OSHA-Fire Service Features is available at www.osha.gov/publications/firefeatures3256.pdf

Fire Engineering – Structural Collapse under Fire Conditions is available at www.fireengineering.com/articles/print/volume-161/issue-5.

NIOSH-Preventing Injuries and death of Fire Fighters due to Truss System Failures is available at www.cdc.gov/niosh/docs/wp-solutions/2005-102/pdfs/2005-102.pdf

Example:

Fire Classification: A, B, C or None.

Roof Type: Asphalt shingle over plywood, rubber membrane/foam over metal,etc...

Truss type: Lightweight wood truss, steel web truss, panelized, truss joist, steel bar truss, engineered I-beam, etc....

Truss Direction: North/South, East/West, Alpha to Bravo, or Charlie to Delta.

(If panelized then state main beam direction)

Cost Impact: The code change will have a very minimal impact on building construction.

S22-12					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
	/ locolinoiy.	, (01	,	51	1505.8 (NEW)-S-MARSILI.doc

S23-12 1505.9 (NEW)

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

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

Add new text as follows:

1505.9 Roof insulation. Roof insulations for Group H-2, H-3, or H-4 occupancies shall comply with the requirements of Class NC (noncombustible core) in accordance with FM 4470.

Reason: This proposal introduces a new type class of non-combustible roof insulation products which are specifically evaluated for a higher level of resistance to ignition based upon testing and conformance with the newest edition (2009) of FM 4470 Approval Standard for Single-Ply, Polymer-Modified Bitumen Sheet,Built-Up Roof (BUR) and Liquid Applied Roof Assemblies. It does not preclude the use of other roof insulation materials. This proposal does not does not preclude the use of other roof insulation materials. It merely recognizes that in order for a roof insulation to be considered non-combustible, it needs to comply with the new FM 4470 standard.

There is a long history of losses connected with fires in roofing materials and roof coverings. According to NFPA statistics, an average of 4,200 fires starting with exterior roof coverings, surfaces or finishes made of sawn wood occurred per year during the five year period from 1994 through 1998. These fires caused an average of five civilian deaths, 23 civilian injuries and an estimated \$7.0 million in direct property damage per year. During this time period, these fires accounted for 0.7% of the 567,100 total reported structure fires, 0.1% of the 3,744 civilian structure fire deaths, 0.1% of the 21,293 civilian structure fire injuries, and 1.1% of the \$7.2 billion in direct property damage. These totals exclude from the analysis fires where the roof covering was recorded as composed of hardboard, plywood, fiberboard or wood pulp, as these products are considered more likely to refer to decking or framing, rather than to shingles and covering. Also excluded are fires where the roof covering was recorded as growing wood, felled but unsawn wood, wood shavings, or unclassified or unknown-type wood. More importantly, this analysis excludes fires that begin with some other fuel but grow and spread primarily through secondary involvement of wooden roof coverings. Such fires cannot be identified in existing national databases.¹

The roof insulation is one of the most vulnerable parts of a building. Group H buildings are designed to address hazards beyond the other occupancies to provide minimum regulations intended to mitigate the risk to life and structures.

¹ Marty Ahrens, NFPA Report, Wood Shingle or Wood Shake Roof Fires, Statistical Analysis, July 2001

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

ANALYSIS: This code change proposal references FM standard 4470, which is already referenced in this code. However, the proposed change to code text is written to correlate with a new edition of the standard 4470-2009, rather than the edition presently referenced in the code, which is the 1993 edition. The update to this standard will be considered by the Administrative Code Committee during the 2013 Code Development Cycle. Should this code change proposal be approved, but the update to the

standard not be approved, the code text will revert to the text as it appears in the 2012 Edition of the Code.

S23-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1505.9 (NEW)-S-CRIMI.doc

S24–12 1505.9 (NEW), Chapter 35 (NEW)

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

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

Add new text as follows:

1505.9 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 VF-1.

Add new standard to Chapter 35 as follows:

SPRI

VF-1-2010 External Fire Design Standard for Vegetative Roofs

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 in conjunction with Green Roofs for Healthy Cities 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 m²), with each section having no dimension greater than 125 ft (39 m) by installing a a minimum of 3ft. (0.9 m) wide, Class A rated assembly barrier zones.

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

S24-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				1505 9-S-ENNIS doc

S25-12 1506.1

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1506.1 Scope. The requirements set forth in this section shall apply to the application of roof-covering materials specified herein. Roof coverings shall be applied in accordance with this chapter and the manufacturer's <u>printed</u> installation instructions. Installation of roof coverings shall comply with the applicable provisions of Section 1507.

Reason: This code change proposal clarifies the intent of the code by specifically stipulating manufacturers' installation instructions need to be in print. Other forms of instructions, such as verbal statements, are not appropriate for code compliance purposes.

S25-12					
Public Hearing: C	Committee:	AS	AM	D	
A	Assembly:	ASF	AMF	DF	
					1506.1-S-GRAHAM

S26-12 1506.2

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Delete without substitution:

1506.2 Compatibility of materials. Roofs and roof coverings shall be of materials that are compatible with each other and with the building or structure to which the materials are applied.

Reason: This code change proposal is intended to facilitate better compliance and easier enforcement of the Code relating to roof coverings.

Specific criteria are not provided in the Code for determining roofing materials' compatibility or incompatibility. Material compatibility is best determined by material manufacturers and should be explained or restricted in manufacturers' installation instructions, which are already provided for in Section 1506.1-Scope.

Deleting this section relieves the building official for making determinations of materials' compatibility or incompatibility without specific criteria.

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

S26-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1506.2-S-GRAHAM

S27-12 1506.3

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1506.3 Material specifications and physical characteristics. Roof-covering materials shall conform to the applicable standards *listed* in this chapter. In the absence of applicable standards or where materials are of questionable suitability, testing by an *approved* agency shall be required by the *building code official* to determine the character, quality and limitations of application of the materials <u>shall be approved</u> by the *building official* in accordance with Section 104.11.

Reason: This code change proposal is intended to clarify the code's intent relating to the use of roofing materials that do not specifically conform to the requirements of this Chapter.

It can be interpreted that Section 1506.3 may conflict somewhat with Section 104.11-Alternative Materials, Design and Methods of Construction and Equipment. The proposal clarifies the Code's language and provides a direct reference to Section 104.11

S27-12				
Public Hearing: Committee	e: AS	AM	D	
Assembly	: ASF	AMF	DF	
•				1506 3-S-GRAHAM

S28-12 1507.10.3 (NEW)

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Add new text as follows:

1507.10.3 Mopping asphalt. Asphalt used in the field application of hot-applied built-up roofs shall comply with ASTM D312 and have a minimum 125°F (69.4°C) temperature differential between the asphalt's equiviscous temperature and its flash point temperature. Asphalt shall not be heated to or above its flash point temperature.

Reason: This code change proposal is intended to add requirements to the Code to provide for the safe and proper installation of hot-applied built-up roofs.

The application of most built-up roofs involves heating asphalt at the jobsite, typically in either an asphalt kettle or asphalt tanker located at ground level, to temperatures in excess of 500 °F (260°C) in order to dispense the asphalt at the point of application (rooftop) at an adequate temperature for proper application. The material standard for roofing asphalt--ASTM D312, which is already referenced in the Code--provides for the testing and labeling of asphalt's maximum heating temperature (flash point temperature) and proper application temperature (equiviscous temperature).

In order to minimize the risks of fires associated with jobsite heating of asphalt, an asphalt should not be heated to its flash point temperature. To allow for the proper application of mopping asphalt, a temperature differential between the asphalt's heating temperature and its equiviscous temperature is necessary to account for the asphalt's cooling during transportation from the heating location (e.g., ground level) and the point of applcaition (rooftop). *The NRCA Roofing Manual* suggests a minimum 125°F (69.4°C) differential between an asphalt's equiviscous temperature and it's flash point temperature for this purpose.

This code change proposal establishes a minimum temperature differential between and asphalt's equiviscous temperature and it's flash point temperature, and stipulates asphalt shall not be heated to or above its flash point temperature.

S28-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1507.10.3 (NEW)-S-GRAHAM

S29–12 1507.2 (NEW), 1507.2.1 (NEW), 1507.2.2 (NEW), 1507.2.3 (NEW), 1507.2.8.1, 1507.3.3.3, 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.9.3.1, Chapter 35

Proponent: T. Eric Stafford, representing Insurance Institute for Business and Home Safety (IBHS)

Revise as follows:

1507.2. Sealed roof decks. When required, a sealed roof deck shall be installed in accordance with Section 1507.2.1, 1507.2.2 or 1507.2.3.

1507.2.1 Self-adhering cap sheet. The entire roof deck shall be covered with a self adhering polymer modified bitumen membrane complying with ASTM D 1970. An approved underlayment for the applicable roof covering shall be applied over the cap sheet, unless the top surface of the membrane provides a bond break between the membrane and the roof covering.

1507.2.2 Self-adhering strips. A minimum 4 inch wide strip of self adhering polymer modified bitumen membrane complying with ASTM D 1970 shall be applied over all joints in the roof decking. An approved underlayment for the applicable roof covering shall be applied.

1507.2.3 Synthetic underlayment. The roof deck shall be covered with a reinforced synthetic roof underlayment approved as an alternate to ASTM D 226 Type I or II. The synthetic underlayment shall have a minimum tear strength of 20 lbs in accordance with ASTM D 1970 or ASTM D 4533. This underlayment shall be attached using annular ring or deformed shank roofing fasteners with minimum 1 inch diameter caps at 6 inches on center spacing along all laps and at 12" on center in the field or a more stringent fastener schedule if required by the manufacturer for high wind installations. Metal caps are required for areas where the V_{asd} , in accordance with Section 1609.3.1, equals or exceeds 110 mph. Side laps shall be a minimum of 2 inches and end laps shall be a minimum of 6 inches. All seams shall be sealed with a compatible adhesive or a compatible 4 inch wide tape. For roofs with slopes of 45 degrees and higher, seams are not required to be sealed provided laps are a minimum of 18 inches. No additional underlayment is required.

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] 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.

Underlayment installed where V_{asd} , in accordance with Section 1609.3.1, equals or exceeds 120 mph (54m/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. Underlayment shall be applied in accordance with Section 1507.2.8 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 a sealed roof deck installed in accordance with Section 1507.2 shall be permitted.

1507.3.3.3 High wind attachment. Underlayment applied in areas subject to high wind [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-

resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with Sections 1507.3.3.1 and 1507.3.3.2 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 a sealed roof deck installed in accordance with Section 1507.2 shall be permitted.

1507.4.5 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 a sealed roof deck installed in accordance with Section 1507.2 shall be permitted.

1507.5.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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 spacing (152 mm) at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 a sealed roof deck installed in accordance with Section 1507.2 shall be permitted.

1507.6.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32- gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 a sealed roof deck installed in accordance with Section 1507.2 shall be permitted.

1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of $_{3/4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 a sealed roof deck installed in accordance with Section 1507.2 shall be permitted.

1507.8.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 a sealed roof deck installed in accordance with Section 1507.2 shall be permitted.

1507.9.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ³/₄ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 a sealed rood deck installed in accordance with Section 1507.2 shall be permitted.

Add new standard to Chapter 35 as follows:

ASTM

D 4533-11 Standard Test Method for Trapezoid Tearing Strength of Geotextiles

Reason: This code change proposal simply seeks to expand and provide additional specification for using self-adhering polymer modified bitumen membrane to prevent water intrusion. The commonly used term "secondary water barrier" is no longer used, since some have argued that underlayment itself is a secondary water barrier. Secondary water barrier has been replaced by the term "sealed roof deck." Regardless of the terminology, the purpose of these provisions is provide an additional level of protection to the roof decking in the event that the primary roof covering is blown off due to high winds. It's important to note that this code change proposal does not require a sealed roof deck. Rather, it provides specific criteria for creating a sealed roof deck as an alternative to the requirements for underlayment in high winds (e.g., Section 1507.2.8.1). While providing specific installation criteria for creating a sealed roof deck. The bitumen membrane, this code change proposal also incorporates the use of reinforced synthetic underlayment for creating a sealed roof deck.

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

S29-12				
Public Hearing: Committee: Assembly:	AS ASF	AM AMF	D DF	
				1507.2 (NEW)-S-STAFFORD

S30–12 1507.2.1

Proponent: Eli P. Howard III, Sheet Metal and Air-Conditioning Contractors' National Association (SMACNA) (ehoward@smacna.org)

Revise as follows:

1507.2.1 Deck requirements. Asphalt shingles shall be fastened to solidly sheathed decks. <u>Installer is</u> required to remove the cover strip protecting adhering tabs.

Reason: Often roofing contractor employees install asphalt shingles without removing the protective strip from underneath that separates shingles and the adhesive from other shingles during storage and shipment. As the code is currently written, this is not a requirement. This code change simply adds this commonsense step into the code and provides the code inspector the ability to enforce the practice.

S30-12				
Public Hearing: Comr	nittee: AS	AM	D	
Asser	nbly: ASF	AMF	DF	
	2			1507.2.1-S-HOWARD

S31–12 1507.2.6.1 (NEW), 1507.2.8.1, 1507.3.3.3, 1507.3.6.1 (NEW), 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.9.3.1

Proponent: T. Eric Stafford, representing Insurance Institute for Business and Home Safety (IBHS)

Revise as follows:

1507.2.6.1 Fasteners and high winds. In areas where the ultimate design wind speed, V_{ult} equals or exceeds 130 mph, fasteners for asphalt shingles shall be annular ring shank nails having not less than 20 rings per inch in addition to the requirements of Section 1507.2.6.

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds $[V_{asd} \text{ greater}]$ than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] $[V_{ult} \text{ equals to or greater than}]$ 130 mph, shall be applied with corrosion-resistant fasteners complying with Section 1507.2.6.1 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.

Underlayment installed where $V_{asd,}$ in accordance with Section 1609.3.1, the ultimate design wind speed, V_{ult} equals or exceeds 120 140 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. Underlayment shall be applied in accordance with Section 1507.2.8 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall comply with Section 1507.2.6.1 and shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

1507.3.3 High wind attachment. Underlayment applied in areas subject to high wind $[V_{asd}$ -greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] $[V_{ult}$ equal or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion-resistant fasteners in accordance with the manufacturer's installation instructions annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where V_{asd} , in accordance with Section 1609.3.1, the ultimate design wind speed, V_{ult} equals or exceeds 120 140 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. Underlayment shall be applied in accordance with Sections 1507.3.3.1 and 1507.3.3.2 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

1507.3.6.1 Fasteners and high winds. In areas where the ultimate design wind speed, V_{ult} equals or exceeds 130 mph, fasteners for tile shall be a minimum 11 gage [0.105 inch (2.67 mm)] annular ring shank nails having not less than 20 rings per inch shank, with a minimum 5/16 inch-diameter (9.5 mm)

head, of a length to penetrate through the roofing sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

1507.4.5 Underlayment and high wind. Underlayment applied in areas subject to high winds $[V_{asd}]$ greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] $[V_{ult}]$ equal to or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where $V_{asd\tau}$ in accordance with Section 1609.3.1, the ultimate design wind speed. V_{ult} equals or exceeds 120 140 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

1507.5.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] [V_{ult} equal to or greater than 130 mph shall be applied with minimum 12 page [0.105 inch (2.67 mm) corrosion-resistant fasteners in accordance with the manufacturer's installation instructions annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where V_{asd} , in accordance with Section 1609.3.1 the ultimate design wind speed, V_{ult} , 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 spacing (152 mm) at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

1507.6.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] [V_{ult} equal to or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion resistant fasteners in accordance with the manufacturer's installation instructions. annular ring shank nails having not less than 20 rings per inch, with a minimum 3./8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of $\frac{34}{4}$ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where V_{asd} , in accordance with Section 1609.3.1, the ultimate design winds peed, V_{utt} equals or exceeds $\frac{120}{140}$ 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] [V_{ult} equal to or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion resistant fasteners in accordance with the manufacturer's installation instructions. annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where $V_{asd,}$ in accordance with Section 1609.3.1, the ultimate design wind speed, V_{ult} equals or exceeds 120 140 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

1507.8.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] [V_{ult} equal to or greater than 130 mph shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion resistant fasteners in accordance with the manufacturer's installation instructions. annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where V_{asd} , in accordance with Section 1609.3.1, the ultimate design wind speed, V_{ult} equals or exceeds 120 140 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

1507.9.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds $[V_{asd}]$ greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] $[V_{ult}]$ equal to or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion resistant fasteners in accordance with the manufacturer's installation instructions. annular ring shank nails having

not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of ³/₄ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where V_{asd} , in accordance with Section 1609.3.1, the ultimate design wind speed, V_{ult} equals or exceeds 120 140 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

Reason: Water intrusion continues to be an issue with hurricanes and high wind events. Significant improvements have been made recently to the codes and other voluntary methods that help prevent water intrusion through the roof decking when the primary roof covering has been blown off or damaged. These include the underlayment and high wind requirements in the 2012 IBC and the 2012 IRC in addition to the Sealed Roof Deck provisions recommended by the IBHS Fortified program and FEMA hurricane retrofit program guidance. However, recent tests on sealed roof decks at the IBHS Research Center indicate that water intrusion through nail holes left in the roof decking when the primary roof covering has been lost is still an issue. In the areas specified, this code change proposal requires the roof underlayment to be attached with ring shank nails. Where nails are specificantly higher withdrawal capacity to similar sized smooth shank nails (up to 131% higher). The use of ring shank nails will help keep the nails in place when the roof covering is blow off and reduce the chance that unfilled nail holes will allow water intrusion.

This code change proposal also changes the wind speed trigger for when the improved underlayment and fastening methods are required. The wind speed is changed to a V_{utt} value consistent with the wind speeds represented in Figures 1609A, 1609B, and 1609C. Additionally, the wind speed threshold that triggers the improved underlayment and fastening methods has been slightly reduced. The proposed 130 mph and 140 mph V_{utt} wind speed triggers are more comparable geographically to the 110 mph and 120 mph wind speeds in the 2009 IBC. The triggers are also consistent with the wind speed limitations on conventional construction and the prescriptive non-high wind provisions of the 2012 IRC (The Wind Design Required Region in the 2012 IRC is tied to the 130 mph V_{utt} wind speed). Post-storm investigations also show that water intrusion is an issue in inland areas when the primary roof covering has been blown off.

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

S31-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1507.2.6.1 (NEW)-S-STAFFORD

S32–12 1507.2.7.1, Table 1507.2.7.1(1), Table 1507.2.7.1(2),

Proponent: Michael D. Fischer, Kellen Company, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

Revise as follows:

1507.2.7.1 Wind resistance. Asphalt shingles shall be tested in accordance with ASTM D 7158. Asphalt shingles shall meet the classification requirements of Table 1507.2.7.1(1) <u>1507.2.7.1</u> for the appropriate maximum basic wind speed. Asphalt shingle packaging shall bear a label to indicate compliance with ASTM D 7158 and the required classification in Table 1507.2.7.1(1).<u>1507.2.7.1</u>.

Exception: Asphalt shingles not included in the scope of ASTM D 7158 shall be tested and labeled to indicate compliance with ASTM D 3161 and the required classification in Table 1507.2.7.1(2). <u>1507.2.7.1</u>.

TABLE 1507.2.7.1(1) CLASSIFICATION OF ASPHALT ROOF SHINGLES PER ASTM D 7158*

NOMINAL DESIGN WIND SPEED, V _{asd} ^b (mph)	CLASSIFICATION REQUIREMENT
85	D, G or H
90	D, G or H
100	G or H
110	G or H
120	G or H
130	H
<u>140</u>	H
150	H

For SI: 1 foot = 304.8 mm; 1 mph = 0.447 m/s.

a. The standard calculations contained in ASTM D 7158 assume exposure category B or C and building height of 60 feet or less. Additional calculations are required for conditions outside of these assumptions.

b. V_{acd} shall be determined in accordance with Section 1609.3

TABLE 1507.2.7.1(2)

CLASSIFICATION OF ASPHALT SHINGLES PER ASTM D 3161

NOMINAL DESIGN WIND SPEED, V _{asd} ^a (mph)	CLASSIFICATION REQUIREMENT
85	A, D or F
90	A, D or F
100	A, D or F
110	F
120	F
130	F
140	F
150	F

For SI: 1 mph = 0.447 m/s.

a. V_{asd} shall be determined in accordance with Section 1609.3.1.

TABLE 1507.2.7.1 CLASSIFICATION OF ASPHALT SHINGLES

<u>Maximum Basic</u> <u>Wind Speed, V_{ult}</u> <u>from Figure 1609A,</u> <u>B, C or ASCE-7</u>	<u>Maximum Basic Wind</u> <u>Speed, V_{asd} from Table</u> <u>1609.3.1</u>	ASTM D 7158 ^ª Shingle Classification	ASTM D 3161 Shingle Classification
<u>110</u>	<u>85</u>	<u>D, G or H</u>	<u>A, D or F</u>
<u>116</u>	<u>90</u>	<u>D, G or H</u>	<u>A, D or F</u>
<u>129</u>	<u>100</u>	<u>G or H</u>	<u>A, D or F</u>
<u>142</u>	<u>110</u>	<u>G or H</u>	<u> </u>
155	<u>120</u>	<u>G or H</u>	<u> </u>
<u>168</u>	<u>130</u>	<u>H</u>	<u> </u>
<u>181</u>	<u>140</u>	<u>H</u>	<u> </u>
194	150	Н	F

For SI: 1 foot = 304.8 mm; 1 mph = 0.447 m/s.

a. The standard calculations contained in ASTM D 7158 assume exposure category B or C and building height of 60 feet or less. Additional calculations are required for conditions outside of these assumptions.

Reason: With the introduction of the updated ASCE-7 into the IBC, there is a disconnect between the referenced standards for the wind resistance of asphalt shingles and the revised wind speed maps in the code. The proposal is based on revisions to the Florida Building Code and will provide for a simpler process for code officials to verify the appropriate selection of asphalt shingles. This is necessary to eliminate confusion in the marketplace caused by the change in how wind speeds are characterized.

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

S32-12 Public Hearing: Committee: AS AM D Assembly: ASF AMF DF 1507.2.7.1-S-FISCHER

S33–12 1507.2.8.2

Proponent: Bill McHugh, Chicago Roofing Contractors Association (bill@crca.org)

Revise as follows:

1507.2.8.2 Ice barrier. In areas where there has been a history of ice forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self adhering polymer modified bitumen sheet shall be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the *exterior wall* line of the building.

Exceptions:

- <u>1.</u> Detached accessory structures that contain no conditioned floor area.
- Roofs with slope equal to or greater than 8/12, the ice barrier shall be applied to a point 36 inches (914 mm) past the outside part of the inside wall line of the building up the slope of the roof deck.

Reason: The Chicago Roofing Contractors Association (CRCA) and other steep slope roofing contractors work in all climates from hot summer to the dead of cold, snowy winters. We have enough snow most years to get much experience in ice dam situations.

In steep slope applications in climates where ice forms at the eave edge of roofs. Ice melts due to heat from below melting snow, then freezes where the water meets roof surfaces that are over unheated areas, making a buildup of ice. This buildup becomes a 'dam' that backs water up under the roof covering and underlayment leaking into the building.

The purpose of this proposal is to bring to the Code into alignment with the practical application of the ice barrier underlayment products in the field. Since gravity stops water from backing up very far on super steep slopes greater than 8" in 12" there needs to be a limit to the amount of ice barrier underlayment applied.

On very steep sloped roofs, the ice dams will still occur. However, buildup of ice cannot build far beyond the ball that forms at the gutter edge on slopes greater than 8" in 12". Secondly, the water will not defy gravity and move very far upward, when the physics of the application are that the water will drip over the dam due to gravity first.

The way the current code is written, ice barrier material may be needed on the complete roof deck rather than to protect just the eave edges and 3' up slope. Through clarifying this requirement with the exception, the intent of the code is met while reducing costs to builders and building owners and managers.

S33-12					
Public Hearing: Co	ommittee:	AS	AM	D	
As	sembly:	ASF	AMF	DF	
	,				1507.2.8.2 #1-S-MCHUGH
S34–12 1507.2.8.2

Proponent: Bill McHugh, Chicago Roofing Contractors Association (bill@crca.org)

Revise as follows:

1507.2.8.2 Ice barrier. In areas where there has been a history of ice forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self adhering polymer modified bitumen sheet shall be used in lieu of normal underlayment and extend <u>2 inches (51 mm) down the fascia and under the drip edge</u>, from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the *exterior wall* line of the building.

Exceptions:

- 1. Detached accessory structures that contain no conditioned floor area.
- 2. Roof recover applications where no new metal drip edges or gutters are incorporated.

Reason: The Chicago Roofing Contractors Association (CRCA) and other steep slope roofing contractors work in all climates from hot summer to the dead of cold, snowy winters. We have enough snow most years to get much experience in ice dam situations. In steep slope applications in climates where ice forms at the eave edge of roofs. Ice melts due to heat from below melting snow, then freezes where the water meets roof surfaces that are over unheated areas, making a buildup of ice. This buildup

becomes a 'dam' that backs water up under the underlayment and roof covering. Studies show that roof recover applications typically fail at flashings on all roof slopes. The roof edge flashings are most susceptible to leaks from water backing up under the underlayment and roof covering because it freezes at the eave edge first driving water up-slope.

According to CRCA roofing contractors, if the code required ice barrier is applied improperly to the top of the metal drip edge, the water will leak into the structure. The leak(s) may be difficult to detect in the concealed space location.

In new construction, tear off and roof replacement situations the roofing underlayment construction is easily phased to be installed before the drip edges at the eave edge.

In roof recover applications where metal is not removed, surfaces may be dirty, uneven, and very difficult even for the best contractors to provide a water tight seal.

To provide the building owner the best application and give the code requirement the best chance at working as intended, this proposal from the Chicago Roofing Contractors Association is presented.

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

S34-12

••••				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1507.2.8.2 #2-S-MCHUGH

S35-12 1507.2.8.2

Proponent: Bill McHugh, Chicago Roofing Contractors Association (bill@crca.org)

Revise as follows:

1507.2.8.2 Ice barrier. In areas where there has been a history of ice forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self adhering polymer modified bitumen sheet shall be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the *exterior wall* line of the building.

Exception: Detached accessory structures that contain no conditioned floor area.

Reason: In a survey of CRCA Steep & Shingle Committee Members it appears this method for ice barrier protection is no longer used due to labor intensive and messy application.

At the time the ice barrier materials were introduced to the code, this was an application used because the ice barrier materials were not in the code. After years of use, it seems the two layers of underlayment cemented together method is not used as it is much more costly than the self adhering polymer modified bitumen sheet materials.

Therefore, we propose to remove this option from the code.

S35-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1507.2.8.2 #3-S-MCHUGH

S36-12 1507.2.9.3

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB) (gehrlich@nahb.org)

Revise as follows:

1507.2.9.3 Drip edge. Provide <u>A</u> drip edge <u>shall be provided</u> at eaves and <u>gables</u> <u>rake edges</u> of shingle roofs. Overlap to be <u>Adjacent segments of drip edge shall be lapped</u> a minimum of 2 inches (51 mm). <u>Eave The vertical leg of</u> drip edges shall <u>be a minimum of 1-1/2 inches (38 mm) in width</u>, extend <u>a</u> <u>minimum of</u> 1/4 inch (6.4 mm) below sheathing and <u>and have a minimum clearance of 3/8" (9.5 mm)</u> from the face of the structure. The drip edge shall extend back on the roof a minimum of 2 inches (51 mm). Underlayment shall be installed over drip edges along eaves. Drip edges shall be installed over <u>underlayment along rake edges</u>. Drip edges shall be mechanically fastened a maximum of 12 inches (305 mm) o.c. <u>Unless specified differently by the shingle manufacturer</u>, shingles are permitted to be flush with the drip edge.

Reason: The purpose of this code change is to revise the IBC drip edge language. The current language is not in proper code format (instructive rather than mandatory) and omits a number of important details necessary for drip edges to function. Notably, the placement of the drip edge relative to the underlayment along eaves and rake edges is critical and differs for each location. Along eaves, the underlayment should be installed on top of the drip edge so that moisture migrating down the roof passes over both the underlayment and drip edge and into the gutter. Along rake edges, the drip edge should be installed over the underlayment to prevent wind-blown moisture from getting below the underlayment. Most of these changes correlate with the language approved last cycle in Section R905.2.8.5 of the IRC. The one provision not appearing in the IRC is the minimum 3/8" clearance from the face of structure. This requirement appears in ICC 600 Section 502.4.2 and gives additional protection to the fascia board or other facing materials overlapped by the vertical leg of the drip edge.

S36-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
· · · · ·				1507.2.9.3-S-EHRLICH

S37–12 1507.2.8.1, Table 1507.2.8.1 (NEW), 1507.3.3.3, 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.9.3.1

Proponent: John Kurtz, International Staple, Nail & Tool Association (isanta@ameritech.net)

Revise as follows:

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] 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.

Underlayment installed where V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with Section 1507.2.8 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exceptions:

- 1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
- 2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

	Maximum center-to-center spacing of alternate fasteners and grid				
	lines if required center-to-center sp	pacing of code fastener is			
Alternate Fastener ^a	<u>6" (152 mm) o.c.</u>	<u>12" (305 mm) o.c.</u>			
5/8" leg, 21 gage staple	<u>3" (76 mm)</u>	<u>6" (152 mm)</u>			
21 gage staple	<u>3" (76 mm)</u>	<u>7" (178 mm)</u>			
20 gage staple	<u>4" (102 mm)</u>	<u>8" (203 mm)</u>			
0.080083 diam. nail	<u>4" (102 mm)</u>	<u>9" (229 mm)</u>			
0.090 diam. Nail	<u>5" (127 mm)</u>	<u>10" (254 mm)</u>			
<u>18 gage staple</u>					
0.105 diam. Nail (12 gage)	<u>6" (152 mm)</u>	<u>12" (305 mm)</u>			
<u>17 gage staple</u>					
0.120 diam. nail (11 gage)					

TABLE 1507.2.8.1 ROOF COVERING UNDERLAYMENT ATTACHMENT

a. Minimum nail shank length or staple leg length is 3/4" (19 mm) unless otherwise stated.

1507.3.3.3 High wind attachment. Underlayment applied in areas subject to high wind [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with Sections 1507.3.3.1 and 1507.3.3.2 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached

using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exceptions:

- 1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
- 2. <u>As an alternative, cap nails and cap staples complying with requirements of ASTM F1667</u> and fastened in accordance with Table 1507.2.8.1 shall be permitted.

1507.4.5 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exceptions:

- 1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
- 2. <u>As an alternative, cap nails and cap staples complying with requirements of ASTM F1667</u> and fastened in accordance with Table 1507.2.8.1 shall be permitted.

1507.5.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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 spacing (152 mm) at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exceptions:

- <u>1.</u> As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
- 2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

1507.6.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with

corrosion-resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32- gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exceptions:

- 1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
- 2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exceptions:

- 1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
- 2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

1507.8.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exceptions:

- 1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
- 2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

1507.9.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the 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 V_{asd} in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exceptions:

- 1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
- 2. <u>As an alternative, cap nails and cap staples complying with requirements of ASTM F1667</u> and fastened in accordance with Table 1507.2.8.1 shall be permitted.

Reason: The fastener listed for attachment of roof covering underlayment in high-wind areas does not reflect commercially available fasteners successfully used in roofing material application. The code presently lists only one nail shank diameter, 0.105". This proposal addresses both commercially available hand-driven and power-driven cap-fasteners.

Tighter spacing of fasteners specified in the proposed table ensures that spacing of fasteners with diameters not currently specified in the Code would achieve equal (or greater) withdrawal strength than the currently listed nail diameter. Sufficient fastener withdrawal ensures that fastener shanks remain in roof deck while cap transfers uplift forces to the deck. This is a conservative approach because developing data indicates that the relevant failure mode is cap pulling through underlayment, rather than fastener shank withdrawal.

ASTM F1667-11a controls fastener nominal dimensions and tolerances as well as relevant fastener features. Structure of proposal minimizes complexity of code requirements. An "Exception" is added to each roof covering's section. One table presents fastener spacing for all roof coverings.

Cost Impact: The code change proposal will not increase the cost of construction. The numerous options would allow contractors to select options which provide equivalent protection with minimized material and labor costs.

S37-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				T1507 2 8 1(NEW)-S-KURTZ doc

S38–12 1507.4.4

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

1507.4.4 Attachment. Metal roof panels shall be secured to the supports in accordance with the *approved* manufacturer's fasteners. In the absence of manufacturer recommendations, the following fasteners shall be used:

- 1. Galvanized fasteners shall be used for steel roofs.
- 2. Copper, brass, bronze, copper alloy or 300 series stainless-steel fasteners shall be used for copper roofs.
- 3. Stainless-steel fasteners are acceptable for all types of metal roofs.
- 4. Aluminum fasteners are acceptable for aluminum roofs attached to aluminum supports.

Reason: New language provides acceptable construction methods for aluminum-only roof systems.

S38-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				1507.4.4-S-MANLEY

S39–12 Table 1507.4.3(1)

Proponent: Eli P. Howard III, Sheet Metal and Air-Conditioning Contractors' Association (SMACNA) (ehoward@smacna.org)

Revise as follows:

ROOF COVERING TYPE	STANDARD APPLICATION RATE/THICKNESS
Aluminum	ASTM B 209, 0.024 inch minimum thickness for roll-formed panels and 0.019 inch minimum thickness for press-formed shingles.
Aluminum-zinc alloy coated steel	ASTM A 792 AZ 50, 0.024 inch minimum thickness for roll-formed panels.
Cold-rolled copper	ASTM B 370 minimum 16 oz./sq. ft. and 12 oz./sq. ft. high yield copper for metal-sheet roof covering systems: 12 oz./sq. ft. for preformed metal shingle systems.
Copper	16 oz./sq. ft. for metal-sheet roof-covering systems; 12 oz./sq. ft. for preformed metal shingle systems.
Galvanized steel	ASTM A 653 G-90 zinc-coated ^a , 0.024 inch minimum thickness for roll-formed panels.
Hard lead	2 lbs./sq. ft.
Lead-coated copper	ASTM B 101 <u>16 oz/sq. ft minimum thickness for roll-formed panels</u>
Prepainted steel	ASTM A 755, 0.024 inch minimum thickness for roll-formed panels.
Soft lead	3 lbs./sq. ft.
Stainless steel	ASTM A 240, 300 Series Alloys, 0.015 inch minimum thickness for roll-formed panels.
Steel	ASTM A 924, 0.024 inch minimum thickness for roll-formed panels.
Terne and terne-coated stainless	Terne coating of 40 lbs. per double base box, field painted where applicable in accordance with manufacturer's installation instructions.
Zinc	0.027 inch minimum thickness; 99.995% electrolytic high grade zinc with alloy additives of copper (0.08% - 0.20%), titanium (0.07% - 0.12%) and aluminum (0.015%).

TABLE 1507.4.3(1) METAL ROOF COVERINGS

For SI: 1 ounce per square foot = 0.0026 kg/m2,

1 pound per square foot = 4.882 kg/m2,

1 inch = 25.4 mm, 1 pound = 0.454 kg.

a. For Group U buildings, the minimum coating thickness for ASTM A 653 galvanized steel roofing shall be G-60.

Reason: There are no required material thicknesses for roof panels of six listed materials while others do have a required thickness. This will place a minimum required thickness for all metal roof panels. These thicknesses are taken from Table 6-1 of SMACNA's Architectural Sheet Metal Manual which has been referenced by thousands of architects on millions of buildings.

Cost Impact: Indeterminate since no requirement is currently present.

S39-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				T1507.4.3(1)-S-HOWARD.doc

S40–12 1507.7.3

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1507.7.3 Underlayment. Underlayment shall comply with ASTM D 226, Type I II or ASTM D 4869, Type III or Type IV.

Reason: This code change proposal is intended to update the Code's minimum requirement for underlayment used with slate roof systems.

Both *The NRCA Roofing Manual* and the National Slate Association's *Slate Roofs Design and Installation Manual* recommend a minimum No. 30 underlayment be used for slate roof systems. A No. 30 designation is consistent with underlayment products designated ASTM D226, Type II or ASTM D4869, Type III or Type IV. Use of these Type classes in the Code is necessary to differentiate to the products from lighter-weight No. 15 underlayment products.

S40-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1507.7.3-S-GRAHAM

S41–12 1507.8, Table 1507.8, 1507.9

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1507.8 Wood shingles. The installation of wood shingles shall comply with the provisions of Sections and Table 1507.8 1507.8.1 and 1507.8.2.

TABLE 1507.8 WOOD SHINGLE AND SHAKE INSTALLATION

1507.9 Wood shakes. The installation of wood shakes shall comply with the provisions of Sections and Table 1507.8 1507.8.1 and 1507.8.2

Reason: This code change proposal is intended to rectify conflicts that have resulted in the Code regarding wood shingle and wood shake roof systems.

In the final stages of development of the IBC, Table 1507.8-Wood Shingle and Shake Installation was added as a summary of the installation specific requirements of Section 1507.8-Wood Shingles and Section 1507.9-Wood Shakes. With the IBC's 2000 Edition, the requirements in Table 1507.8 matched those of Section 1507.8 and Section 1507.9.

With the publication of IBC's 2003, 2006, 2009 and 2012 editions, changes have been made to the requirements in Section 1507.8 and Section 1507.9; however, these same changes have not been consistently made to Table 1507.8. For example, in IBC 2012 new requirements for underlayment in high wind regions were added in Section 1507.8.3.1 and Section 1507.9.3.1; these requirements were not added to Table 1507.8. There are a number of other similar examples. As a result, the requirements of Table 1507.8 are inconsistent and at times in conflict with those of Section 1507.8 and Section 1507.9.

Deletion of Table 1507.8 and the pointers of the table in Section 1507.8 and Section 1507.9 eliminates the inconsistency and conflicts.

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

S41-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1507.8-S-GRAHAM

S42–12 1507.12.1, 1507.13.1

Proponent: David R. Scott, AIA, Target Corporation (david.scott@target.com)

Revise as follows:

1507.12.1 Slope. Thermoset single-ply membrane roofs shall have a design slope of a minimum of one-fourth unit vertical in 12 units horizontal (2-percent slope) for drainage.

Exception: Thermoset single-ply membrane roofs designed for water accumulation in accordance with Section 1611.2 shall have a design slope of a minimum of one-eighths unit vertical in 12 units horizontal (1-percent slope).

1507.13.1 Slope. Thermoplastic single-ply membrane roofs shall have a design slope of a minimum of one fourth unit vertical in 12 units horizontal (2-percent slope).

Exception: Thermoplastic single-ply membrane roofs designed for water accumulation in accordance with Section 1611.2 shall have a design slope of a minimum of one-eighths unit vertical in 12 units horizontal (1-percent slope).

Reason: The designer should have the option if designing for ponding instability per 1611.2, to use a lower roof slope such as 1/8" per foot. In addition, the IPC allows the option for pipe to slope at 1/8" per foot.

S42-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1507.12.1-S-SCOTT

S43–12 1507.12.3, 1507.13.3, Chapter 35 (NEW)

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1507.12.3 Ballasted thermoset low-slope roofs. Ballasted thermoset low-slope roofs (roof slope < 2:12) shall be installed in accordance with this section and Section 1504.4. Stone used as ballast shall comply with ASTM D 448 or ASTM D 7655.

1507.13.3 Ballasted thermoplastic low-slope roofs. Ballasted thermoplastic low-slope roofs (roof slope < 2:12) shall be installed in accordance with this section and Section 1504.4. Stone used as ballast shall comply with ASTM D448 or ASTM D 7655.

Add new standard to Chapter 35 as follows:

ASTM

D 7655-12 Standard Classification for Size of Aggregate Used as Ballast for Roof Membrane Systems

Reason: This code change proposal is intended to add a new recognized standard to the Code for the size classification of aggregate used as ballast for membrane roof systems.

ASTM D 7655, "Standard Classification for Size of Aggregate Used as Ballast for Membrane Roof Systems," has just been published in 2012 and provides a method for the definition of sizes of aggregate used as ballast for membrane roof systems.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S43-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1507.12.3-S-GRAHAM

S44–12 1507.16, Chapter 35 (NEW)

Proponent: Robert J. Davidson, Davidson Code Concepts, LLC, representing Single Ply Roofing Industry (SPRI) (rjd@davidsoncodeconcepts.com)

Revise as follows:

1507.16 Roof gardens and landscaped roofs. Roof gardens and landscaped roofs shall comply with the requirements of this chapter and Sections 1607.12.3 and 1607.12.3.1 and the *International Fire Code* and ANSI/SPRI VF-1.

Add new standard to Chapter 35 as follows:

ANSI

ANSI/SPRI VF-1-2010 External Fire Design Standard for Vegetative Roofs

Reason: In developing the "Roof gardens and landscaped roofs" requirements that were placed within the International Fire Code, the standard "ANSI/SPRI VF-1" was utilized for the technical installation requirements.

There are approximately 19 states that adopt a fire code other than the IFC and in the process for some of those states linkage is lost to the specific requirements in the IFC that are meant to complement and add to the IBC language, in this case for the rooftop gardens and landscaped roofs. This proposal recommends that a reference to ANSI/SPRI VF-1 be added to the IBC Section 1507.16. This reference would complement the language in the IFC and ensure that the same requirements are available to the code official and apply regardless of what fire code is adopted in a given jurisdiction.

S44-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1507.16-S-DAVIDSON

S45–12 1507.17.1, Chapter 35 (NEW)

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1507.17.1 Material standards. Photovoltaic modules/shingles shall be listed and labeled in accordance with <u>ICC-ES AC365 and</u> UL 1703.

Add new standard to Chapter 35 as follows:

ICC ES ICC Evaluation Service 5360 Workman Mill Rd Whittier, California 90601

AC365-Acceptance Criteria for Building-Integrated Photovoltaic (BIPV) Roof Covering Systems

Reason: This code change proposal is intended to add specific product performance requirements to the IBC for photovoltaic shingles.

UL 1703, "Flat-Plate Photovoltaic Modules and Panels," that is currently referenced in the IBC, addresses the construction, performance (e.g., voltage, current and power requirements test), and wind and fire classification of flat-plate photovoltaic modules. Performance attributes for the products performance as a roof covering (e.g., water-shedding capacity, durability) are not addressed in UL 1703.

The addition of ICC ES AC 365 adds specific product roof covering performance requirements—such as wind-driven rain and product durability—for photovoltaic shingles that are critical for assessing product's long-term performance as a roof covering component.

Currently, a roofing product standard (e.g., ASTM) for photovoltaic shingles does not exist in the industry, making the use of the ICC EC acceptance criteria necessary.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S45-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1507.17.1-S-GRAHAM

S47–12 1507.17.1 (NEW), 1507.17.2 (NEW), 1507.17.3 (NEW), 1507.17.4 (NEW), 1507.17.4.1 (NEW), 1507.17.4.2 (NEW), 1507.17.5 (NEW)

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Add new text as follows:

1507.17.1 Deck requirements. Photovoltaic shingles shall be applied to a solid or closely fitted deck, except where the roof covering is specifically designed to be applied over spaced sheathing.

1507.17.2 Deck slope. Photovoltaic shingles shall not be installed on roof slopes less than three units vertical in 12 units horizontal (25-percent slope).

1507.17.3 Underlayment. Unless otherwise noted, required underlayment shall conform to ASTM D 226, ASTM D 4869, or ASTM D 6757.

1507.17.4 Underlayment application. Underlayment shall be applied shingle fashion, parallel to and starting from the eave, lapped 2 inches (51 mm) and fastened sufficiently to hold in place.

1507.17.4.1 High wind attachment. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's instructions. Fasteners shall be applied along the overlap at a maximum spacing of 36 inches (914 mm) on center. Underlayment installed where V_{asd} , in accordance with Section 1609.3.1, 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. Underlayment shall be applied in accordance with Section 1507.2.8 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of not less than 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

1507.17.4.2 Ice barrier. In areas where there has been a history of ice forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self adhering polymer modified bitumen sheet shall be used instead of normal underlayment and extend from the lowest edges of all roof surfaces to a point not less than 24 inches (610 mm) inside the *exterior wall* line of the building.

Exception: Detached accessory structures that contain no conditioned floor area.

1507.17.5 Fasteners. Fasteners for photovoltaic shingles shall be galvanized, stainless steel, aluminum or copper roofing nails, minimum 12 gage [0.105 inch (2.67 mm)] shank with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing materials and a minimum of 3/4 inch (19.1 mm) into the roof sheathing. Where the roof sheathing is less than 3/4 inch (19.1 mm) thick, the nails shall penetrate through the sheathing. Fasteners shall comply with ASTM F 1667.

Reason: This code change proposal adds specific requirements for roof decks, roof deck slope, underlayment, underlayment application, underlayment attachment in high wind regions, ice barrier and fasteners to Section 1507.17 on photovoltaic shingles.

The specific requirements being added are consistent with similar attributes for other steep-slope, shingle-type roof coverings. For example, the added Section 1507.17.1 and Section 1507.7.2 are adapted from Section 1507.5.1 and Section 1507.5.2,

respectively. Section 1507.3 and Section 1507.4 are adapted from Section 1507.2.3 and Section 1507.2.8, respectively. Section 1507.17.5 is adapted from Section 1507.2.6

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

S47-12

• • • • =				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1507.17.1 (NEW)-S-GRAHAM

S46–12 202, 1505.8, 1507.17, 1507.17.1, 1507.17.2, 1507.17.3,

Proponent: Lorraine Ross, Intech Consulting Inc., representing The Dow Chemical Company

Revise as follows:

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

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

1507.17.1 Material standards. Photovoltaic modules <u>BIPV</u>/shingles shall be listed and labeled in accordance with UL 1703.

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

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

Revise as follows:

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

BUILDING INTEGRATED PHOTOVOLTAIC (BIPV) PRODUCT. A building product that incorporate photovoltaic modules, which covert solar radiation into electricity, and functions as a component of the building envelope.

PHOTOVOLTAIC PANEL (PV PANEL) SYSTEM. A system that combines discrete photovoltaic panels with photovoltaic modules, which covert solar radiation into electricity, with rack support systems that are mounted on a building or installed on a building site.

Reason: This code change proposal harmonizes the IBC definition for BIPV shingles with that contained in the recently approved ICC ES AC 365 (ACCEPTANCE CRITERIA FOR BUILDING-INTEGRATED PHOTOVOLTAIC (BIPV) ROOF COVERING SYSTEMS). Accordingly, Sections 1505.8 and 1507.17 are revised editorially to reflect the new definition.

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

Staff note: The terminology proposed "photovoltaic panel (PV panel) system is currently not used in the IBC.

S46-12

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

202-PHOTOVOLTAIC MODULES SHINGLES-S-ROSS.doc

S48–12 1507.18 (NEW), 1507.18.1 (NEW), 1507.18.2 (NEW), 1507.18.3 (NEW)

Proponent: Bob Eugene, Underwriters Laboratories (robert.eugene@ul.com)

Add new text as follows:

1507.18 Polymer composite shingles. The installation of polymer composite shingles shall comply with the provisions of Sections 1507.18.1 through 1507.18.3.

1507.18.1 Material standards. Polymer composite shingles shall be listed and labeled in accordance with ASTM E 108 or UL 790.

1507.18.2 Attachment. Polymer composite shingles shall be attached as required by the manufacturer.

1507.18.3 Wind resistance. Polymer composite shingles shall be tested in accordance with procedures and acceptance criteria in ASTM D 3161. Polymer composite shingles shall comply with the classification requirements of Table 1507.2.7.1(2) for the appropriate maximum nominal design wind speed. Formed polymer composite shingle packaging shall bear a label to indicate compliance with the procedures in ASTM D 3161 and the required classification from Table 1507.2.7.1(2)

Reason: The proposal provides guidance for installers and code officials regarding the installation of polymer composite 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.

S48-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1507.18 (NEW)-S-EUGENE

S49–12

1508.1

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

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

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 passes the tests of FM 4450 or UL 1256 when tested as an assembly.

Exceptions:

- 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.
- 3. Where a thermal barrier meeting the requirements of Section 2603.4 is used to separate the foam plastic insulation from the interior of the building 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 an exception to the test requirements when a thermal barrier is used. 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.

S49-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					1508.1-S-ENNIS

S50–12 Table 1508.2

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

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

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	ASTM C 578
Fiber-reinforced gypsum board	<u>ASTM C 1278</u>
Glass-faced gypsum board	<u>ASTM C 1177</u>
Mineral fiber insulation board	ASTM C 726
Perlite board	ASTM C 728
Polyisocyanurate board	ASTM C1289, Type I or Type II
Wood fiberboard	ASTM C 208

Reason: This code change proposal is intended to add recognized product standards to Table 1508.2-Material Standards for Roof Insulation for fiber-reinforced gypsum board and glass-faced gypsum board commonly used in roof assemblies.

ASTM C1278, "Standard Specification for Fiber-Reinforced Gypsum Panel," is the U.S. product standard applicable to fiberreinforced gypsum board used in roof assemblies.

ASTM C1177, "Standard Specification for Glass Matt Substrate Used as Sheathing," is the U.S. product standard applicable to glass-faced gypsum board used in roof assemblies.

S50-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					T1508.2-S-GRAHAM.doc

S51–12 202 (NEW), 1509 (NEW), 1509.1 (NEW), 1509.2 (NEW), 1509.3 (NEW), Chapter 35 (NEW)

Proponent: Ken Sagan, NRG Code Advocates, representing Reflective Insulation Mfg. Assoc. International (ken@nrgcodeadvocates.com)

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

Add new text as follows:

SECTION 202 DEFINITIONS

RADIANT BARRIER. A material having a low emittance surface (0.1 or less) and where installed in building assemblies, the low emittance surface shall face a ventilated or unventilated air space.

Add new text as follows:

SECTION 1509 RADIANT BARRIER-ABOVE DECK

1509.1 General. The use of above-deck radiant barriers shall be permitted provided that the radiant barrier is covered with an *approved* roof covering and passes the tests of FM 4450 or UL 1256 when tested as an assembly.

1509.2 Radiant barrier. Installed above-deck shall have a continuous 0.5 inch (minimum) air space on the low emittance side of the product.

1509.3 Material standards, Above-deck radiant barrier shall comply with ASTM C1313/1313M

Add new standard to Chapter 35 as follows:

ASTM

C1313/C1313M-10 Standard Specification for Sheet Radiant Barriers for Building Construction Applications

Reason: There is a common misunderstanding in the market that some radiant barrier products installed above-deck, typically between the deck and the felt, provide some level of thermal benefit. This is not the case and this proposal intends to clarify the air gap requirements for above-deck radiant barriers.

References:

ASTM C1313/C1313M-10 Standard Specification for Sheet Radiant Barriers for Building Construction Applications

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

001-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1509-S-SAGAN

Proponent: Marshall Klein, Marshall A. Klein & Associates, Inc.

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

Revise as follows:

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

Exceptions:

- On buildings of Type I construction, the exterior walls and roofs of penthouses with a fire separation distance greater than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be permitted to have not less than a 1-hour fire-resistance rating. The exterior walls and roofs of penthouses with a fire separation distance of 20 feet (6096 mm) or greater shall not be required to have a fire-resistance rating.
- On buildings of Type I construction two stories or less in height above grade plane or of Type II construction, the exterior walls and roofs of penthouses with a fire separation distance greater

than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be permitted to have not less than a 1-hour fire-resistance rating or a lesser fire-resistance rating as required by Table 602 and be constructed of fire-retardant-treated wood. 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 fire-retardant-treated wood and shall not be required to have a fire-resistance rating. Interior framing and walls shall be permitted to be constructed of fire-retardant-treated wood.

- 3. On buildings of Type III, IV or V construction, the exterior walls of penthouses with a fire separation distance greater than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be permitted to have not less than a 1-hour fire-resistance rating or a lesser fire-resistance rating as required by Table 602. On buildings of Type III, IV or VA construction, the exterior walls of penthouses with a fire separation distance of 20 feet (6096 mm) or greater shall be permitted to be of Type IV or noncombustible construction or fire-retardant-treated wood and shall not be required to have a fire-resistance rating.
- 4. Where the penthouse is constructed in accordance with Section 1509.2 and is considered as a portion of the story directly below the roof deck, the floor of the penthouse is permitted to be constructed as required by Table 601 for the roof of the building on which such penthouse is built.

Reason: Based on the intent of Section 1509.2, the "floor" of the penthouse should be considered for its fire resistance rating requirement under Table 601 as the "roof" of the building on which it is built.

If the provisions of the base paragraph of Section 1509.2.5 are followed as the code currently states, then the penthouse floor is required to be constructed "as required for the type of construction of the building on which such penthouses are built." This would imply that the floor of the penthouse must be built to the higher floor construction requirements of Table 601 instead of the more relaxed roof construction requirements for Construction Type I under Table 601 (i.e. Type IA from 2 hours to 1½ hours and Type IB from 2 hours to 1 hour).

The language of this new exception will clarify that the floor of penthouses that comply with the height, area, and use limitations mentioned in 1509.2 shall be built to the building's roof construction fire resistance rating of Table 601. In addition, penthouses' floors that comply with Section 1509 are intended to permit the use of the shaft requirement under Section 713.12 that relate to terminating the shaft at the roof not the floor. Conversely, if the penthouse does not meet the limitations that Section 1509.2 mentions (which would be compliance with Sections 1509.2.1 through 1509.2.5), then as the last sentence of that section states, the penthouse "shall be considered as an additional story", and the floor of the penthouse would need to meet the fire resistance floor requirements of Table 601 for the building's type of construction.

S52-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1509.2.5-S-KLEIN

S53–12 1509.7.1

Proponent: Christine Covington, Solar Energy Industries Association

Revise as follows:

1509.7.1 Wind resistance <u>Structural loads</u>. Rooftop mounted photovoltaic systems shall be designed for wind loads for component and cladding capable of resisting applicable structural loads in accordance with Chapter 16 using an effective wind area based on the dimensions of a single unit frame.

Reason: Rooftop PV systems may be subjected to structural loads other than wind. Seismic and snow loads may also be applicable and should be evaluated as part of the design.

IBC Chapter 16 addresses design loads with reference to ASCE 7. Chapter 16 and ASCE 7 include requirements for combinations of loads. Wind requirements are the subject of Chapters 26-31 of ASCE 7-10, which include multiple methods of determining wind loads. Components and cladding methods are appropriate for some rooftop PV systems, but not all. For example, some tall rooftop systems experience wind behavior appropriate to the Main Wind Force Resisting System, and some systems held close to the roof surface have been studied using Wind Tunnel testing. These approved wind load evaluation methods appear to be prohibited by the current language without justification.

S53-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	,				1509.7.1-S-COVINGTON

Proponent: Joseph H. Cain, P.E., SolarCity Corporation, representing self

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

Revise as follows:

1509.7.2 Fire classification. Rooftop mounted photovoltaic systems panels and modules shall have the same a fire classification as the roof assembly required for the roof assembly by Section 1505.

Reason: The testing method currently under development by the UL 1703 Standards Technical Committee is overly restrictive. The proposed code change will restore a photovoltaic (PV) panel/module fire classification without disproportionate economic burden to one industry.

Revisions to UL 1703 to include a PV system test are not yet ready for 2012 IBC implementation. Preliminary study of PV system fire classification has been conducted by Solar America Board for Codes and Standards in conjunction with Underwriter's Laboratories. For further information on Solar ABC's on-going fire testing, visit <u>http://www.solarabcs.org/current-issues/fire_class_rating.html</u>

Preliminary results indicate the test procedures being developed under proposed revisions to UL 1703 are overly restrictive, as the tests show nearly all common existing configurations of PV systems as "non-compliant." Using the existing language in 2012 IBC Section 1509.7.2, the UL 1703 Standards Technical Panel is developing language that would effectively trigger a fundamental re-start of the solar industry. The preliminary results indicate a testing standard with possible mitigation measures disproportionate in cost to the risks associated with rooftop installation of solar energy systems.

Preliminary tests by Underwriter's Laboratories have identified only three possible mitigation measures to date for rackmounted PV systems. While further study is in order, the mitigation measures studied to date do not appear to be practical or costeffective. The first mitigation measure is to install panels/modules in direct contact with the roof surface. This mitigation will be difficult or impossible to achieve on varying profiles of roof covering materials or on many roof surfaces experiencing code-allowable deflection. The second mitigation measure is to provide a barrier to prevent fire from entering between the PV panel system and the roof covering. An enclosure around the perimeter of all arrays installed tight to the profile of the roof covering will adversely affect the cost-effectiveness of solar electric power as it will simultaneously increase the cost of installation and decrease the production of electricity owing to over-heating panels/modules. The third mitigation measure is to install PV systems high above the roof surface. This will simultaneously increase cost and reduce sales of PV systems, as building owners and architects/designers are concerned about aesthetics as well as system performance.

The rapid adoption of solar technologies is a matter of national importance.

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

S54-12

Public Hearing: Comn	nittee: A	AS	AM	D	
Asser	mbly: A	ASF	AMF	DF	
	•			1	509.7.2 #1-S-CAIN

S55-12 1509.7.2

Proponent: Christine Covington, Solar Energy Industries Association

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

Revise as follows:

1509.7.2 Fire classification. Rooftop mounted photovoltaic systems shall have the same fire classification as the roof assembly required by Section 1505.

Exception: Photovoltaic panels and modules having a minimum Class C fire classification are permitted where listed and labeled in accordance with UL 1703 and where installed in direct contact with the roof surface.

Reason: Fire testing of photovoltaic panels/modules was conducted on various roof systems by Underwriter's Laboratories in conjunction with Solar America Board for Codes and Standards (Solar ABC's). This study was conducted to assess the influence of PV panels/modules on the performance of classified roofing systems. This testing found that PV panels/modules placed in contact with the roof deck eliminated channeling of fire that was observed in some of the fire testing for elevated rack mounted systems. Channeling has been shown to contribute to flame spread when conducting the "spread of flame" test component of the fire classification evaluation. When PV panels/modules are installed in contact with the roof, the fire classification of the underlying roof system was not diminished. Therefore, this exception meets the ICC membership's intent to ensure that the installation of PV panels/modules do not degrade the fire classification rating of underlying roof systems.

For further information on Solar ABC's on-going fire testing, visit http://www.solarabcs.org/current-issues/fire_class_rating.html

S55-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1509.7.2 #1-S-COVINGTON

S56-12 1509.7.2

Proponent: Joseph H. Cain, P.E., SolarCity Corporation, representing self

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

Revise as follows:

1509.7.2 Fire classification. Rooftop mounted photovoltaic systems shall have the same fire classification as the roof assembly required by Section 1505.

Exception: Photovoltaic panels and modules having a minimum Class C fire classification are permitted where listed and labeled in accordance with UL 1703 and where installed on unlimited area buildings, as established in Section 507.

Reason: The testing method currently under development by the UL 1703 Standards Technical Committee is overly restrictive. The proposed code change will provide an exception for buildings with less risk, such as sprinklered, single story box stores surrounded and adjoined by public ways or yards.

Revisions to UL 1703 to include a PV system test are not yet ready for 2012 IBC implementation. Preliminary study of PV system fire classification has been conducted by Solar America Board for Codes and Standards in conjunction with Underwriter's Laboratories. For further information on Solar ABC's on-going fire testing, visit <u>http://www.solarabcs.org/current-issues/fire_class_rating.html</u>

Preliminary results indicate the test procedures being developed under proposed revisions to UL 1703 are overly restrictive, as the tests show nearly all common existing configurations of PV systems as "non-compliant." Using the existing language in 2012 IBC Section 1509.7.2, the UL 1703 Standards Technical Panel is developing language that would effectively trigger a fundamental re-start of the solar industry. The preliminary results indicate a testing standard with possible mitigation measures disproportionate in cost to the risks associated with rooftop installation of solar energy systems.

Preliminary tests by Underwriter's Laboratories have identified only three possible mitigation measures to date for rackmounted PV systems. While further study is in order, the mitigation measures studied to date do not appear to be practical or costeffective. The first mitigation measure is to install panels/modules in direct contact with the roof surface. This mitigation will be difficult or impossible to achieve on varying profiles of roof covering materials or on many roof surfaces experiencing code-allowable deflection. The second mitigation measure is to provide a barrier to prevent fire from entering between the PV panel system and the roof covering. An enclosure around the perimeter of all arrays installed tight to the profile of the roof covering will adversely affect the cost-effectiveness of solar electric power as it will simultaneously increase the cost of installation and decrease the production of electricity owing to over-heating panels/modules. The third mitigation measure is to install PV systems high above the roof surface. This will simultaneously increase cost and reduce sales of PV systems, as building owners and architects/designers are concerned about aesthetics as well as system performance.

The rapid adoption of solar technologies is a matter of national importance.

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

S56-12 Public Hearing: Committee: AS AM D Assembly: ASF AMF DF

S57-12 1509.7.2

Proponent: Christine Covington, Solar Energy Industries Association

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

Revise as follows:

1509.7.2 Fire classification. Rooftop mounted photovoltaic systems shall have the same fire classification as the roof assembly required by Section 1505.

Exception: Photovoltaic panels having a minimum Class C fire classification are permitted where listed and labeled in accordance with UL 1703 and where installed at least 12 inches (305mm) above the roof surface.

Reason: Fire testing of photovoltaic panels/modules was conducted on various roof systems by Underwriter's Laboratories in conjunction with Solar America Board for Codes and Standards (Solar ABCs). This study was conducted to assess the influence of PV panels/modules on the performance of classified roofing systems. This testing found that PV panels/modules raised sufficiently above the roof deck reduced heat flux temperatures and mitigated any deleterious effects caused by channeling of fire underneath raised "rack mount" systems. Channeling has been shown to contribute to flame spread when conducting the "spread of flame" test component of the fire classification evaluation. When PV panels/modules are raised at least 12", the fire classification of the underlying roof system was not diminished. Therefore, this exception meets the ICC membership's intent to ensure that the installation of PV panels/modules do not degrade the fire classification rating of underlying roof systems.

S57-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
2				1509.7.2 #2-S-COVINGTON

S58-12 1509.7.2

Proponent: Christine Covington, Solar Energy Industries Association

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

Revise as follows:

1509.7.2 Fire classification. Rooftop mounted photovoltaic systems shall have the same fire classification as the roof assembly required by Section 1505.

Exception: Photovoltaic panels and modules arranged in arrays are permitted where the following conditions are met:

- 1.1. Photovoltaic panels and modules have a minimum Class C fire classification where listed and labeled in accordance with UL 1703;
- 1.2. The entire perimeter of the array is provided with a 0.0187-inch (0.4712 mm) (No. 26 gage) corrosion resistant steel or equivalent approved barrier extending from the array to the roof and;
- <u>1.3. Where any openings between the individual panels and modules on the upper face are</u> provided with screens, flashing or otherwise protected to prevent entry of vegetative debris.

Reason: Fire testing of photovoltaic panels/modules was conducted on various roof systems by Underwriter's Laboratories in conjunction with Solar America Board for Codes and Standards (Solar ABCs). This study was conducted to assess the influence of PV panels/modules on the performance of classified roofing systems. This testing found that PV panels/modules provided with perimeter fire barrier flashing extending from the panel/module to the roof eliminated channeling of fire that was observed in some of the fire testing for elevated rack mounted systems. Channeling has been shown to contribute to flame spread when conducting the "spread of flame" test component of the fire classification evaluation. When PV panels/modules are installed with barrier flashing, the fire classification of the underlying roof system was not diminished. Therefore, this exception meets the ICC membership's intent to ensure that the installation of PV panels/modules do not degrade the fire classification rating of underlying roof systems. For further information on Solar ABC's on-going fire testing, visit <u>http://www.solarabcs.org/current-issues/fire_class_rating.html</u>

S58-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	•				1509.7.2 #3-S-COVINGTON

S59-12

Proponent: Michael D. Fischer, Kellen Company, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

Revise as follows:

1510.1 General. Materials and methods of application used for recovering or replacing an existing roof covering shall comply with the requirements of Chapter 15.

Exception: Reroofing Roof replacement or roof recover of existing low-slope roof coverings shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 for roofs that provide positive roof drainage.

Reason: The current text is not clear that steep slope roof coverings are not included in the exception to the ¼ minimum slope requirement. This change is largely editorial since the roof covering approvals should govern.

S59-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1510.1-S-FISCHER

S60-12 1510.1

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1510.1 General. Materials and methods of application used for recovering or replacing an existing roof covering shall comply with the requirements of Chapter 15.

Exceptions:

- 1. Reroofing shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 for roofs that provide positive roof drainage.
- 2. Recovering or replacing an existing roof covering shall not be required to meet the requirement for secondary (emergency overflow) drains or scuppers in Section 1503.4 for roofs that provide for positive roof drainage.

Reason: IBC 2006 and subsequent editions include a requirement in Section 1503.4-Roof Drainage that for roof drainage systems with roof drains or scuppers, secondary (emergency overflow) drains or scuppers also be provided in the event the primary roof drainage system becomes clogged.

Section 1510-Reroofing requires all materials and methods used in recovering or replacing an existing roof covering comply with the requirements of Chapter 15 (except the minimum roof slope requirement of 1/2:12 can be waived for roofs that provide ...positive roof drainage."). This can be interpreted to require the secondary (emergency overflow) drains and scupper provision also apply in reroofing. Since many existing buildings were designed and constructed before the code included a secondary drainage requirement, the secondary drainage provision being applicable in reroofing and the need for adding secondary drains in exsiting buildings during reroofing can be a very costly and disruptive undertaking for owners and occupants.

This proposed code change adds an exception in Section 1510-Reroofing that waives the secondary drainage provision when reroofing existing buildings when the roof drains properly, that being hat provide for positive roof drainage. The term "positive roof drainage' is already defined in Section 202.

S60-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1510 1-S-GRAHAM

S61 – 12 1510.2

Proponent: Michael D. Fischer, Kellen Company, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

Revise as follows:

1510.2 Structural and construction loads. Structural roof components shall be capable of supporting the roof-covering system and the material and equipment loads that will be encountered during installation of the system. Existing structural assemblies shall comply with the requirements of Section <u>3404.</u>

Reason: Chapter 34 provides good guidance to the designer regarding the types of conditions that should be evaluated during alterations. This proposal provides a necessary reference for the purposes of linking those requirements.

S61-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1510.2-S-FISCHER

S62–12 1510.3 (NEW), 1510.4

Proponent: Michael D. Fischer, Kellen Company, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

Revise as follows:

1510.3 Roof replacement. Roof replacement shall include the removal of all existing layers of roof coverings down to the roof deck.

Exceptions:

- 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 where 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.
- 5. 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.
- 6. Where the existing roof covering is wood shake, slate, clay, cement or asbestos-cement tile.
- 7. Where the existing roof has two or more applications of any type of roof covering.

1510.3 <u>**1510.4**</u> Recovering versus replacement <u>Roof recovering</u>. New roof coverings shall not be installed without first removing all existing layers of roof coverings down to the roof deck <u>Roof recovering</u> shall be prohibited 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:

- 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.

Reason: The current text is confusing and contains directions on what NOT to do regarding roof recovering. The proposal reorganizes the text without making any technical changes in order to add clarity to the code. The revisions provide clear distinction between roof replacement and roof recovering

S62-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	•				1510.3 (NEW)-S-FISCHER

S63–12 1510.3, 1510.4

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

Delete and substitute 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:

- 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 lower of ice barrier membrane in pacendance with Section 1507
 - layer of ice barrier membrane in accordance with Section 1507.

1510.4 Roof recovering. Where the application of a new roof covering over wood shingle or shake roofs creates a combustible concealed space, the entire existing surface shall be covered with gypsum board, mineral fiber, glass fiber or other *approved* materials securely fastened in place.

1510.3 Roof replacement. Roof replacement requires the removal of all existing roof coverings layers down to the roof deck before the new roof covering is installed. A roof replacement is required and roof recovering shall be prohibited where any of the following conditions exist:

- 1. <u>Areas of the existing roof or roof covering that are water soaked or that have deteriorated to the</u> point that the existing roof or roof covering is not adequate as a base for additional roofing unless those areas are removed and replaced prior to recover.
- 2. <u>The existing roof covering is wood shake, slate, clay, cement, or asbestos-cement tile unless it is</u> recovered in accordance with Section 1510.4.2.
- 3. <u>The existing roof has two or more applications of any type of roof covering unless recovered in accordance with Section1510.4.5.</u>

1510.4 Roof recovering. Roof recovering shall be permitted except where prohibited by section 1510.3.

1510.4.1 Existing complete and separate roofing systems. 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 do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.

1510.4.2 Existing wood shake roofs. Metal panel, metal shingle and concrete and clay tile roof
coverings shall be permitted to be installed over existing wood shake roofs where the entire surface is covered with gypsum board, mineral fiber, glass fiber, or other *approved* materials securely fastened in place.

1510.4.3 Existing spray polyurethane foam roofing systems. 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.

1510.4.4 Existing ice barrier membrane. 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.

1510.4.5 New single-ply membrane. The application of a new single-ply membrane directly over an existing low-slope (roof slope < 2:12) roofing system shall be permitted without tear-off of the existing roof coverings.

Reason: This code change proposal accomplishes several objectives:

- 1. Clarifies when a roof needs to be replaced and when is may be recovered.
- 2. Requires that existing water soaked or deteriorated sections of the roof be removed and replaced prior to recovering the roof. The current language would allow those areas to remain in place if any of the exceptions are used.
- 3. Adds an exception for the installation of a new single ply membrane because a layer of single-ply membrane is very lightweight, adding approximately 1/3 of a pound per square foot to the existing structure. In Climate Zones 1, 2 and 3, a single-ply membrane can be used as a reflective layer to reduce rooftop temperatures, thus providing a cooling benefit for the building, meeting the requirements of the International Energy Conservation Code and the heat island mitigation requirements of the International Green Construction Code. This exception will provide building owners with a cost effective option, with a variety of membrane colors and types, for installing a new waterproofing layer in all climate zones.

The new roof system will still need to meet the requirements of Chapter 15 as called out in Section 1510.1.

S63-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1510.3-S-ENNIS

S64–12 1510.7 (NEW)

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Proponent: Al Godwin, CBO, CPM, representing Aon Fire Protection Engineering (al.godwin@aon.com)

Add new text as follows:

1510.7 Construction of sloped roof over flat roof. Construction of a new roof over an existing roof, in a manner that creates an attic or concealed space shall require the removal of any existing roofing material composed of tar, asphalt or roof insulation not designed for interior use from the newly created interior space.

Reason: It is not uncommon for building owners to convert a flat roof to a sloped roof. When doing so, the former roofing material should be removed from the newly created interior space.

S64-11				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1510.7 (NEW)-S-GODWIN

S65–12

Proponent: Maureen Traxler, City of Seattle Department of Planning & Development (Maureen.traxler@seattle.gov); Thomas Meyers, City of Central, CO, representing self

Delete without substitution:

1511.1.1 Structural fire resistance. The structural frame and roof construction supporting the load imposed upon the roof by the photovoltaic panels/modules shall comply with the requirements of Table 601.

Reason: (Traxler) This section is not needed because Table 601 will apply regardless of this section. In addition, the terminology used is not consistent with the terms used in Table 601, creating confusion about whether the "structural frame...supporting the load imposed upon the roof" is different than the primary structural frame and secondary members referenced in Table 601. If they are different, then Table 601 doesn't have any applicable requirements. If they are the same, the section isn't necessary because compliance with Table 601 is already required by Chapter 6.

(Meyers) This new section was added as part of a comprehensive code change submitted to the IFC and ultimately approved as modified by public comment at the Dallas Final Action Hearings. The new subsection 1511.1.1 has generated considerable confusion. It has been interpreted to require any of the stand-off rack frame used to mount solar panels to the roof to be fire resistance rated consistent with the Type of Construction used by the building. In the case of I-A construction, this interpretation would require the typical aluminum square tube "column" supports to exhibit 3 hour fire endurance. This is extremely excessive and very difficult to achieve in an exposed, exterior application.

It appears that the intent may have been to ensure that the underlying supporting roof structure be provided with the fire performance prescribed by Chapter 6 when supporting any loads imposed by the solar panel array system that includes the racking system. The code already ensures that in Chapter 6. Therefore, this section is completely redundant. As such, it should be eliminated to avoid confusion.

S65-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1511.1.1-S-TRAXLER.doc

S66–12 202

Proponent: Phillip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Delete without substitution:

ARCHITECTURAL TERRA COTTA. Plain or ornamental hard-burned modified clay units, larger in size than *brick*, with glazed or unglazed ceramic finish.

BOND BEAM. A horizontal grouted element within masonry in which reinforcement is embedded.

DURATION OF LOAD. The period of continuous application of a given *load*, or the aggregate of periods of intermittent applications of the same *load*.

GLUED BUILT-UP MEMBER. A structural element, the section of which is composed of built-up lumber, wood structural panels or wood structural panels in combination with lumber, all parts bonded together with structural adhesives.

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"*).

RUBBLE MASONRY. Masonry composed of roughly shaped stones.

Coursed rubble. Masonry composed of roughly shaped stones fitting approximately on level beds and well bonded.

Random rubble. Masonry composed of roughly shaped stones laid without regularity of coursing but well bonded and fitted together to form well-divided *joints*.

Rough or ordinary rubble. Masonry composed of unsquared field stones laid without regularity of coursing but well bonded.

STACK BOND. The placement of *masonry units* in a bond pattern is such that head *joints* in successive courses are vertically aligned. For the purpose of this code, requirements for stack bond shall apply to *masonry* laid in other than *running bond*.

SUBDIAPHRAGM. A portion of a larger wood *diaphragm* designed to anchor and transfer local forces to primary *diaphragm* struts and the main *diaphragm*.

Revise as follows:

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

[A] 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").

[A] MARK. An identification applied on a product by the manufacturer indicating the name of the manufacturer and the function of a product or material (see also "Inspection certificate," "Label" and "Manufacturer's designation").

Reason: The definitions are being deleted because they serve no purpose in the building code. There are no instances of any of the defined terms in the 2012 IBC other than as shown in this proposal.

S66-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1602.1-S-BRAZIL.doc

S67–12 1602

Proponent: Charles S. Bajnai, Chesterfield County, VA, representing ICC Building Code Action Committee (bajnaic@chesterfield.gov)

Revise as follows:

DIAPHRAGM. A horizontal or sloped system acting to transmit lateral forces to the vertical-resisting elements vertical elements of the lateral-force-resisting-system. When the term "diaphragm" is used, it shall include horizontal bracing systems.

Reason: This proposal cleans up a grammatical error with the current language. The current definition reads, "...transmit lateral forces to the **vertical-resisting** elements."

As written with the hyphenated term "vertical-resisting", it means that the "elements" resist "vertical" which doesn't make sense. The definition should convey that the vertical elements of the system resist the lateral forces transmitted from the diaphragm(s). The current definition is the same as the definition in American Forest & Paper Associations' *Special Design Provisions for Wind and Seismic* (SDPWS) with the exception of the hyphen between "vertical" and "resisting" that does not occur in SDPWS. Better language is provided in American Society of Civil Engineers', *ASCE 7* where it states, for "Diaphragm Flexibility" in Section 12.3.1, "The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the lateral-forceresisting systems." The ASCE 7 language is the best definition of the three and this proposal corrects the error in the current language and aligns it with ASCE 7.

IBC section 1604.4 reads correctly and states,

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.

This proposal does not change any technical requirements of the code. It merely addresses a grammar error and promotes consistency with ASCE 7 standard.

S67-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1602-S-BAJNAI-BCAC.doc

S68–12 202 (NEW), 1607.12.3.1

Proponent: Sarah A. Rice, C.B.O., The Preview Group (srice@preview-group.com)

Revise as follows:

1607.12.3.1 Landscaped <u>and vegetative</u> roofs. The uniform design live load in unoccupied landscaped areas on roofs <u>and vegetative roofs</u> shall be 20 psf (0.958 kN/m²). The weight of all landscaping <u>and</u> <u>vegetative roof</u> materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

Add new text as follows:

SECTION 202 DEFINITIONS

VEGETATIVE ROOF. An assembly of interacting components designed to waterproof and normally insulate a building's top surface that includes, by design, vegetation and related landscaping elements.

Reason: The IgCC includes an option for addressing the heat island impact of roofs with the installation of a vegetative roof on all or portions of the roof. The IBC addresses landscaped roofs and roof gardens. The vegetative roof is more akin to the landscaped roof in that it is not intended to be an area regularly occupied by building occupants, but is a feature similar to other roof coverings. This change places the loading concerns for vegetative roofs in the same section of Chapter 16 that regulates landscaped roofs. The proposed definition is a direct copy for the 2012 IgCC. It should be scoped for long term maintenance to the Code Development committees for the IgCC.

S68-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1607.12.3.1-S-RICE.doc

S69–12 1603.1.3

Proponent: Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithustoninc.com)

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/m2), 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. Drift surcharge load, p_d , where the sum of p_d and P_f exceeds 20 pounds per square foot (psf).
- 6. Width of snow drift, w.

Reason: The addition of loading information and design assumptions to drawings has been valuable to owners and the engineers who are tasked with re-evaluating existing structures. This additional requirement of snow drift design information supplements the information already required and indicates how the registered design professional interpreted the design codes relative to snow drift intensity and width.

S69-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				1603.1.3-S-HUSTON

S70-12

703.2.3, [F]909.9, 1603.1.4, 1607.10.1.2, 1607.10.2, 1704.3.1, 1704.5 (NEW), 1704.5.1, 1704.5.2, Table 1705.3, Table 1705.7, 1705.9, 1705.12.3, 1803.4, 1910.9.3, 2207.2, 2207.3, 2303.4.1.4.1

THIS IS A 3 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE FIRESAFETY COMMITTEE AND PART II WILL BE HEARD BY THE FIRE CODE COMMITTEE, AS THREE SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent: Phillip Brazil, P.E., S.E., Reid Middleton, representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

PART I – IBC STRUCTURAL

Revise as follows:

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 structure:

- 1. 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. 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/m²) to be used for the design of exterior component and cladding materials not specifically designed by the <u>responsible</u> registered design professional.

1607.10.1.2 Heavy live loads. Live loads that exceed 100 psf (4.79 kN/m²) shall not be reduced.

Exceptions:

- 1. The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than *L* as calculated in Section 1607.10.1.
- 2. For uses other than storage, where *approved*, additional live load reductions shall be permitted where shown by the <u>responsible</u> registered design professional that a rational approach has been used and that such reductions are warranted.

1607.10.2 Alternative uniform live load reduction. As an alternative to Section 1607.10.1 and subject to the limitations of Table 1607.1, uniformly distributed 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.

 A reduction shall not be permitted where 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 a maximum of 20 percent.

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

 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.

(Portions of section not shown remains unchanged)

Revise as follows:

1704.3.1 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 <u>responsible</u> *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 1705.10, 1705.11 and 1705.12.
- 5. For each type of *special inspection*, identification as to whether it will be continuous *special inspection* or periodic *special inspection*.

1704.5 Submittals to the building official. In addition to the submittal of reports of special inspections and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by the responsible registered design professional and prior to the construction or work being performed for each of the following:

(Renumber remaining sections)

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 where one or more of the following conditions exist:

- 1. The structure is classified as *Risk 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 *Risk 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 <u>responsible</u> registered design professional responsible for the structural design.
- 5. When such observation is specifically required by the *building official*.

1704.5.2 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where *Vasd* as determined in accordance with Section 1609.3.1 exceeds 110 mph (49 m/sec), where one or more of the following conditions exist:

- 1. The structure is classified as *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 <u>responsible</u> registered design professional responsible for the structural design.
- 4. When such observation is specifically required by the building official.

TABLE 1705.3 REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspections for seismic resistance.

b. Specific requirements for special inspection shall be included in the research report for the anchor issued by an approved source in accordance with ACI 355.2 or other qualification procedures. Where specific requirements are not provided, special inspection

requirements shall be specified by the <u>responsible</u> registered design professional and shall be approved by the building official prior to the commencement of the work.

(Portions of table not shown remains unchanged)

TABLE 1705.7

REQUIRED VERIFICATION AND INSPECTION OF DRIVEN DEEP FOUNDATION ELEMENTS VERIFICATION AND INSPECTION TASK CONTINUOUS DURING TASK LISTED PERIODICALLY DURING TASK LISTED 7. For specialty elements, perform additional inspections as determined by the responsible registered design professional in responsible charge. — —

(Portions of table not shown remains unchanged)

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 <u>responsible</u> registered design professional in responsible charge. The approved geotechnical report and the *construction documents* prepared by the registered design professionals shall be used to determine compliance.

1705.12.3 Seismic certification of nonstructural components. The <u>responsible</u> *registered design professional* shall specify on the construction documents the 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 1705.12.

Revise as follows:

1803.4 Qualified representative. The investigation procedure and apparatus shall be in accordance with generally accepted engineering practice. The <u>responsible</u> registered design professional shall have a fully qualified representative on site during all boring or sampling operations.

Revise as follows:

1910.9.3 Natural curing. Natural curing shall not be used in lieu of that specified in this section unless the relative humidity remains at or above 85 percent, and is authorized by the <u>responsible</u> registered design professional and approved by the *building official*.

Revise as follows:

2207.2 Design. The <u>responsible</u> registered design professional shall indicate on the *construction documents* the steel joist and/or steel joist girder designations from the specifications listed in Section 2207.1 and shall indicate the requirements for joist and joist girder design, layout, end supports, anchorage, non-SJI standard bridging, bridging termination connections and bearing connection design to resist uplift and lateral loads. These documents shall indicate special requirements as follows:

- 1. Special loads including:
 - 1.1. Concentrated loads;
 - 1.2. Non-uniform loads;
 - 1.3. Net uplift loads;
 - 1.4. Axial loads;
 - 1.5. End moments; and
 - 1.6. Connection forces.
- 2. Special considerations including:
 - 2.1. Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder configurations are as indicated in the SJI catalog);

- 2.2. Oversized or other nonstandard web openings; and
- 2.3. Extended ends.
- 3. Deflection criteria for live and total loads for non-SJI standard joists.

2207.3 Calculations. The steel joist and joist girder manufacturer shall design the steel joists and/or steel joist girders in accordance with the current SJI specifications and load tables to support the load requirements of Section 2207.2. The <u>responsible</u> *registered design professional* may require submission of the steel joist and joist girder calculations as prepared by a *registered design professional* responsible for the product design. If requested by the <u>responsible</u> *registered design professional*, the steel joist manufacturer shall submit design calculations with a cover letter bearing the seal and signature of the joist manufacturer's *registered design professional*. In addition to standard calculations under this seal and signature, submittal of the following shall be included:

- 1. Non-SJI standard bridging details (e.g.for cantilevered conditions, net uplift, etc.).
- 2. Connection details for:
 - 2.1. Non-SJI standard connections (e.g.flushframed or framed connections);
 - 2.2. Field splices; and
 - 2.3. Joist headers.

2303.4.1.4.1 Truss design drawings. Where required by the <u>responsible</u> *registered design professional*, the *building official* or the statutes of the jurisdiction in which the project is to be constructed, each individual truss design drawing shall bear the seal and signature of the truss designer.

Exceptions:

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

PART II - IBC FIRE SAFETY

Revise as follows:

703.2.3 Restrained classification. Fire-resistance-rated assemblies tested under ASTM E 119 or UL 263 shall not be considered to be restrained unless evidence satisfactory to the *building official* is furnished by the <u>responsible</u> *registered design professional* showing that the construction qualifies for a restrained classification in accordance with ASTM E 119 or UL 263. Restrained construction shall be identified on the plans.

PART III - IFC

[F] 909.9 Design fire. The design fire shall be based on a rational analysis performed by the *registered design professional* and *approved* by the fire code official. The design fire shall be based on the analysis in accordance with Section 909.4 and this section.

Reason: The building code frequently refers to registered design professionals by specifying requirements for "the registered design professional" to perform. The design of a building or structure, however, is not accomplished by a single ("the") registered design professional but by a design team consisting of several registered design professionals, including an architect, structural engineer, geotechnical engineer, mechanical engineer, electrical engineer, plumbing engineer, fire protection engineer, civil engineer and others. In these cases, requiring "the registered design professional" to perform certain tasks is ineffective in that the particular registered design professional expected to perform the task is not identified. The proposal resolves this by revising the code to specify that the "responsible" registered design professional" shall perform the tasks.

The building code also frequently refers to "a registered design professional" to perform certain tasks. In these cases, the required tasks are typically not associated with the actions of a design team for a building or structure but are for a single individual who is also a registered design professional. There are also instances where the language is more specific than "a registered

design professional" but the result is the same. We have judged these to be sufficiently clear that changes to the building code consistent with the intent of the proposal are not warranted. The instances are located in Sections 107.1, 107.3.4, 202 ("structural observation"), 909.21.4.4, 1603.1.9, 1605.1.1, 1612.3.1(2), 1612.5, 1704.3-Exc., 1704.5, 1705.6, 1705.7, 1705.8, 1709.2, 1709.3, 1709.3.2, 1710.3, 1803.1, 1803.3.1, 1803.5.10, 1804.4(2), 1810.2.1, 1810.2.4, 1810.3.3.1.2, 1810.3.5.2.2-Exc., 1810.4.11, 2109.3.4.1, 2207.4, 2211.3.3, 2303.4.1.2(3), 2303.4.1.3, 2303.4.4, 2303.4.5, 2308.8.2.1, 2308.10.7, 2403.2, 2404.4, 3405.2.1, B101.2.2, G103.3(2), J103.2(7), K105.3, K105.4, K105.5 and K105.6.

All instances of "registered design professional" in the building code were considered and are either in the proposal or are listed in the paragraph immediately above.

Note that there are instances of the "responsible registered design professional" in the building code and they are located in Section 909.18.8.3. Also, the definition of "registered design professional in responsible charge" was added to the building code by ICC proposal G33-06/07 – AS, Phillip Brazil, Proponent.

This proposal is also a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official (Sxx-12/13). The charging language in new Section 1704.5 is identical in both proposals except that this proposal adds "approved" before "construction documents."

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

S73-12

PART I - INTER	RNATIONAL BU	ILDING CODE	- STRUCTURA	L				
Public Hearing:	Committee:	AS	AM	D				
-	Assembly:	ASF	AMF	DF				
PART II – INTE	PART II – INTERNATIONAL BUILDING CODE – FIRE SAFETY							
Public Hearing:	Committee:	AS	AM	D				
	Assembly:	ASF	AMF	DF				
PART III – INTE	RNATIONAL FI	RE CODE						
Public Hearing:	Committee:	AS	AM	D				
	Assembly:	ASF	AMF	DF				

1603.1.4 #2-S-BRAZIL.doc

S71–12 1603.1.7

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov) (gregory.p.wilson@dhs.gov), Rebecca C. Quinn, RCQuinn Consulting, Inc., representing Department of Homeland Security, Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

1603.1.7 Flood design data. For buildings located in whole or in part in *flood hazard areas* as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

1.	Risk Category assigned according to ASCE 24.
<u> 1. 2.</u>	In flood hazard areas not subject to high-velocity wave action, the elevation of the proposed
	lowest floor, including the basement.

- 2.3. In *flood hazard areas* not subject to high-velocity wave action, the elevation to which any nonresidential building will be dry flood proofed.
- 3. <u>4.</u> In *flood hazard areas* subject to high-velocity wave action, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

Reason: The current edition of ASCE 24 uses the assigned occupancy/structure category primarily to determine elevation of buildings above the design flood elevation, in keeping with the general approach that more important buildings be designed for less frequent environmental loads. The next edition of ASCE 24 will include the Risk Category table from ASCE 7-10. The ASCE committee recognized that ASCE 7-10 eliminated the lists of buildings for each category and determined it important to ensure that the assignment of risk category be guided by definitions that are specifically developed to ensure that buildings in flood hazard areas are appropriately protected. Therefore, the next edition of ASCE 24 requires the user to reevaluate and possibly reassign a risk category specifically for the purpose of flood loads and flood resistant construction requirements.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: The code change proposal will not increase the cost of construction. The definitions of each risk category that will be in the revised ASCE 24 and used only for the purpose of assigning risk category for flood-resistant design essentially retain the descriptions from the 2012 IBC Table 1604.5 of which buildings fall into each of the risk categories.

Analysis: Will the proposal introduce a conflict with Section 1604.5?

1603.1.7-S-INGARGIOLA-WILSON-QUINN.doc

S72–12 1603.1.8.1 (NEW), 1607.12.5 (NEW), 1607.12.5.1 (NEW), 1607.12.5.2 (NEW), 1607.12.5.3 (NEW), 1607.12.5.4 (NEW)

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Add new text as follows:

1603.1.8.1 Solar Photovoltaic (PV) Panels/Modules. The Roof/PV live load used in the design of Solar PV Panels shall be indicated on the construction documents.

1607.12.5 Solar Photovoltaic (PV) panels/modules. Solar PV panels/modules shall be designed in accordance with Sections 1607.12.5.1 through 1607.12.5.4, as applicable.

1607.12.5.1 Roof/PV live load. The roof/PV live load is a 20 psf uniform load. Unless each Solar PV panel/module is clearly and permanently marked "Do not walk on this surface – not intended for maintenance access or pedestrian traffic", and appropriate maintenance access paths are provided a non-concurrent 300 pound concentrated load as set forth in Table 1607.1 shall also be applied. The individual Solar PV panels/modules shall be designed to withstand the Roof/PV live load, in combination with other applicable loads.

1607.12.5.2 PV panels/modules. Solar PV panels/modules designed to be installed over and supported by a roof, shall have the structural supports of the roof designed to accommodate the full dead load, including the Solar PV panels/modules dead load; the Roof/PV live load in the areas of the Solar PV panels/modules in combination with other applicable loads. The roof area underneath any Solar PV panels/modules shall also be designed for load combinations including roof live load, in combination with other applicable loads, without the Solar PV panels/modules.

1607.12.5.3 PV panels/modules installed as an independent structure. Solar PV panels/modules that are independent structures and do not have accessible /occupied space underneath are not required to accommodate a roof/PV live load, provided they are marked as required in Section 1607.12.5.1, and the area under the structure is restricted to keep the public away. All other loads and combinations per Section 1605 shall be accommodated.

Solar PV panels/modules that are designed to be the roof, and span to structural supports, and have accessible/occupied space underneath shall have the panels/modules and all supporting structure designed to support a Roof/PV live load, as defined in section 1607.12.5.1 in combination with other applicable loads. Solar PV panels/modules in this application are not permitted to be classified as "not accessible" per 1607.12.5.1.

1607.12.5.4 Ballasted systems. Solar PV panels/modules installed on a roof as a ballasted system need not be rigidly attached to the roof or supporting structure. Ballasted systems shall be designed and installed only on roofs with slopes of ½" per foot or less. The structural supports of the roof under a ballasted system shall be designed, or analyzed, per section 1604.4; checked in accordance with Section 1604.3.6 for deflections; and checked in accordance with Section 1611 for ponding. The ballasted system shall be designed to resist sliding and uplift resulting from lateral and vertical forces as required by Section 1605, using a coefficient of friction determined by acceptable engineering principles.

Reason: This new section is bringing in requirements for Solar PV panels that is currently absent in the code.

S72-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
2				1603.1.8.1 (NEW)-S-HUSTON

S73–12 1603.1.9

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Delete without substitution:

1603.1.9 Systems and components requiring special inspections for seismic resistance. Construction documents or specifications shall be prepared for those systems and components requiring special inspection for seismic resistance as specified in Section 1705.11 by the *registered design* **professional responsible for their design and shall be submitted for approval in accordance with Section 107.1. Reference to seismic standards in lieu of detailed drawings is acceptable.**

Reason: Section 1603.1.9 is being deleted because it serves no purpose not already being served by Section 107.1, which requires construction documents that are submitted with each permit application to be prepared by a registered design professional but only where required by the statutes of the jurisdiction in which the construction or work is located. Section 1603.1.9, however, requires preparation of construction documents or specifications by the registered design professional responsible for the design of the system or component and references Section 107.1 for the submittal, but not the preparation, of the construction documents.

The deletion also eliminates a conflict with the charging language in Section 1603.1, which requires design loads and other information pertinent to the structural design to be specified on the construction documents. Section 1603.1.9, however, specifies no such design loads or other pertinent information.

S73-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1603.1.9-S-BRAZIL.doc

S74-12 Table 1604.3

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee - General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

DEFLECTION LIMITS ^{a,b,c,h,i}					
CONSTRUCTION	L	S or W ^t	D + L ^{d,g}		
Roof Members: ^e					
Supporting plaster ceiling	// 360	// 360	// 240		
Supporting plaster ceiling	//240	// 240	// 180		
Not supporting ceiling	// 180	// 180	// 120		
Floor Members	// 360	-	// 240		
Exterior walls and interior partitions:					
With plaster or stucco finishes	-	// 360	-		
With other brittle finishes	-	// 240	-		
With flexible finishes	-	// 120	-		
Farm buildings	-	-	// 180		
Greenhouses	-	-	// 120		

TABLE 1604.3

Interior partitions not exceeding 6ft in height and Flexible, folding and portable partitions are not governed by the provisions of b. this section. The deflection criterion for interior partitions is based on the horizontal load defined in section 1607.14.

(Portions of Table not shown remain unchanged)

Reason: In footnote b the reference to interior partitions not exceeding 6ft in height is redundant and not needed. The second sentence of the footnote refers the user to Section 1607.14 (attached to the proposed change for reference) which already limits the live loading to partitions exceeding 6 feet.

S74-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				T1604.3#1-S-HUSTON.doc

S75–12 Table 1604.3, 1607.14, 1607.14.1

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

DEFLECTION LIMITS ^{a, b, c, h, i}					
CONSTRUCTION	L	S or W [†]	D + L ^{a,g}		
Roof Members: ^e					
Supporting plaster ceiling	// 360	// 360	// 240		
Supporting plaster ceiling	// 240	// 240	// 180		
Not supporting ceiling	// 180	// 180	// 120		
Floor Members	// 360	-	// 240		
Exterior walls and interior partitions:					
With plaster or stucco finishes	-	// 360	-		
With other brittle finishes	-	// 240	-		
With flexible finishes	-	// 120	-		
Interior Partitions: ^b					
With plaster or stucco finishes	<u>// 360</u>	<u>-</u>	<u>-</u>		
With other brittle finishes	<u>//240</u>	<u>-</u>	<u>-</u>		
With flexible finishes	<u>// 120</u>	=	<u>-</u>		
Farm buildings	-	-	//180		
Greenhouses	-	-	//120		

TABLE 1604.3 DEFLECTION LIMITS^{a, b, c, h, i}

(Portions of Table not shown remain unchanged)

1607.14 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength <u>and stiffness</u> to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m2).

Exception: Fabric partitions complying with Section 1607.14.1 shall not be required to resist the minimum horizontal load of 5 psf (0.24 kN/m2).

1607.14.1 Fabric partitions. Fabric partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength <u>and stiffness</u> to resist the following load conditions:

- 1. A horizontal distributed load of 5 psf (0.24 kN/m2) applied to the partition framing. The total area used to determine the distributed load shall be the area of the fabric face between the framing members to which the fabric is attached. The total distributed load shall be uniformly applied to such framing members in proportion to the length of each member.
- A concentrated load of 40 pounds (0.176 kN) applied to an 8-inch diameter (203 mm) area [50.3 square inches (32 452 mm2)] of the fabric face at a height of 54 inches (1372 mm) above the floor.

Reason: Currently Table 1604.3 does not have deflection limits for Live Loads on Interior walls. The 5.0psf requirement in section 1607.14 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 deflection limits similar 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.

S75-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				T1604.3#2-S-HUSTON.doc

S76–12 Table 1604.3

Proponent: Brad Douglas, PE, American Wood Council (pcoats@awc.org)

Revise as follows:

TABLE 1604.3DEFLECTION LIMITS^{a, b, c, h, i}

(Portions of Table and footnotes not shown remain unchanged)

d For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry conditions, the deflection resulting from L + 0.5D is permitted to be substituted for the deflection resulting from L + D.

d. The deflection limit for the *D+L* load combination only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection. For wood structural members that are dry at time of installation and used under dry conditions, the creep component of the long-term deflection shall be permitted to be estimated as the immediate dead load deflection resulting from 0.5D. For wood structural members at all other moisture conditions, the creep component of the long-term deflection as the immediate dead load deflection is permitted to be estimated as the immediate dead load deflection is permitted to be estimated as the immediate dead load deflection resulting from *D*. The value of 0.5D shall not be used in combination with NDS provisions for long-term loading.

Reason: Deflection limits for the load combination D+L, were taken from the UBC deflection limits. However, the intent of the UBC limits was not brought forward. The original intent of these provisions was to limit the total deflection based on the combination of live load deflection and the creep component of the dead load deflection. As a result, there have been several prior code cycle modifications to these provisions to re-instate the original intent, such as the addition of footnote "g" for steel structural members which effectively excludes steel from checking for the creep component of dead load deflection. As currently written and formatted, the D+L deflection provision can be misinterpreted to suggest that the total deflection due to dead load, D, including both the immediate and creep components of the dead load deflection, should be used with the deflection limit in this column. Additionally, use of 0.5D in footnote "d" is potentially non-conservative without clarification, a potential misinterpretation is that the creep component of dead load deflection is to be calculated using NDS provisions and the reduced dead load (i.e. 0.5D). This change makes calculation of D+L deflection for comparison against the D+L deflection limit in Table 1604.3 consistent with the provisions in NDS 3.5.2 for long-term loading and consistent with the stated intent in the UBC and with similar provisions in ACI 318 as described in the ACI 318 Commentary. The applicable NDS provisions are shown below for reference.

NDS 3.5.2 Long-Term Loading:

3.5 Bending Members - Deflection

3.5.1 Deflection Calculations

If deflection is a factor in design, it shall be calculated by standard methods of engineering mechanics considering bending deflections and, when applicable, shear deflections. Consideration for shear deflection is required when the reference modulus of elasticity has not been adjusted to include the effects of shear deflection (see Appendix F).

3.5.2 Long-Term Loading

Where total deflection under long-term loading must be limited, increasing member size is one way to

- 2.0 for structural glued laminated timber used in wet service conditions as defined in 5.1.4.
- = 2.0 for wood structural panels used in dry service conditions as defined in 9.1.4.

provide extra stiffness to allow for this time dependent deformation (see Appendix F). Total deflection, Δ_T , shall be calculated as follows: $\Delta_\tau = K_{\rm er} \Delta_{L\tau} + \Delta_{\rm s\tau} \qquad (3.5\text{-}1)$

where:

- $K_{\rm er}$ = time dependent deformation (creep) factor
 - = 1.5 for seasoned lumber, structural glued laminated timber, prefabricated wood Ijoists, or structural composite lumber used in dry service conditions as defined in 4.1.4, 5.1.4, 7.1.4, and 8.1.4, respectively.
- = 2.0 for unseasoned lumber or for seasoned lumber used in wet service conditions as defined in 4.1.4.
- $$\label{eq:Lagrangian} \begin{split} \Delta_{c\tau} &= \text{immediate deflection due to the long-term} \\ & \text{component of the design load, in.} \end{split}$$
- $$\label{eq:definition} \begin{split} \Delta_{s\tau} = & \text{deflection} \mbox{ due to the short-term or normal} \\ & \text{component of the design load, in.} \end{split}$$

S76-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				T1604.3-S-DOUGLAS.doc

S77-12 Table 1604.3

Proponent: John Woestman, Kellen Company, representing Builders Masonry Veneer Manufacturers Association (MVMA) (jwoestman@kellencompany.com)

Revise as follows:

CONSTRUCTION	L	S OR W ^t	D + L ^{a, g}			
Exterior walls and interior partitions:						
With plaster or stucco finishes		//360				
With other brittle finishes ¹		//240				
With flexible finishes		//120				
Includes adhered masonry veneer.						

TABLE 1604.3 DEFLECTION LIMITS^{a,b,c,h, i}

(Portions of Table not shown remain unchanged)

Reason: This code proposal should help with a consistent deflection limit applied to wall systems with adhered masonry veneer. Adhered masonry veneer does not have the large, flat, monolithic surface of plaster or stucco finishes. As such, adhered masonry veneer can accommodate more deflection.

S77-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				T1604.3-S-WOESTMAN.doc

S78–12 Table 1604.3

Proponent: Thomas S. Zaremba, Roetzel & Andress, representing Glazing Industry Code Committee (tzaremba@ralaw.com)

TABLE 1604.3

Revise as follows:

DEFLECTION LIMITS ^{a, b, c, h, i}						
CONSTRUCTION	L	S or W	D + L ^{d, g}			
Roof members: ^e						
Supporting plaster or stucco ceiling	1/360	1/360	1/240			
Supporting nonplaster ceiling	1/240	1/240	1/180			
Not supporting ceiling	1/180	1/180	1/120			
Floor members	1/360		1/240			
Exterior walls and interior partitions:						
With plaster or stucco finishes		1/360				
With other brittle finishes		1/240				
With flexible finishes		1/120				
Farm buildings			1/180			
Greenhouses			1/120			

For SI: 1 foot = 304.8 mm.

For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed 1/60. For а. secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed 1/150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed 1/90. For roofs, this exception only applies when the metal sheets have no roof covering.

Interior partitions not exceeding 6 feet in height and flexible, folding and portable partitions are not governed by the provisions b. of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.14.

See Section 2403 for glass supports. C.

For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry d conditions, the deflection resulting from L + 0.5D is permitted to be substituted for the deflection resulting from L + D.

- The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate e. drainage shall be investigated for ponding. See Section 1611 for rain and ponding requirements and Section 1503.4 for roof drainage requirements.
- W shall be taken as the nominal load for wind. The wind load is permitted to be taken as 0.42 times the "component and f. cladding" loads for the purpose of determining deflection limits herein for main windforce-resisting systems.

For steel structural members, the dead load shall be taken as zero.

For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers, not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed 1/60. For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed 1/175 for each glass lite or 1/60 for the entire length of the member, whichever is more stringent. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed 1/120.

For cantilever members, 1 shall be taken as twice the length of the cantilever. i.

Reason: Section 1605.2 of the IBC provides load combinations using strength or factored load design while 1605.3 provides load combinations using allowable stress design. For wind load determination, ASCE 7-10 is now based on strength design and ultimate wind speeds. Conversion to allowable working stress (nominal) loads is accomplished by multiplying the factored wind load by 0.6.

The original reduction of 0.7 in footnote (f) in earlier editions of the IBC was changed last cycle by multiplying 0.7 and 0.6 to get the 0.42 factor now shown in the current code. The added sentence to the beginning of the footnote is a clarification to use nominal loads for wind used in allowable stress design to determine deflection which corresponds to the use of the 0.42 reduction.

A second clarification is also added at the end of footnote (f) to properly restrict the use of the 0.42 reduction to deflection calculations of main structural members shown in the Table 1604.3 when using "component and cladding wind load and preventing users from inappropriately combining this reduction with component and cladding deflections described in footnotes (a), (c), and (h) of this table.

S78-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					T1604.3-S-ZAREMBA.doc

S79–12 202, 1602.1, 1604.4, 1610.1 1613.5.6.1

Proponent: Charles S. Bajnai, Chesterfield County, VA, ICC Building Code Action Committee

Delete without substitution:

SECTION 202 DEFINITIONS

DIAPHRAGM. A horizontal or sloped system acting to transmit lateral forces to the vertical-resisting elements. When the term "diaphragm" is used, it shall include horizontal bracing systems.

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.

Diaphragm, **rigid**. A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift.

Revise as follows:

SECTION 1602 DEFINITIONS AND NOTATIONS

1602.1 Definitions. The following terms are defined in Chapter 2:

DIAPHRAGM. Diaphragm, blocked. Diaphragm boundary. Diaphragm chord. Diaphragm flexible. Diaphragm, rigid.

(Portions of text not shown remains unchanged)

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 that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.

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, except where diaphragms are considered as flexible, permitted to be idealized as flexible or semi-rigid, in accordance with Section 12.3.1 of ASCE for seismic loads or Chapter 26 of ASCE 7 for wind loads.

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.

1610.1 General. Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless determined otherwise by a geotechnical investigation in accordance with Section 1803. Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1805.4.2 and 1805.4.3.

Exception: Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by flexible diaphragms considered as flexible, permitted to be idealized as flexible or semi-rigid, in accordance with Section 12.3.1 of ASCE for seismic loads or Chapter 26 of ASCE for wind loads shall be permitted to be designed for active pressure.

1613.3.5.1 Alternative seismic design category determination. Where S_1 is less than 0.75, the *seismic design category* is permitted to be determined from Table 1613.3.5(1) alone when all of the following apply:

- 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 <u>considered</u> flexible, <u>permitted to be idealized as flexible or semi-rigid in accordance with Section</u> <u>12.3.1 of ASCE 7</u>, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

Reason: The ICC Building Code Action Committee was asked to look at clearing up potential conflicts between the references to, and definitions of, flexible and rigid diaphragms in the IBC and ASCE-7-10. The BCAC did identify potential conflicts between the IBC's definition of a rigid diaphragm and the ASCE 7-10 criteria for classifying a diaphragm as rigid, semi-rigid or flexible. Also, it is considered inappropriate to include enforceable code requirements or references to standards as part of a definition. Thus, by this proposal, the BCAC proposes to remove the separate definitions for flexible and rigid diaphragms from the IBC and supply direct references in IBC Chapter 16 to the relevant requirements in the ASCE 7 seismic and wind chapters for when a diaphragm can be idealized as flexible or semi-rigid. This reference only occurs in the IBC in the sections noted in the code change proposal. In practical application, the code user will be turning to the requirements of ASCE-7 to categorize the diaphragm and perform the design. Therefore, there is no real need or advantage to provide the definitions in the IBC and this will prevent future maintenance of the terms and/or conflict between them.

For reference, ASCE 7-10 states,

12.3.1 Diaphragm Flexibility

The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic forceresisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).

12.3.1.1 Flexible Diaphragm Condition

Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

a. In structures where the vertical elements are steel braced frames, steel and concrete composite braced frames or concrete, masonry, steel, or steel and concrete composite shear walls.

- b. In one-and two-family dwellings.
- c. In structures of light-frame construction where all of the following conditions are met:

1. Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 ½" in (38mm) thick.

2. Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift of Table 12.12-1

12.3.1.2 Rigid Diaphragm Condition

Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.

12.3.1.3 Calculated Flexible Diaphragm Condition

Diaphragms not satisfying the conditions of Sections 12.3.1.1 or 12.3.1.2 are permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Fig. 12.3-1. The loadings used for this calculation shall be those prescribed by Section 12.8.

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

S79-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
· · · · ·				1604.4-S-BAJNAI-BCAC.doc

S80–12 202, 1604.4, 1810.3.1.4, 1810.3.1.5, 1908.2, Table 1908.2 and Table 1908.3

Proponent: Jerry R. Tepe, FAIA, JRT-AIA, representing American Institute of Architects (jrtaia@aol.com)

Revise as follows:

SECTION 202 DEFINITIONS

DANGEROUS. Any *building*, *structure* or portion thereof that meets any of the conditions described below shall be deemed dangerous:

- 1. The *building* or *structure* has collapsed, has partially collapsed, has moved off its foundation or lacks the necessary support of the ground.
- 2. There exists a significant risk of collapse, detachment or dislodgment of any portion, member, appurtenance or ornamentation of the *building* or *structure* under service nominal loads.

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 <u>nominal</u> 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 that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.

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.

Revise as follows:

1810.3.1.4 Driven piles. Driven piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service nominal loads.

1810.3.1.5 Helical piles. Helical piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by installation into the ground and service nominal loads.

Revise as follows:

1908.2 Allowable service load. The allowable service <u>nominal</u> load for headed anchors in shear or tension shall be as indicated in Table 1908.2. Where anchors are subject to combined shear and tension, the following relationship shall be satisfied:

$$(P_s / P_t) 5/3 + (V_s / V_t)^{5/3} \le 1$$

(Equation 19-1)

where:

 P_{s} = Applied tension service <u>nominal</u> load, pounds (N).

 P_t = Allowable tension service <u>nominal</u> load from Table 1908.2, pounds (N).

 V_s = Applied shear service <u>nominal</u> load, pounds (N).

 V_t = Allowable shear service <u>nominal</u> load from Table 1908.2, pounds (N).

TABLE 1908.2

ALLOWABLE SERVICE NOMINAL LOAD ON EMBEDDED BOLTS (pounds)

(Portions of table not shown remain unchanged)

1908.3 Required edge distance and spacing. The allowable service nominal loads in tension and shear specified in Table 1908.2 are for the edge distance and spacing specified. The edge distance and spacing are permitted to be reduced to 50 percent of the values specified with an equal reduction in allowable service nominal load. Where edge distance and spacing are reduced less than 50 percent, the allowable service nominal load shall be determined by linear interpolation.

Reason: "Nominal loads" is a defined term whereas "service loads" is not. Per IBC Interpretation 23-10 issued 12-08-2010, the terms are synonymous. (Note interpretation was from the 2009 edition) Dangerous is used in Chapter 34. The intent is to make this change wherever it occurs in the IBC. What is shown was derived from a word search of the PDF document.

IBC Interpretation 23-10

Q: Is the term "service loads" as used in the definition of DANGEROUS synonymous with the definition of NOMINAL LOADS as defined in Section 1602? A: Yes. The intent is to address loads that a building is likely to experience and precludes consideration of a

FACTORED LOAD which applies to limit state or strength design.

NOMINAL LOADS. The magnitudes of the *loads* specified in Chapter 16 (dead, live, soil, wind, snow, rain, *flood* and earthquake). FACTORED LOAD. The product of a *nominal load* and a *load factor*.

S80-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					202 DANCEDOUS C TEDE

S81–12 1604.5.1

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

1604.5.1 Multiple occupancies. Where a building or structure is occupied by two or more occupancies not included in the same *risk category*, it shall be assigned the classification of the highest *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 *risk category*, both portions shall be assigned to the higher *risk category*.

Exception: A single public assembly room with an occupant load of less than 500 shall be allowed in a *Risk Category II* building or structure and not be considered a multiple occupancy or a separate occupancy.

Reason: The revision to 1604.5.1 will allow a single, modest meeting room or auditorium within an office building (a Risk Category II Building) without requiring the entire building to be designed as a Risk Category III.

S81-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				1604.5.1-S-HUSTON

S82–12 1604.5

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithustoninc.com)

Revise as follows:

1604.5 Risk category. Each building and structure shall be assigned 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. <u>Where a referenced standard specifies</u> that the assignment of a risk category be in accordance with ASCE 7, Table 1.5-1, Table 1604.5 shall be used in lieu of ASCE 7, Table 1.5-1.

Reason: IBC Table 1604.5 has a concise and extensive list of various occupancies, whereas ASCE 7, Table 1.5-1 is limited and , being a standard, rather than a code, much more general. This can lead to confusion in the appropriate determination of a risk category, if one tries to comply with both.

As examples of when one can be referred to both tables, consider:

1. IBC Section 1609 Wind Loads requires wind loads to be determined in accordance with ASCE 7, chapters 26 thru 30. The confusion comes in when you are in those chapters of ASCE 7, risk categories per Table 1.5-1 are referenced (26.5.1; Table 27.5-1; Table 28.2-1; Table 29.1-1; Tables 30.4-1 thru 30.7-1).

2. AISC 360-10, Section N5.5b also references ASCE 7 Table 1.5-1 as follows:

5b. CJP Groove Weld NDT

For structures in Risk Category III or IV of Table 1.5-1, Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake and Ice Loads, of ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures*, UT shall be performed by QA on all CJP groove welds subject to transversely applied tension loading in butt, T- and corner joints, in materials 5/16 in. (8 mm) thick or greater. For structures in Risk Category II, UT shall be performed by QA on 10% of CJP groove welds in butt, T- and corner joints subject to transversely applied tension loading, in materials 5/16 in. (8 mm) thick or greater.

This code change is intended to provide consistency by using only IBC Table 1604.5.

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

Analysis: Does IBC Section 102.4.1 already provide sufficient clarification?

S82-12

Public Hearing: Commit	tee: AS	AM	D	
Assemb	ly: ASF	AMF	DF	
	-			1604.5-S-HUSTON

S83–12 Table 1604.5

Proponent: William W. Stewart, FAIA, representing self (codedoc@sbcglobal.net)

Revise as follows:

RISK CATEGORYNATURE OF OCCUPANCYIIIBuildings 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 Group E occupancies with an occupant load greater than 250. • Buildings and other structures containing adult education facilities, such as colleges and universities, educational occupancies for students above the 12th grade with an occupant load greater than 500. • Group I-2 occupancies with an occupant load of 50 or more resident care recipients 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 structures not included in Risk Category IV. • Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive 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 pose a threat to the public if released ^b .	R	RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES
 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 Group E occupancies with an occupant load greater than 250. Buildings and other structures containing adult education facilities, such as colleges and universities, educational occupancies for students above the 12th grade with an occupant load greater than 500. Group I-2 occupancies with an occupant load of 50 or more resident care recipients 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 Risk Category IV. Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive 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 <i>International Fire Code</i>; and Are sufficient to pose a threat to the public if released ^b. 	RISK CATEGORY	NATURE OF OCCUPANCY
		 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 Group E occupancies with an occupant load greater than 250. Buildings and other structures containing adult education facilities, such as colleges and universities, educational occupancies for students above the 12th grade with an occupant load greater than 500. Group I-2 occupancies with an occupant load of 50 or more resident care recipients 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 Risk Category IV. Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive 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 <i>International Fire Code</i>; and Are sufficient to pose a threat to the public if released ^b.

TABLE 1604.5 RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

(Portions of table not shown remain unchanged)

Reason: Consistency. The laundry list in the second bullet is exactly the same as the entire list of items that make up E Occupancies in 305. This just substitutes a defined term for a laundry list. My change has the added advantage of making it clearer that the 250 occupant load trigger applies to all, not just day care facilities.

The change in bullet 3 uses the words from 304. Current text says the same thing as in 304 but uses different words.. Additionally it relieves the code from deciding which college freshmen are adults.

This change also makes it clear that trade schools are covered .

S83-11					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					T1604.5-S-STEWART

S84–12 1604.8.2

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB) (gehrlich@nahb.org)

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 and members that provide lateral support for the wall or that are supported by the wall. The connections shall be capable of resisting the horizontal forces specified in Section 1.4.4 of ASCE 7 for walls of structures assigned to Seismic Design Category A and to Section 12.11 of ASCE 7 for walls of structures assigned to all other seismic design categories. Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609 for wind design requirements and 1613 for earthquake design requirements.

Exceptions:

- <u>1. In Risk Category I and II buildings or structures, connections for light-frame wood or cold-formed steel walls not exceeding 15 pounds per square foot (718 N/m2) in weight designed and constructed in accordance with Section 2304.9, Section 2308, or Section 2210.7 shall be exempt from the provisions of this section.</u>
- 2. In Risk Category I and II buildings or structures assigned to Seismic Design Category A, B, or C, connections for light-frame wood or cold-formed steel walls with stone or masonry veneer not exceeding 48 pounds per square foot (2298 N/m2) in weight designed and constructed in accordance with Section 2304.9, Section 2308, or Section 2210.7, shall be exempt from the provisions of this section.

Reason: The purpose of this amendment is to supply exceptions to the new wall anchorage provisions added in ASCE 7-10 and the 2012 IBC. These new provisions were much needed, and we supported their inclusion in ASCE 7. However, during the ASCE 7-10 development process the provisions were expanded to apply to all bearing walls including light frame walls. The result is to impose an unnecessary and unjustified light-frame wall design check on already-overburdened engineers. We are concerned this will be a "nuisance" provision; glossed over until the code official or peer reviewer calls an engineer on it. We are also concerned engineers using prescriptive fastener schedules such as those in the IBC, WFCM or COFS-PM will be asked to justify them. This amendment supplies two exemptions: (1) for light-frame walls less than 15psf in weight in any seismic design category; and (2) for veneered walls less than 48 psf in weight in seismic design categories A, B and C. This will reduce burdens on engineers and code officials applying the new provisions.

S84-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1604 8 2-S-EHRLICH doc

S85–12 1604.11 (NEW), 1604.11.1 (NEW), 1604.11.2 (NEW), 1615

Proponent: Charles S. Bajnai, Chesterfield County, VA, ICC Building Code Action Committee

Revise as follows:

1604.11 Structural integrity. Structural integrity for buildings and other structures shall be provided in accordance with this section and shall not be less than specific applicable requirements elsewhere in this code.

1604.11.1 General. Buildings and other structures shall comply with Sections 1.4 through 1.4.5 of ASCE 7.

Exceptions:

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

1604.11.2 High rise buildings classified as Risk Category III or IV. In addition to the requirements of Section 1604.11.1 high-rise buildings that are classified as Risk Category III or IV shall comply with Section 1615.

SECTION 1615 STRUCTURAL INTEGRITY OF HIGH-RISE RISK CATEGORY III AND IV BUILDINGS

(Portions of text not shown remains unchanged)

Reason: Since the IBC drafting stages, attempts have been made to add minimum general structural integrity requirements based on ASCE 7 Section 1.4. Those attempts have been rejected because the ASCE 7 provisions of Section 1.4 were considered to be unenforceable. With concerns that have been raised over requiring minimum general structural integrity, it was recognized that the Seismic Design Category (SDC) A requirements under earthquake loads constitute a "de facto" set of minimum structural integrity requirements that all structures must meet. Those minimum requirements would be exceeded in the case of higher seismic design categories.

The 2010 edition of ASCE 7 has, in fact, relocated the seismic design requirements for SDC A to Section 1.4 of the standard which is titled "General Structural Integrity". Section 1.4 of ASCE 7 is then referenced by Section 11.7 of the standard for minimum earthquake load and detailing requirements in SDC A. Section 11.7 states,

11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

Buildings and other structures assigned to Seismic Design Category A need only comply with the requirements of Section 1.4. Nonstructural components in SDC A are exempt from seismic design requirements. In addition, tanks assigned to Risk Category IV shall satisfy the freeboard requirement in Section 15.7.6.1.2

The ASCE 7 seismic loading requirements are applied by reference from IBC Section 1613.1, "EARTHQUAKE LOADS". The intent is to ensure minimum structural design criteria by applying Section 1.4 of ASCE 7 to structures that are classified as SDC A under the 2012 IBC.

The proposed scope of reference does not include Section 1.4.6 of ASCE 7 which deals with extraordinary loads and events for which no specific criteria are provided.

Since the structural integrity requirements in Section 1.4 of ASCE 7 are implemented via the earthquake load provisions, the four exceptions currently in Section 1613.1 are copied here, verbatim, as Exceptions 2 through 5. This is done to avoid any unintended technical changes. In 1604.11.2 a cross-reference is added to the structural integrity requirements that are currently in Section 1615.

Since the new proposed section 1604.11 is titled "STRUCTURAL INTEGRITY", the title of the existing section 1615, "STRUCTURAL INTEGRITY", is changed to reflect the specific scope of that section which is High-rise risk category III and IV

S85-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1604.11 (NEW)-S-BAJNAI-BCAC.doc

S86–12 1605.2

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

1605.2 Load combinations using strength design or load and resistance factor design. Where strength design or load and resistance factor design is used, buildings and other structures, and portions thereof, shall be designed to resist the most critical effects resulting from the following combinations of factored loads:

 $\begin{array}{l} 1.4(D + F) \\ 1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \\ 1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1L \text{ or } 0.5W) \\ 1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R) \\ 1.2(D + F) + 1.0E + f_1L + 1.6H + f_2S \\ 0.9D + 1.0W + 1.6H \\ 0.9(D + F) + 1.0E + 1.6H \end{array}$

(Equation 16-1) (Equation 16-2) (Equation 16-3) (Equation 16-4) (Equation 16-5) (Equation 16-6) (Equation 16-7)

where:

- $f_1 = 1$ for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/m2), and parking garages; 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 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.
- 3. Crane wheel 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. Alternatively, industry standard reference documents citing additional crane load combinations shall be permitted for the design of buildings subject to horizontal and vertical crane loads.

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 D + H + F + L D + H + F + (Lr or S or R) $D + H + F + 0.75(L) + 0.75(L_r \text{ or } S \text{ or } R)$ D + H + F + (0.6W or 0.7E) $D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ D + H + F + 0.75 (0.7 E) + 0.75 L + 0.75 S 0.6D + 0.6W + H0.6(D + F) + 0.7E + H (Equation 16-8) (Equation 16-9) (Equation 16-10) (Equation 16-11) (Equation 16-12) (Equation 16-13) (Equation 16-14) (Equation 16-15) (Equation 16-16)

Exceptions:

- Crane hook wheel loads need not be combined with roof live load or with more than threefourths of the snow load or one-half of the wind load. <u>Alternatively, industry standard</u> reference documents citing additional crane load combinations shall be permitted for the design of buildings subject to horizontal and vertical crane loads.
- Flat roof snow loads of 30 psf (1.44 kN/m2) or less and roof live loads of 30 psf (1.44 kN/m2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m2), 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.
- 4. In Equation 16-15, the wind load, *W*, is permitted to be reduced in accordance with Exception 2 of Section 2.4.1 of ASCE 7.
- 5. In Equation 16-16, 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.

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. When using allowable tresses which have been increased or load combinations which have been reduced as permitted by the material chapter of this code or the referenced standards, where wind loads are calculated in accordance with Chapters 26 through 31 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. When allowable stresses have not been increased or load combinations have not been reduced as permitted by the material chapter of this code or the referenced standards, (ω) 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 , in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

$D + L + (L_r \text{ or } S \text{ or } R)$
$D + L + 0.6 \omega W$
$D + L + 0.6 \omega W + S/2$
$D + L + S + 0.6 \omega W/2$
<i>D</i> + <i>L</i> + <i>S</i> + <i>E</i> /1.4
0.9 <i>D</i> + <i>E</i> /1.4

(Equation 16-17) (Equation 16-18) (Equation 16-19) (Equation 16-20) (Equation 16-21) (Equation 16-22)

Exceptions:

- Crane hook wheel loads need not be combined with roof live loads or with more than threefourths of the snow load or one-half of the wind load. <u>Alternatively, industry standard</u> reference documents citing additional crane load combinations shall be permitted for the design of buildings subject to horizontal and vertical crane loads.
- Flat roof snow loads of 30 psf (1.44 kN/m2) or less and roof live loads of 30 psf (1.44 kN/m2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m2), 20 percent shall be combined with seismic loads.

Reason: Current code language does not completely or adequately address the issue of load combinations for the design of buildings with bridge cranes. This includes buildings and other structures that have multiple crane runways adjacent to one another and/or multiple cranes on the same runway. An exception pointing to industry standard reference documents, such as the Association of Iron and Steel Technology (AIST) "Technical Report No. 13 - Guide for the Design and Construction of Mill Buildings", allows the engineer to utilize such resources when determining additional load combinations that may control in the design of such buildings.
S86-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	•				1605.2-S-HUSTON

S87–12 202, Table 1607.1

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB) (gehrlich@nahb.org)

Delete without substitution:

SECTION 202 DEFINITIONS

MARQUEE. A canopy that has a top surface which is sloped less than 25 degrees from the horizontal and is located less than 10 feet (3.05 m) from operable openings above or adjacent to the level of the marquee.

Revise as follows:

TABLE 1607.1 MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS⁹

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
21. Marquees	75	-
26. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies:		
Fabric construction supported by a	5	
skeleton structure	Nonreducible	
All other construction	20 "	
Ordinary flat, pitched, and curved roofs	20	
(that are not occupiable)		
Where primary roof members are exposed		
to a work floor, at single panel point of		
lower chord of roof trusses or any point		
along primary structural members		
supporting roots:		
Over manufacturing, storage		0.000
warenouses, and repair garages		2,000
All other primary root members		300
	100	
Accomply cross	100	
Assembly aleas	IUU Note 1	Note 1
n Where a canony has a top surface sloped less than 25 d	INOLE I	I INULE I

Where a canopy has a top surface sloped less than 25 degrees from the horizontal and is located less than 10 feet (3.05 m) from operable openings above or adjacent to the level of the canopy, the minimum live load shall be taken as the live load of the adjacent room or space, but not less than 40psf. The maximum live load for canopies less than or equal to 100 square feet in area shall be 60psf.

(Portions of Table and footnotes not shown remain unchanged)

Reason: The purpose of this amendment is to revise the 2012 IBC language regarding canopies and marquees. The language approved for the 2012 IBC will substantially change the design requirements for many small porch and patio roofs on buildings nowhere near public streets. These roofs are currently designed for standard roof live loads or local ground snow loads (typically in the range of 20 or 30 pounds per square foot). These elements will now need to be designed for 75psf if they happen to be less than 10 feet vertically from a window above or horizontally from a window at the level of the canopy. This represents a substantial increase in design requirements for apartment or condominium complexes with these elements, as well as a substantial issue for renovations. This change deletes the definition for marquees in its entirety and transfers the language regarding canopy slope and

ability to access the top surface from nearby openings to a footnote on the standard canopy live load. It also requires the window to be operable. The live load for the accessible canopy condition is set to the adjacent occupancy, with a minimum floor of 40psf (equivalent to the traditional load for a residential deck). To avoid effectively further raising the live load requirement from 75psf to 100psf for a small canopy accessible from an egress hallway or stair, a maximum live load of 60psf is established for canopies not exceeding 100 square feet in area (similar to what the traditional load cases were for residential balconies).

S87-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				T1607.1-S-EHRLICH.doc

S88–12 Table 1607.1

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (Huston@smithhustoninc.com)

Revise as follows:

TABLE 1607.1 MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS⁹

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs)
24.Recreational uses:		
Ice Skating Rink	<u>250^m</u>	See Section 1607.7.4
Roller Skating Rink	<u>100^m</u>	

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

(Portions of Table and footnotes not shown remain unchanged)

Reason: : Uniformly distributed live load for rinks were in previous editions of the IBC. They were removed from the IBC 2009, as part of a larger CCP. The intent of this code change proposal is to once again list the recommended minimum uniform live load for rinks back into IBC. The proposed loads are consistent with the recommendations in ASCE7 commentary for minimum uniformly distributed live load.

S88-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				T1607.1-S-HUSTON.doc

S89–12 1607.5

Proponent: Edwin Huston, National Council of Structural Engineers Associations- Code Advisory Committee - General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

1607.5 Partition loads. In office buildings and in other buildings where partition locations are subject to change, provisions for partition weight shall be made, whether or not partitions are shown on the *construction documents*, unless the specified live load exceeds is 80 psf (3.83 kN/m^2) or greater. The partition load shall not be less than a uniformly distributed live load of 15 psf (0.72 kN/m^2).

Reason: IBC Table 1607.1, item #22 requires a live load of 80 psf for corridors above the first floor. It is a common practice to design an entire floor for an 80 psf live load, and thereby not need to worry about the locations of the corridors, or whether the corridor locations may be moved in the future. The way the code is written now, a floor would have to be designed for a live load of 81 psf (it must "exceed" 80 psf) to be able to take advantage of the exception written into section 1607.5. Otherwise one has to add a 15 psf partition load on top of an 80 psf corridor live load.

This change does not alter the requirements of ASCE 7, section 12.7.2 Effective Seismic Weight, #2 (the greater of 10 psf or the actual weight of the partitions must be used for calculating the seismic weight of a building).

S89-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1607.5-S-HUSTON.doc

S90–12 901.5, 1004.3, 1607.7.2, 1703.4.1, 1703.6, 1703.6.1, 17042, 1704.2.4, 1704.4, 1704.5, 1707.1, 1803.6, 2211.3.3, 2303.4.1.3, 3306.8, 3401.2, G104.1, J106.1, K102.3

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

THIS IS A FOUR PART CODE CHANGE. ALL PARTS WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. SEE TENTATIVE HEARING ORDER FOR THIS COMMITTEE

PART I – IBC STRUCTURAL

Revise as follows:

1607.7.5 Posting. The maximum weight of the vehicles allowed into or on a garage or other structure shall be posted by the owner <u>or the owner's authorized agent</u> in accordance with Section 106.1.

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 applicant owner or the owner's authorized agent.

1703.6 Evaluation and follow-up inspection services. Where structural components or other items regulated by this code are not visible for inspection after completion of a prefabricated assembly, the applicant owner or the owner's authorize agent shall submit a report of each prefabricated assembly. The report shall indicate the complete details of the assembly, including a description of the assembly and its components, the basis upon which the assembly is being evaluated, test results and similar information and other data as necessary for the *building official* to determine conformance to this code. Such a report shall be *approved* by the *building official*.

1703.6.1 Follow-up inspection. The applicant <u>owner or the owner's authorized agent</u> shall provide for *special inspections* of fabricated items in accordance with Section 1704.2.5.

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 <u>authorized</u> agent shall employ one or more *approved agencies* to perform inspections during construction on the types of work listed under Section 1705. These inspections are in addition to the inspections identified in Section 110.

Exceptions:

- 1. *Special inspections* are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as *approved* by the *building official*.
- 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.
- 3. Special inspections are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.

1704.2.4 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 <u>owner or the owner's authorized agent to</u> the *building official*.

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 <u>or the owner's authorized agent</u>, 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*.

1704.5 Structural observations. Where required by the provisions of Section 1704.5.1 or 1704.5.2, the owner <u>or the owner's authorized agent</u> 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 that, to the best of the structural observer's knowledge, have not been resolved.

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 <u>owner or the owner's authorized agent</u>.

Revise as follows:

1803.6 Reporting. Where geotechnical investigations are required, a written report of the investigations shall be submitted to the *building official* by the owner or authorized agent permit applicant at the time of *permit* application. This geotechnical report shall include, but need not be limited to, the following information:

- 1. A plot showing the location of the soil investigations.
- 2. A complete record of the soil boring and penetration test logs and soil samples.
- 3. A record of the soil profile.
- 4. Elevation of the water table, if encountered.
- 5. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of expansive soils; mitigation of the effects of liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads.
- 6. Expected total and differential settlement.
- 7. Deep foundation information in accordance with Section 1803.5.5.
- 8. Special design and construction provisions for foundations of structures founded on expansive soils, as necessary.
- 9. Compacted fill material properties and testing in accordance with Section 1803.5.8.
- 10. Controlled low-strength material properties and testing in accordance with Section 1803.5.9.

Revise as follows:

2211.3.3 Trusses spanning 60 feet or greater. The owner <u>or the owner's authorized agent</u> shall contract with a *registered design professional* for the design of the temporary installation restraint/bracing and the permanent individual truss member restraint/ bracing for trusses with clear spans 60 feet (18 288 mm) or greater. *Special inspection* of trusses over 60 feet (18 288 mm) in length shall conform to Section 1705.

2303.4.1.3 Trusses spanning 60 feet or greater. The owner <u>or the owner's authorized agent</u> shall contract with any qualified *registered design professional* for the design of the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing for all trusses with clear spans 60 feet (18 288 mm) or greater.

G104.1 Required. Any person, owner or <u>owner's</u> authorized agent who intends to conduct any development in a flood hazard area shall first make application to the *building official* and shall obtain the required *permit*.

J106.1 Maximum slope. The slope of cut surfaces shall be no steeper than is safe for the intended use, and shall be no steeper than two units horizontal to one unit vertical (50-percent slope) unless the owner or <u>the owner's</u> authorized agent furnishes a geotechnical report justifying a steeper slope.

Exceptions:

- 1. A cut surface shall be permitted to be at a slope of 1.5 units horizontal to one unit vertical (67percent slope) provided that all of the following are met:
 - 1.1. It is not intended to support structures or surcharges.
 - 1.2. It is adequately protected against erosion.
 - 1.3. It is no more than 8 feet (2438 mm) in height.
 - 1.4. It is approved by the building code official.
 - 1.5. Ground water is not encountered.
- 2. A cut surface in bedrock shall be permitted to be at a slope of one unit horizontal to one unit vertical (100-percent slope).

K102.3 Maintenance. Electrical systems, equipment, materials and appurtenances, both existing and new, and parts thereof shall be maintained in proper operating condition in accordance with the original design and in a safe, hazard-free condition. Devices or safeguards that are required by this code shall be maintained in compliance with the code edition under which installed. The owner or the owner's designated authorized agent shall be responsible for the maintenance of the electrical systems and equipment. To determine compliance with this provision, the *building official* shall have the authority to require that the electrical systems and equipment be re-inspected.

PART II – IBC GENERAL

Revise as follows:

3306.8 Repair, maintenance and removal. Pedestrian protection required by this chapter shall be maintained in place and kept in good order for the entire length of time pedestrians are subject to being endangered. The *owner* or the *owner*'s <u>authorized</u> agent, upon the completion of the construction activity, shall immediately remove walkways, debris and other obstructions and leave such public property in as good a condition as it was before such work was commenced.

3401.2 Maintenance. Buildings and structures, and parts thereof, shall be maintained in a safe and sanitary condition. Devices or safeguards which are required by this code shall be maintained in conformance with the code edition under which installed. The owner or the owner's designated <u>authorized</u> agent shall be responsible for the maintenance of buildings and structures. To determine compliance with this subsection, the *building official* shall have the authority to require a building or structure to be re-

inspected. The requirements of this chapter shall not provide the basis for removal or abrogation of fire protection and safety systems and devices in existing structures.

PART III – IBC FIRE SAFETY

Revise as follows:

901.5 Acceptance tests. *Fire protection systems* shall be tested in accordance with the requirements of this code and the *International Fire Code*. When required, the tests shall be conducted in the presence of the *building official*. Tests required by this code, the *International Fire Code* and the standards listed in this code shall be conducted at the expense of the owner or the owner's representative <u>authorized agent</u>. It shall be unlawful to occupy portions of a structure until the required *fire protection systems* within that portion of the structure have been tested and *approved*.

PART IV - IBC MEANS OF EGRESS

Revise as follows:

1004.3 Posting of occupant load. Every room or space that is an assembly occupancy shall have the *occupant load* of the room or space posted in a conspicuous place, near the main *exit* or *exit access doorway* from the room or space. Posted signs shall be of an *approved* legible permanent design and shall be maintained by the owner or <u>the owner's</u> authorized agent.

Reason: The purpose for the proposal is to update the references to "applicant" and "owner" throughout the building code by changing them to the "owner or the owner's authorized agent" where it is warranted. In conjunction with this proposal there are also changes to Chapter 1 and 2, which are in a separate proposal that will be heard by the Administration Committee. In Sections 1703.4.1 and 1707.1, "the applicant" is changed to "the owner or the owner's authorized agent" because the latter should be responsible for the costs of required tests, reports and investigations. In Sections 1703.6 and 1704.2.4, "the applicant" is changed to "the owner or the owner or the owner's authorized agent" because the latter should be responsible for submitting required reports to the building official. In Section 1703.6.1, the applicant" is changed to "the owner or the owner's authorized agent" for consistency with Section 1704.2 that requires the latter to employ the approved agencies. In Section 1803.6, the "owner or authorized agent" is changed to the "permit applicant" because it should be permissible for the latter to submit the geotechnical report with the other submittal documents at the time of permit application.

The 2012 IBC contains additional references to "owner" but, based on the context in which they are used, it is not considered appropriate or useful to revise the language in conjunction with this proposal (e.g., from "the owner" to "the owner or the owner's authorized agent"). See Sections 101.4.4, 104.6, 111.2, 112.3, 116.3, 116.4, 402.3, 913.4, 1107.4-Exc. 1, 1607.7.4, 3108.2, 3307.1, 3412.4, 3412.4.1, G101.2, G105.6-Item 3, K103.1 and L101.3.

The 2012 IBC contains additional references to "applicant" but, based on the context in which they are used, it is also not considered appropriate or useful to revise the language in conjunction with this proposal (e.g., from "the applicant" to "the owner or the owner's authorized agent"). See Sections 104.10.1-Item 5, 105.1.1, 105.3, 107.3.1, 109.3, 109.5, 1612.3.1, 1612.3.2, 1704.2.3, 1704.3, G103.3, G103.4, G103.5.1, G103.6, G104.2, G105.7-Item 5 and J104.1.

All instances in the 2012 IBC of "applicant" and "owner," other than listed above, are included in this proposal.

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

S90-12

PART I - INTERN	IATIONAL BUIL	DING COD	E - STRUCTURAL	
Public Hearing:	Committee:	AS	AM	D
-	Assembly:	ASF	AMF	DF
PART II – INTER	NATIONAL BUI		DE - GENERAL	
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF
PART III – INTER	NATIONAL BUI	LDING CO	DE – FIRE SAFETY	
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART IV - INTERNATIONAL BUILDING CODE - MEANS OF EGRESS

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

1607.7.5-S-BRAZIL.doc

S91–12 1607.7, 1607.7.1, 1607.7.3

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB) (gehrlich@nahb.org)

Revise as follows:

1607.7 Heavy vehicle loads. Floors and other surfaces that are intended to support vehicle loads greater than a 10,000 pound (4536 kg) <u>12,000 pound (5443 kg)</u> gross vehicle weight rating shall comply with Sections 1607.7.1 through 1607.7.5.

1607.7.1 Loads. Where any structure does not restrict access for vehicles that exceed a 10,000-pound (4536 kg) <u>12,000 pound (5443 kg)</u> gross vehicle weight rating, 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.

1607.7.3 Heavy vehicle garages. Garages designed to accommodate vehicles that exceed a 10,000 pound (4536 kg) <u>12,000 pound (5443 kg)</u> gross vehicle weight rating, shall be designed using the live loading specified by Section 1607.7.1. For garages the design for impact and fatigue is not required.

Exception: The vehicular 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 (2.9 kN/m^2). This live load shall not be reduced.

Reason: The purpose of this amendment is to revise the minimum Gross Vehicle Weight Rating (GVWR) necessary to trigger the new heavy vehicle load design provisions approved for the 2012 IBC. The original intent was to address the design of floors and other surfaces needing to support the weight of commercial trucks, buses, fire engines and other large vehicles. It certainly makes sense for a garage or plaza accessible to these large vehicles to be designed for higher loads. However, during the code development process, a 10,000 pound GVWR trigger was added for the special design requirements, unless the owner posts a weight limit. The problem is that many common pick-up trucks and minivans have GVWR's exceeding 10,000 pounds; for example, the Chevy Silverado 3500 (11,400 pounds for the 2006 & 2007 editions), Dodge Ram 3500 (11,000 pounds for the 2006-2008 editions), or Ford F-350 (10,100 pounds for the 2006-2008 editions). Thus, the 2012 IBC language could negatively affect multifamily and mixed-use projects with garages or plazas accessible to these common vehicles. An owner may decide it is not worth the cost to design his garage to the local bridge and highway design standards mandated by the provisions, in which case they would have to post a weight limit and tell residents and visitors they can't park pickup trucks and minivans in the garage. This amendment raises the trigger to a 12,000 pound GVWR, which would clear all of the large pickup trucks and minivans commonly used as individual and family passenger vehicles.

AS	AM	D	
ASF	AMF	DF	1607 7-S-EHRLICH doc
	AS ASF	AS AM ASF AMF	AS AM D ASF AMF DF

S92-12 1607.9.3 (NEW)

Proponent: Gary R. Searer, Wiss, Janney, Elstner Associates, Inc, representing self

Add new text as follows:

1607.9.3 Elements supporting hoists for façade access equipment: In addition to any other

applicable live loads, structural elements that support hoists for façade access equipment shall be designed for a live load consisting of the larger of the rated load of the hoist times 2.5 and the stall load of the hoist.

Reason: Historically, the code has been silent on structural requirements for elements that support facade access equipment, such as swing stages and window washing platforms. The Occupational Safety & Health Administration (OSHA) requires that facade access platforms be designed for four to four-and-a-half times the rated load of the suspended platform. Another OSHA requirement is that the platforms should be designed for one-and-a-half times the stall load of the hoist (this applies to platforms that are used for painting and hanging signs or holiday lights as well as other construction activities). Although OSHA requirements for suspended platforms. Using a design live load of 2.5 times the rated load, when combined with a live load factor of 1.6, results in a total factored load of 4.0 times the rated load, which matches OSHA's requirements for scaffolds used for building maintenance. Although this overall factor might appear excessive, it is intended by OSHA to address accidental hang-up-and-fall scenarios as well as starting and stopping forces that the platforms experience on a day-to-day basis.

Designing for the stall load of the hoist also makes sense, because suspended platforms can get hung up while ascending, generating forces <u>much</u> larger than the rated load of the platform or hoist. If the stall cut-off is working properly, the stall load should be the maximum load that can be delivered to the structural elements supporting the hoist. The load factor of 1.6 typically associated with live loads should safely accommodate variability in the stall load cut-off mechanism, and provides a factored load that closely matches the requirements of OSHA for facade access platforms that are used for construction activities.

These loads have been missing from the building code for far too long and many engineers do not even know that there are specific design requirements for these elements; these are important loads and need to be provided in the building code.

S92-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1607.9.3 #1-S-SEARER.doc

S93-12 1607.9.3 (NEW)

Proponent: Gary R. Searer, Wiss, Janney, Elstner Associates, Inc., representing self

Add new text as follows:

1607.9.3 Lifeline anchorages for façade access equipment: In addition to any other applicable live loads, lifeline anchorages and structural elements that support lifeline anchorages shall be designed for a live load of at least 3100 pounds (13.8 kN) per attached lifeline, in every direction that a fall arrest load may be applied.

Reason: Historically, the code has been silent on structural requirements for elements that support lifelines used to safely access the facades of buildings.

The Occupational Safety & Health Administration (OSHA) requires that lifeline anchorages be designed for an ultimate load of at least 5,000 pounds per attached person. Although OSHA requirements are not written in either code language or engineering language, this proposed change closely matches OSHA requirements for lifeline anchorages. Using a design live load of 3,100 pounds, when combined with a live load factor of 1.6, results in a total factored load of 4,960 pounds, which essentially matches OSHA's requirements for lifeline anchorages. Although this load might appear excessive, it is intended by OSHA to address the fall arrest loads that can and do reasonably occur in typical lanyards for body harnesses, and which are highly variable. OSHA allows stopping forces as high as 2540 pounds to be generated by a person free-falling six feet. Since sometimes people weigh more than the weight assumed by OSHA, since sometimes people may fall more than six feet, and since the lifeline anchorages are used if something has gone wrong with the primary suspension system (and thus represents the user's last hope of avoiding a potentially fatal fall), the effective factor of safety of two -- from an ideal design load of 2540 pounds to an ultimate design load of 5,000 pounds --- is what OSHA deems necessary to provide an acceptable level of safety.

These loads have been missing from the building code for far too long and many engineers do not even know that there are specific design requirements for these elements; these are important loads and need to be provided in the building code.

S93-12				
Public Hearing: Committe	e: AS	AM	D	
Assembly	/: ASF	AMF	DF	
-				1607.9.3 #2-S-SEARER.doc

S94–12 1607.10.2

Proponent: Gary R. Searer/Wiss, Janney, Elstner Associates, Inc., representing self

Revise as follows:

1607.10.2 Alternative uniform live load reduction. As an alternative to Section 1607.10.1 and subject to the limitations of Table 1607.1, uniformly distributed 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 where 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 a maximum of 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.

- 2. 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.
- 3. 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 Equation 16-24.
- 4. For one-way slabs, the area, *A*, for use in Equation 16-24 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)

For SI: R = 0.861(A - 13.94)

Such reduction shall not exceed the smallest of:

- 1. 40 percent for horizontal members supporting one floor;
- 2. 60 percent for vertical members supporting two or more floors; or
- 3. *R* as determined by the following equation.

$$R = 23.1(1 + D/L_o)$$

where:

- A = Area of floor supported by the member, square feet (m²).
- D = Dead load per square foot (m²) of area supported.
- L_{\circ} = Unreduced live load per square foot (m²) of area supported.
- R = Reduction in percent.

Reason: The alternate live load reductions contained in Section 1607.9.2 originated in the Uniform Building Code and were the primary live load reduction formulas used in the western United States for decades. When the live load reductions were brought into the IBC, they were incorporated as an alternate to Section 1607.9.1. During the incorporation of these reductions into the IBC, the maximum reductions were changed from "40 percent for members receiving load from one level only" and "60 percent for other members" (in the 1997 UBC) to the current 40/60 differentiation between horizontal and vertical members. This current differentiation does not match the original wording (because some horizontal members receive live load from more than one floor and because many vertical elements do not receive live load from more than one floor) and does not match the differentiation in Section 1607.9.1, which, like the UBC, differentiates reductions based on whether a member supports one floor or more than one floor: "L shall not be less than $0.50L_{o}$ for members supporting one floor and L shall not be less than $0.40L_{o}$ for members supporting

(Equation 16-25)

(Equation 16-24)

two or more floors." The premise behind differentiating between supporting one floor or more than one floor is basically probabilitybased, and reasonably assumes that the probability that two or more floors are experiencing a relatively large live load is smaller than that of a single floor experiencing a relatively large live load; hence the larger reduction for elements that support more than one floor. The same premise cannot be said of differentiating live load reductions based on horizontality or verticality of the element under consideration.

Since basing allowable live load reductions on number of floors supported as opposed to whether a member is horizontal or vertical makes more sense, this proposal restores the original intent of the UBC provision and brings the provision into better alignment with Section 1607.9.1.

S94-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1607.9.2-S-SEARER.doc

S95–12 1607.12.3.1, Chapter 35 (NEW)

Proponent: Jonathan Siu, City of Seattle, Department of Planning & Development (jon.siu@seattle.gov), Mark S. Graham, National Roofing Contractors Association

Revise as follows:

1607.12.3.1 Landscaped roofs. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m²). The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil <u>as determined in accordance with ASTM E</u> 2397. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m²). The uniform design live load for occupied landscaped areas on roofs shall be determined in accordance with Table 1607.1.

Add new standard to Chapter 35 as follows:

ASTM

<u>E 2397-11 – Standard Practice for Determination of Dead Loads and Live Loads Associated with Green</u> <u>Roof Systems</u>

Reason: ASTM E 2397 is the standard for how to determine the dead load of soils. This is being inserted in the IBC to coordinate with the IGCC, which has many provisions regarding landscaped roofs (aka "vegetative roofs"). This proposal addresses a gap in the regulations, providing an appropriate standard for addressing soil loads. The other changes are editorial:

The weight of landscaping materials applies to all landscaped roofs, and therefore is more appropriate at the beginning of the paragraph.

Adding the reference to Table 1607.1 for occupied landscaped areas is for clarification.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S95-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	2				1607.11.3.1-S-GRAHAM-SIU.doc

S96–12 1607.14, 1607.14.1

Proponent: Edwin Huston, National Council of Structural Engineers Associations- Code Advisory Committee - General Requirements Subcommittee (Huston@smithhustoninc.com)

Revise as follows:

1607.14 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).

Exception: Fabric partitions complying with Section 1607.14.1 shall not be required to resist the minimum horizontal load of 5 psf (0.24 kN/m^a).

1607.14.1 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 horizontal distributed load of 5 psf (0.24 kN/m²) The horizontal distributed load need only be applied to the partition framing. The total area used to determine the distributed load shall be the area of the fabric face between the framing members to which the fabric is attached. The total distributed load shall be uniformly applied to such framing members in proportion to the length of each member.
- A concentrated load of 40 pounds (0.176 kN) applied to an 8-inch diameter (203 mm) area [50.3 square inches (32 452 mm²)] of the fabric face at a height of 54 inches (1372 mm) above the floor.

Reason: Section 1607.14.1, which is limited to only fabric partitions, restates the loading criteria found in Section 1607.14. Since the 5psf loading for partitions under 1604.14 the load is also applicable to fabric partitions. Having the exception to Section 1607.14 is redundant and not necessary

S96-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1607.14-S-HUSTON.doc

S97–12 1609.1.1, Chapter 35 (NEW)

Proponent: Ray C. Minor, P.E., Hapco, representing self (ray.minor@hapco.com)

Revise as follows:

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, *Vult*, 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 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 horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
- 6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7.
- 7. Luminaire support structures designed in accordance with AASHTO LTS-5.

The wind speeds in Figures 1609A, 1609B and 1609C are ultimate design wind speeds, *Vult*, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, *Vasd*, when the provisions of the standards referenced in Exceptions 1 through 5 and 7 are used.

Add new standard to Chapter 35 as follows:

AASHTO

0 44

American Association of State Highway and Transportation Officials 444 North Capitol Street, NW Suite 249 Washington, DC 20001

LTS-5 Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals

Reason: AASHTO LTS-5 is based on much research and many years of experience in using primarily pole type structures to support signs, luminaires and traffic signals along roadways. These type structures are also used for non-roadway applications such as sports lighting and parking lot lighting which may fall under the jurisdiction of the IBC. AASHTO LTS-5 incorporates the results of wind tunnel tests specific to shapes of these structures and the equipment they support. The wind pressure calculations are based on ASCE-7 except with some refinements such as more detailed drag coefficients. Stadium lighting poles involved in several recent failures would not meet the fatigue requirements of AASHTO LTS-5 primarily because the base plates were too thin. These failures most likely would not have occurred if the poles were designed to AASHTO LTS-5. AASHTO LTS-5 is developed by an AASHTO committee with a consensus procedure.

There are other exceptions as precedents for this exception, including similar specifications for flagpoles and communications antennae. The flagpole specification NAAMM 1001 Guide Specification for Design of Metal Flagpoles includes flag wind load equations but otherwise uses the AASHTO LTS-5 procedures for flagpoles

3-11				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1609.1.1-S-MINOR

S98–12 1609.1.1, 1609.3.1

Proponent: Randall Shackelford, P.E., Simpson Strong-Tie Company, Inc. (rshackelford@strongtie.com)

Revise as follows:

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the 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 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.
- Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
- 6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7.

The wind speeds in Figures 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 $4 \underline{3}$ through 5 are used.

1609.3.1 Wind speed conversion. When required, the ultimate design wind speeds of Figures 1609A, 1609B and 1609C shall be converted to nominal design wind speeds, V_{asd} , using Table 1609.3.1 or Equation 16-33.

$$V_{asd} = V_{ult} \sqrt{0.6}$$

(Equation 16-33)

where:

 V_{asd} = nominal design wind speed applicable to methods specified in Exceptions 4 <u>3</u> through 5 of Section 1609.1.1 <u>and other standards not based on ultimate wind speeds</u>.

 V_{ult} = ultimate design wind speeds determined from Figures 1609A, 1609B or 1609C.

Reason: The 2012 WFCM, as referenced in Exception 2 above, is based on Ultimate Wind Speeds, V_{ult} , and therefore does not require conversion of the ultimate wind speed to the nominal wind speed, V_{asd} . Further, the WFCM is the reference standard for wood framing in the ICC-600, so conversion should not take place when using ICC-

Further, the WFCM is the reference standard for wood framing in the ICC-600, so conversion should not take place when using ICC-600 to design wood framing. A committee has been appointed to revise ICC-600, and this code change is written assuming that the basis of ICC-600 will be changed to V_{ult} windspeeds, with conversion factors in the standard for converting to V_{asd} where needed. If by the Public Comment deadline it is not clear that this will be the case, I will prepare a Public Comment to restore Exception 1 to the list of items where conversion is required.

If this code change is not approved, structures designed using the 2012 WFCM with converted windspeeds will be designed for pressures that are only 60% of the pressures they should be designed for.

Section 1609.3.1 needs to be revised for similar reasons. Also, there are other building materials that require testing to "nominal" windspeeds, such as composition shingles in Section 1507.2.7.1. So nominal wind speeds, V_{asd} , is not just used in the Exceptions to 16009.1.1.

Cost Impact: This is not really a fair question for this code change. Yes, there will be a cost impact, because it would definitely be cheaper to design to wind loads that are 40% too low. But you don't want to do that.

S98-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1609.1.1-S-SHACKELFORD.doc

S99–12 1609.1.2

Proponent: Edward L. Keith, P.E., APA – The Engineered Wood Association (ed.keith@apawood.org)

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 buildings with a mean roof height of 33 feet or less 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_{asc} determined in accordance with Section 1609.3.1 does not exceed 140 mph (63 m/s).
- 2. Glazing in *Risk 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 *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.

Reason: In the early days of the development of the SBCCI Deemed-to-Comply document (The precursor to the SBCCI Standard for Hurricane Resistant Residential Construction, SSTD-10, and ultimately the ICC Standard for Residential Construction in High Wind Regions, ICC 600), limits were developed to the geometry of the structures covered by the standard. These limits included a height limit of 33 feet mean roof height. The 33 feet was based on then-current height zoning regulations, the referenced wind speed height in the contemporary ASTM wind standard, as well as height of most anemometers (wind measuring devices). As the Deemed-to-Comply and later documents were limited for wood buildings to two stories in height and as the standards evolved, the height limit was changed from 33 feet mean roof height to simply two stories. Note that the information in the code is based on a mean roof height of 33 feet and NOT two stories. APA developed this information and it is based on 33 feet mean roof height. (APA Form Number T450, free PDF download at apawood.org.)

From a wind perspective, only the geometry of the structure matters. Its internal make-up of floors and walls affect the *resistance* of the structure to the wind but has no impact on the load on the structure. The reason for this change is that the "two story-only" requirement puts artificial limitations on the use of the shutter provisions. This requirement has been used to limit the use of the shutter provisions from 3-story residential structures built on sloped surfaces or with the first story partially embedded in the ground. In either of the cases, the mean roof height may be 33 feet or less.

From the building geometry perspective, the two-story house could be such that the mean roof height exceeds 33 feet. This would make the analytical basis for the shutter design incorrect.

Note that there is no conflict with this proposal and the references to 30 feet in the body of Section 1609.1.2. These provisions are measurements to the glazed openings and are still appropriate with a mean main roof height of 33 feet.

The provisions in the code were originally based on a mean roof height of 33 feet. The shift to two-story was an unfortunate attempt at simplifying the provisions of the early high-wind prescriptive publications. Approval of this change will correct an unintended consequence of this attempt at simplification. Please vote for approval of this provision.

S99-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	•				1609.1.2-S-KEITH.doc

S100–12 1609.1.1, Chapters 35 (NEW)

Proponent: Jennifer Goupil, P.E., The Structural Engineering Institute of ASCE, representing self (jgoupil@asce.org)

Revise as follows:

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the 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.
- Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of 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.
- Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
- 6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7 ASCE 49 and with Sections 31.4 and 31.5 of ASCE 7.

The wind speeds in Figures 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.

Add new standard to Chapter 35 as follows:

ASCE/SEI

49-07 Wind Tunnel Testing for Buildings and Other Structures

Reason: This change proposes to add the new referenced standard *ASCE 49 Wind Tunnel Testing for Buildings and Other Structures.* This standard provides minimum requirements for wind-tunnel tests to determine wind loads on and responses of buildings and other structures. Loads considered in this standard are wind loads for main wind-force resisting systems and for individual structural components and cladding of buildings and other structures. Loads produced by these tests are suitable for use in building codes and standards.

Provisions of this standard satisfy the requirements for wind-tunnel testing of the ASCE Standard ASCE 7, Minimum Design Loads for Buildings and Other Structures. Wind-tunnel testing has the capability to perform measurements beyond those specifically addressed in this standard, including pedestrian wind evaluations, dispersion of airborne pollutants, fugitive particulates, and wind energy siting studies. These studies are permitted to be included within the test report addressing wind loads.

Limited by the scope of ASCE 49, ASCE 7 Sections 31.4 Load Effects and ASCE 7 Section 31.5 Wind-Borne Debris are still essential for determining wind loads and are retained by this proposal.

ASCE/SEI 49 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).

The ASCE 49 committee developed the Standard in coordination with the ASCE 7 Wind Loads Subcommittee with the expectation that the ASCE 7 subcommittee will fully adopt ASCE 49. Further, the ASCE 49 standard is expected to be considered for adopted by reference by the ASCE 7 Main Committee during the next revision cycle.

As of the submission date of this code change proposal, the Standard is currently being published by ASCE. The document is designated ASCE 49 *Wind Tunnel Testing for Buildings and Other Structures* it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in October of 2012. Any person interested in obtaining a public comment copy of ASCE/SEI 49-07 may do so by contacting the proponent at jgoupil@asce.org. A copy of the standard has been submitted with this proposal.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S100-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					1609.1-S-GOUPIL.doc

S101–12 1609.5.2

Proponent: Mark S. Graham, National Roofing Contractors Association (mgraham@nrca.net)

Revise as follows:

1609.5.2 Roof coverings. Roof coverings shall comply with Section 1609.5.1.

Exceptions:

- 1. Rigid tile roof coverings that are air permeable and installed over a roof deck complying with Section 1609.5.1 are permitted to be designed in accordance with Section 1609.5.3.
- 2. Asphalt shingles installed over a roof deck complying with Section 1609.5.1 shall comply with he wind-resistance requirements of Section 1507.2.7.1.

Reason: This code change proposal is intended to clarify the intent of the Code.

Section 1609.5.2 currently has an exception applicable to rigid tile roof coverings and asphalt shingles separately. As currently formatted--as two continuous paragraphs--the intent of these items can be easily misconstrued.

This proposed code change separates these two paragraphs into two separate numbered items, clarifying their intent. This change is not intended to change the Code's current technical requirements for rigid tile or asphalt shingles roofs.

S101-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1609.5.2-S-GRAHAM.doc

S102–12 202 (NEW), 1403.7, 1603.1.7, 1612.4, 1612.5, G103.7, G301.2, G401.2; IPC 309.3; IMC 301.16.1

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net).

Add new text as follows:

SECTION 202 DEFINITIONS

COASTAL A ZONE. Area within a special flood hazard area, landward of a V zone or landward of an open coast without mapped V Zones. In a coastal A zone, the principal source of flooding must be astronomical tides, storm surges, seiches, or tsunamis, not riverine flooding. During the base flood conditions, the potential for breaking wave height shall be greater than or equal to 1.5 ft. The inland limit of the coastal A zone is (a) the Limit of Moderate Wave Action if delineated on a FIRM, or (b) designated by the authority having jurisdiction.

LIMIT OF MODERATE WAVE ACTION. Line that may be shown on FIRMs to indicate the inland limit of the 1.5-foot wave height during the base flood.

Revise as follows:

1403.7 Flood resistance for high-velocity wave action areas <u>and coastal A zones</u>. For buildings in flood hazard areas subject to high-velocity wave action <u>and coastal A zones</u> as established in Section 1612.3, electrical, mechanical and plumbing system components shall not be mounted on or penetrate through exterior walls that are designed to break away under flood loads.

Revise as follows:

1603.1.7 Flood design data. For buildings located in whole or in part in *flood hazard areas* as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

- 1. In *flood hazard areas* not subject to high-velocity wave action <u>or coastal A zones</u>, the elevation of the proposed lowest floor, including the basement.
- 2 In *flood hazard areas* not subject to high-velocity wave action <u>or coastal A zones</u>, the elevation to which any nonresidential building will be dry flood proofed.
- 3. In *flood hazard areas* subject to high-velocity wave action <u>or coastal A zones</u>, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

1612.4 Design and construction. The design and construction of buildings and structures located in *flood hazard areas*, including flood hazard areas subject to high-velocity wave action <u>and coastal A</u> <u>zones</u>, shall be in accordance with Chapter 5 of ASCE 7 and with ASCE 24.

1612.5 Flood hazard documentation. The following documentation shall be prepared and sealed by a *registered design professional* and submitted to the *building official*:

1. For construction in flood hazard areas not subject to high-velocity wave action or coastal A zones:

- 1.1. The elevation of the lowest floor, including the 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 of 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 and coastal A zones:
 - 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 have a resistance of more than 20 psf (0.96 kN/m²) determined using allowable stress design, *construction documents* shall include a statement that the breakaway wall is designed in accordance with ASCE 24.

Revise as follows:

G103.7 Alterations in coastal areas. Prior to issuing a permit for any alteration of sand dunes and mangrove stands in flood hazard areas subject to high velocity wave action <u>and coastal A zones</u>, the *building official* shall require submission of an engineering analysis which demonstrates that the proposed alteration will not increase the potential for flood damage.

G301.2 Subdivision requirements. The following requirements shall apply in the case of any proposed subdivision, including proposals for manufactured home parks and subdivisions, any portion of which lies within a flood hazard area:

- 1. The flood hazard area, including floodways, and areas subject to high velocity wave action, and coastal A zones, as appropriate, shall be delineated on tentative and final subdivision plats;
- 2. Design flood elevations shall be shown on tentative and final subdivision plats;
- 3. Residential building lots shall be provided with adequate buildable area outside the floodway; and
- 4. The design criteria for utilities and facilities set forth in this appendix and appropriate *International Codes* shall be met.

G401.2 Flood hazard areas subject to high-velocity wave action <u>and coastal A zones</u>. In *flood hazard areas* subject to high-velocity wave action <u>and coastal A zones</u>:

- 1. New buildings and buildings that are substantially improved shall only be authorized landward of the reach of mean high tide.
- 2. The use of fill for structural support of buildings is prohibited.

[B] 309.3 Flood hazard areas subject to high-velocity wave action <u>and coastal A zones</u>. Structures located in flood hazard areas subject to high-velocity wave action <u>and coastal A zones</u> shall meet the requirements of Section 309.2. The plumbing systems, pipes and fixtures shall not be mounted on or penetrate through walls intended to break away under flood loads.

[B] 301.16.1 High-velocity wave action <u>and coastal A zones</u>. In flood hazard areas subject to high-velocity wave action <u>and coastal A zones</u>, mechanical systems and *equipment* shall not be mounted on or penetrate walls intended to break away under flood loads.

Reason: The IBC achieves compliance with the NFIP in Sec. 1612, by reference to ASCE 24 for the specific design and construction requirements. This proposal is to insert the term "coastal A zone" wherever the term "flood hazard area subject to high velocity wave action" appears, to be consistent with ASCE 24. Because of the way the term is defined, only if the Limit of Moderate Wave Action is delineated (or otherwise designated by the AHJ), is the area to be regulated as coastal A zone. ASCE 24-05 has provisions that apply in all Coastal High Hazard Areas (Zone V) and coastal A zones, essentially treating them the same (there are some slight differences because coastal A zones are shown as "Zone A" on Flood Insurance Rate Maps). When 1612.4 refers the user to ASCE 24, one of the first determinations is which flood hazard zone affects the building site. Currently, ASCE 24-05 requires the designer to determine whether conditions landward of Zone V meet the characteristics necessary for coastal A zone conditions. The proposed definition is consistent with the next edition of ASCE 24 that will specify that only if the Limit of Moderate Wave Action (LiMWA) is delineated on the FIRM (or otherwise designated by the AHJ) will the requirements for CAZ apply. FEMA uses the LiMWA to delineate the inland extend of CAZ.

A separate proposal was submitted to change the term "flood hazard area subject to high velocity wave action" to be "coastal high hazard area," which is the term used in the IRC and ASCE 24.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: Costs will be lower because the RDP and the building official will not have to made independent determinations as to whether a site landward of a Zone V does or does not have coastal A zone conditions. For areas that are subject to coastal A zone conditions there is no change in construction costs because ASCE 24 already has specifications based on whether a building site is or is not subject to coastal A zone conditions.

S102-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1403.7-S-INGARGIOLA-WILSON.doc

S103–12 202, 1403.7, 1603.1.7, 1612.3, 1612.5, 1804.4, G103.7, G301.2, G401.2, G601.1; IPC P309.3, IMC M301.16.1

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net).

THIS IS A THREE PART CODE CHANGE. ALL THREE PARTS WILL BE HEARD BY THE STRUCTURAL COMMITTEE AS THREE SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDER FOR THIS COMMITTEE

PART I – IBC STRUCTURAL

Revise as follows:

SECTION 202 DEFINITIONS

FLOOD HAZARD AREA SUBJECT TO HIGHVELOCITY WAVE ACTION COASTAL HIGH HAZARD AREA. Area within the special flood hazard area extending from offshore to the inland limit of a primary dune along an open coast and any other area that is subject to high-velocity wave action from storms or seismic sources, and shown on a Flood Insurance Rate Map (FIRM) or other flood hazard map as velocity zones Zone V, VO, VE or V1-30.

Revise as follows:

1403.7 Flood resistance for high-velocity wave action areas <u>coastal high hazard areas</u>. For buildings in flood hazard areas subject to high-velocity wave action <u>coastal high hazard area</u> as established in Section 1612.3, electrical, mechanical and plumbing system components shall not be mounted on or penetrate through exterior walls that are designed to break away under flood loads.

Revise as follows:

1603.1.7 Flood design data. For buildings located in whole or in part in *flood hazard areas* as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

- 1. In flood hazard areas not subject to high-velocity wave action other than coastal high hazard areas, the elevation of the proposed lowest floor, including the basement.
- 2. In flood hazard areas not subject to high-velocity wave action other than coastal high hazard areas, the elevation to which any nonresidential building will be dry flood proofed.
- In *flood hazard areas* subject to high-velocity wave action <u>coastal high hazard areas</u>, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

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

1612.5 Flood hazard documentation. The following documentation shall be prepared and sealed by a *registered design professional* and submitted to the *building official*:

- 1. For construction in *flood hazard areas* not subject to high-velocity wave action other than coastal high hazard areas:
 - 1.1. The elevation of the lowest floor, including the 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 of 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.
- For construction in flood hazard areas subject to high-velocity wave action coastal high hazard areas:
 - 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 have a resistance of more than 20 psf (0.96 kN/m2) determined using allowable stress design, *construction documents* shall include a statement that the breakaway wall is designed in accordance with ASCE 24.

Revise as follows:

1804.4 Grading and fill in flood hazard areas. In *flood hazard areas* established in Section 1612.3, grading and/or fill shall not be *approved*:

- 1. Unless such fill is placed, compacted and sloped to minimize shifting, slumping and erosion during the rise and fall of flood water and, as applicable, wave action.
- 2. In floodways, unless it has been demonstrated through hydrologic and hydraulic analyses performed by a *registered design professional* in accordance with standard engineering practice that the proposed grading or fill, or both, will not result in any increase in flood levels during the occurrence of the *design flood*.
- In flood hazard areas subject to high-velocity wave action coastal high hazard areas, unless such fill is conducted and/or placed to avoid diversion of water and waves toward any building or structure.
- 4. Where design flood elevations are specified but floodways have not been designated, unless it has been demonstrated that the cumulative effect of the proposed *flood hazard area* encroachment, when combined with all other existing and anticipated *flood hazard area* encroachment, will not increase the design flood elevation more than 1 foot (305 mm) at any point.

Revise as follows:

G103.7 Alterations in coastal areas. Prior to issuing a permit for any alteration of sand dunes and mangrove stands in flood hazard areas subject to high velocity wave action coastal high hazard areas, the *building official* shall require submission of an engineering analysis which demonstrates that the proposed alteration will not increase the potential for flood damage.

G301.2 Subdivision requirements. The following requirements shall apply in the case of any proposed subdivision, including proposals for manufactured home parks and subdivisions, any portion of which lies within a flood hazard area:

- 1. The flood hazard area, including floodways and areas subject to high velocity wave action coastal high hazard areas, as appropriate, shall be delineated on tentative and final subdivision plats;
- 2. Design flood elevations shall be shown on tentative and final subdivision plats;
- 3. Residential building lots shall be provided with adequate buildable area outside the floodway; and
- 4. The design criteria for utilities and facilities set forth in this appendix and appropriate *International Codes* shall be met.

G401.2 Flood hazard areas subject to high-velocity wave action Coastal high hazard areas. In flood hazard areas subject to high-velocity wave action coastal high hazard areas:

- 1. New buildings and buildings that are substantially improved shall only be authorized landward of the reach of mean high tide.
- 2. The use of fill for structural support of buildings is prohibited.

G601.1 Placement prohibited. The placement of recreational vehicles shall not be authorized in *flood hazard areas* subject to high velocity wave action coast high hazard areas and in *floodways*.

PART II – IPC

Revise as follows:

[B] P309.3 Flood hazard areas subject to high-velocity wave action Coastal high hazard areas. Structures located in flood hazard areas subject to high-velocity wave action coastal high hazard areas shall meet the requirements of Section 309.2. The plumbing systems, pipes and fixtures shall not be mounted on or penetrate through walls intended to break away under flood loads.

PART III – IMC

Revise as follows:

[B] 301.16.1 High-velocity wave action <u>Coastal high hazard areas</u>. In flood hazard areas subject to high-velocity wave action coastal high hazard areas, mechanical systems and *equipment* shall not be mounted on or penetrate walls intended to break away under flood loads.

Reason: This proposal is to simply replace one term with another and edit the definition to be consistent with how the term is defined in ASCE 24. The term "Flood Hazard Area Subject to High-Velocity Wave Action" is descriptive of the flood hazard areas designated Zone V on Flood Insurance Rate Maps. However, the term is not used by the NFIP, nor is it used in the IRC or in ASCE 24, which is referenced by the IBC (1612.4). The NFIP regulations define "coastal high hazard area" at 40 CFR 59.1.

S103-12				
PART I - INTER	RNATIONAL BUILDING	G CODE - STRU	ICTURAL	
Public Hearing:	Committee:	AS	AM	D
C	Assembly:	ASF	AMF	DF
PART II – INTE	RNATIONAL PLUMBIN	NG CODE		
Public Hearing:	Committee:	AS	AM	D
Ū	Assembly:	ASF	AMF	DF
PART III – INTE	ERNTIONAL MECHANI	CAL CODE		
Public Hearing:	Committee:	AS	AM	D
Ū	Assembly:	ASF	AMF	DF
			202-COASTAL HIG	H HAZARD AREA-S-INGARGIOLA-WILSON.doc

S104–12 202

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, pregory.p.wilson@dhs.gov), Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

DRY FLOODPROOFING. A combination of design modifications that results in a building or structure, including the attendant <u>utility utilities and equipment</u> and sanitary facilities, being water tight with walls substantially impermeable to the passage of water and with structural components having the capacity to resist *loads* as identified in ASCE 7.

Reason: This editorial change is proposed for consistency with term as used in the next edition of ASCE 24. The current edition, ASCE 25-05, uses both the term "attendant utilities and equipment" (preferred) and the term "utilities and attendant equipment." All uses of the latter will be revised for consistency.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

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

S104-12				
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

202-S-DRY FLOODPROOFING-INGARGIOLA-WILSON-QUINN.doc

S105– 12 1612.4, 1612.4.1 (NEW), 1612.4.2 (NEW),

Proponent: Stephen V. Skalko, P.E., Portland Cement Association, Eric T. Stafford, P.E., representing Institute for Business and Home Safety and Jason Thompson, P.E., National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

Revise as follows:

1612.4 Design and construction. The design and construction of buildings and structures located in *flood hazard areas*, including flood hazard areas subject to high-velocity wave action, shall be in accordance with Chapter 5 of ASCE 7, ASCE 24, Sections 1612.4.1 and 1612.4.2 as applicable.

1612.4.1 Floor elevation. Floors required by ASCE 24 to be built above the base flood elevations shall have the floor and their lowest horizontal supporting members not less the higher of the following:

- 1. design flood elevation,
- 2. base flood elevation plus 3 feet, or
- 3. advisory base flood elevation plus 3 feet, or
- 4. the 500-year flood, if known.

1612.4.2 Flood protective works. Buildings designed and constructed in accordance with ASCE 24 shall not consider levees and floodwalls for providing flood protection during the design flood.

Reason: Buildings constructed in accordance with the Section 1612 of the International Building Code are considered to meet minimum requirements. However recent flood hazard events have demonstrated that the requirements in the present code are not sufficient. This proposal strengthens the requirements in the code for establishing the habitable floor elevation with a reasonable safety factor.

First, the elevation of lowest floor level above the base flood elevation is increased from the level normally considered acceptable to meet minimum requirements of the IBC. Many local jurisdictions already modify the IBC with provisions of two, three, or even more feet above the base flood elevation as the required minimum elevation of floors for occupiable space.

Secondly, levees and floodwalls should not be considered as flood protection for structures during a design flood. This is consistent with the primary directive of ASCE 24, *Flood Resistant Design and Construction*, referenced in Section 1612 of the IBC. In recent times there has been an increase in the amount of individual property damage, loss of life and destruction of whole neighborhoods when areas at risk to flooding along river basins are inundated by water after protective works failed or were overtopped or breeched. Examples of levees that failed to protect properties are shown in the attached photographs.



17th Street Levee – New Orleans – USACE



Levee Break – Grand Forks, ND - FEMA



Flood damage - FEMA

S105-12				
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

1612.4-S-SKALKO-STAFFORD-THOMPSON.doc

S106–12 1612.5

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov) (gregory.p.wilson@dhs.gov), Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

1612.5 Flood hazard documentation. The following documentation shall be prepared and sealed by a *registered design professional* and 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 the basement, as required by the lowest floor elevation inspection in Section 110.3.3 and for the final inspection in Section 110.3.10.1.
 - 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 of 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 and for the final inspection in Section 110.3.10.1.
 - 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 have a resistance of more than 20 psf (0.96 kN/m2) determined using allowable stress design, *construction documents* shall include a statement that the breakaway wall is designed in accordance with ASCE 24.

Reason: This proposal achieves consistency with Section 110. The 2012 IBC includes a requirement, added in the last code change cycle, that surveyed building elevations be submitted to the building official prior to the final inspection (approved by ADM14-09/10).

S106-12			
Public Hearing: Committee:	AS	AM	D
Assembly:	ASF	AMF	DF
			1612.5-S-INGARGIOLA-WILSON-QUINN.doc
S107–12 1613.1

Proponent: James Bela, Oregon Earthquake Awareness, representing self

Revise as follows:

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

Exceptions:

- 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, SS, is less than 0.4 g.
- The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section

2308 are not required to be analyzed as specified in this section.

- 3. Agricultural storage structures intended only for incidental human occupancy.
- 4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

Reason: (1) ASCE 7 adopted the NEHRP Provisions (developed at the public's expense) as its "standard, then proceeded to charge the engineering community (and the public) for its "commandeering" of those Provisions as its standard.

(a) NEHRP Provisions previously have been adopted into model building codes, as in the Southern Building Code, with no problems (and, particularly, with no "added expense." ASCE 7 carries a "disclaimer" for its use.

(2) ASCE 7 contains no "references" to justify its legitimacy.

(3) ASCE 7 was the instigator of so-called:) <u>RISK-TARGETED</u> MAXIMUM CONSIDERED EARTHQUAKE (MCE_B) (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR 0.2- and 1SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B.

(a) this is based on fatally flawed "applied mathematics" assumed in probabilistic seismic hazard assessment, or psha: see discussions under Code Change: FIGURES 1613.3.1 (1)(2)(3)(4)(5)(6)

(4) ASCE 7 is "codifying everything," and is becoming a de-facto code. Code provisions need to remain in a public consensus arena; their "disclaimer" perhaps absolves them from the problems they are creating – but they are creating "unintended consequences" for professional practice.

(5) ASCE 7 is full of errata, which casts substantial questions about the quality of effort and rigor that is going into its formulation.

S107-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
5				1613.1-S-BELA.doc

S108-12

1613.1

Proponent: Jeff Sprout, AIA, Target Corporation (jeff.sprout@target.com)

Revise as follows:

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

Exception:

- 1. Detached one- and two-family dwellings, assigned to *Seismic Design Category* A, B or C, or located where the mapped short-period special response acceleration, Ss, is less than 0.4g.
- 2. The seismic force-resisting system of wood-frame building that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
- 3. Agricultural storage structures intended only for incidental human occupancy.
- 4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.
- 5. <u>Anchorage of fixtures, cases, shelves, counters and partitions not over 8'-0" in height when</u> designed to resist overturning.

Reason: To provide further clarification that ties back into Section105.2 Work exempt from permit, item #13: "Nonfixed and movable fixtures, cases, racks, counters and partitions not over 5 feet 9 inches (1753mm) in height". It has been shown in shake table tests and in the recent Japan earthquake, where unanchored fixtures under 8' tall, that did resist overturning, kept the aisle ways reasonably clear, allowing for safe exiting.

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

S108-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
2				1613.1-S-SPROUT.doc

S109–12 1613.3.1

Proponent: Nicolas Luco, US Geological Survey (USGS), representing National Earthquake Hazards Reduction Program (nluco@usgs.gov), Michael Mahoney, Federal Emergency Management Agency (FEMA), representing National Earthquake Hazards Reduction Program

Revise as follows:

1613.3.1 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 1613.3.1(1) through 1613.3.1($\underline{67}$) 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 *Seismic Design Category* A. The parameters Ss and S1 shall be, respectively, 1.5 and 0.6 for Guam and 1.0 and 0.4 for American Samoa.



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



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



Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for Guam and American Samoa of 0.2-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

FIGURE 1613.3.1(7) RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR GUAM AND AMERICAN SAMOA OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Reason: The US Geological Survey (USGS) has the responsibility under the National Earthquake Hazards Reduction Program to develop and maintain seismic hazard maps that are the basis of the Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion maps in the nation's model building codes. As part of that responsibility, the USGS recently developed seismic hazard and MCE_R ground motion maps for Guam and American Samoa, using the same methodology as for the conterminous US, Hawaii, Alaska, and Puerto Rico and the US Virgin Islands. The MCE_R ground motion maps developed are being proposed as an addition to the existing maps in Figure 1613.3.1.

Cost Impact: The code change proposal will increase or decrease the cost of construction, depending on the geographic location.

S109-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AME	DF	1613.3.1-S-LUCO-MAHONEY.doc

S110–12 Figures 1613.3.1(1) (NEW), 1613.3.1(2) (NEW), 1613.3.1(3) (NEW), 1613.3.1(4) (NEW), 1613.3.1(5) (NEW), 1613.3.1(6) (NEW)

Proponent: James Bela, Oregon Earthquake Awareness, representing self

Delete and substitute as follows:

FIGURE 1613.3.1(1)

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

FIGURE 1613.3.1(2)

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

FIGURE 1613.3.1(3)

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

FIGURE 1613.3.1(4)

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

FIGURE 1613.3.1(5)

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

FIGURE 1613.3.1(6)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCER) GROUND MOTION RESPONSE ACCELERATIONS FOR PUERTO RICO AND THE UNITED STATES VIRGIN ISLANDS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5%OF CRITICAL DAMPING), SITE CLASS B



FIGURE 1613.3.1(1) MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



FIGURE 1613.3.1(1) - continued MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



FIGURE 1613.3.1(2) MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



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



FIGURE 1613.3.1(3)

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



FIGURE 1613.3.1(4) MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



FIGURE 1613.3.1(5)

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



FIGURE 1613.3.1(6)

MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR PUERTO RICO AND THE UNITED STATES VIRGIN ISLANDS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Reason: (1) Constantly changing the USGS National Seismic Hazard Maps' "ground motion response accelerations contours" is **destabilizing** to design practice, plan review requirements, and code enforcement provisions, because such changes are:

(a) creating yo-yo earthquake design standards - "high" one code cycle and "low" the next; or vice-versa; making it, as a result,

ever more difficult to develop, practice and apply "professional engineering judgment" in the design process. (b) creating serious and perplexing problems for addressing seismic hazards for **existing buildings** – which must then "**benchmark**" to a specific year and to a specific version (year & edition) of seismic hazard map (for any specific public policy mandate/requirements for earthquake retrofit/mitigation ordinances or measures. These required "benchmark" seismic hazard maps will then be different (sometimes a lot different) from the current (and ever-changing and ever-evolving) USGS National Seismic Hazard Maps. This is, and will continue to be, a big source of confusion.

(2) RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE

ACCELERATIONS contours in the IBC 2012 / ASCE 7-10 are sometimes 30% lower than previous map values of just a decade ago:

(a) the recent 08-23-2011 M 5.8 Mineral VA (Cuckoo) earthquake had 30% lower design values (with these new maps) than a decade ago – making the earthquake's epicentral region **Seismic Design Category A-B**; yet the **actual intensity of earthquake ground shaking** experienced there was the "stated intensity" that could be expected for the IBC/ASCE 7-10 designation **SDC D!** (Bela 2011)

(b) when the seismic hazard maps depict such low hazard ground motion response accelerations and their corresponding low Seismic Design Categories, they both foster and create the "circumstances" for "comfortable inaction;" and, unfortunately, this feeling of "comfortable inaction" easily transfers to the arena of public policy.
(c) The condition of "comfortable inaction" (due to perceived low hazard - depicted on the seismic hazard map) was cited as perhaps the main culprit in Christ Church, New Zealand's lack of adequate preparedness during its recent hammering by a "pair" of earthquakes – which killed around 200 people in unsafe "Killer Buildings.".

(3) The basic underlying methodology for preparing the USGS National Seismic Hazard Maps (and their derivative socalled Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations contours); i.e., probabilistic seismic hazard assessment (or psha) is fatally "**flawed**" – due to systemic "errors" in the applied mathematics which both create and define it. And it is, unfortunately, these same flawed "mathematics" that are prescribing how these psha-determined ground motion contours are ultimately derived, computed ... and then finally codified.

(4) Errors in its methodology aside, the basic problems, difficulties and really insurmountable obstacles to performing a psha seismic hazard assessment (*Mualchin, 2010; Bela and Mualchin 2011*) have **never** actually been "solved." And they still remain unsolved! These problems involve data-driven earth-science requirements for a knowledge and understanding of:

(a) fault slip rates;

(b) frequency of occurrence of earthquakes (and their known magnitudes); and

(c) earthquake source mechanisms – specifically, (i) the style of faulting: and (ii) the hypocentral depth (or where exactly the earthquake rupture process begins).

(5) The psha methodology is easily "manipulated," particularly in the sense that: (i) selecting the probabilistic hazard level is a totally arbitrary process; and (ii) changing the hazard level (higher hazard or lower hazard) gives a completely different ground motion response acceleration contour – and consequently, then, different code requirements!
(6) These very real and insurmountable problems with psha's methodology have been swept away by its proponents: by convoluted (and mostly unintelligible) efforts and preoccupations with "logic trees," "quantifying uncertainties," etc. These efforts proceed busily ahead; but, meanwhile, they are "neglecting baseline principles" (of "what" the earthquake can do to you – and "how" it can do it – and the maximum Magnitude it could be). All that mathematical busywork, logic-tree accounting, and so-called "expert opinion" built a the "better model" (or -- so the proponents believe). Unfortunately, that "better model" then:

(a) has become "substituted" for "reality" by its creators;

(b) has dismissed criticisms of it -- by claiming (itself) to be "best available science;" and

(c) has become ultimately so "complicated" -- that not even its proponents now can logically and successfully explain how it came to be (Hamburger et. al., 2010; Bela, 2011); nor can they effectively explain how to apply it to the real world of earthquake engineering, public safety, and socioeconomic issues of community resiliency.

(7) The ground motion accelerations, and their probabilities for exceeding them, are combined and co-mingled in such a way that the actual sources (or earthquake magnitudes, frequency content of earthquake ground motions, and duration of strong ground shaking) are treated more-or-less equally—and they are most certainly not!
(8) The "Maximum Credible Earthquake" (MCE) or "Maximum Capable Earthquake" or "Maximum Possible Earthquake" (within ¼ unit of Magnitude, M) is never explicitly stated. And it's really "Magnitude, Magnitude, Magnitude!" (and for the same reasons previously stated in (4)) – that has everything to do with building performance (damage and repair costs) and, more importantly, public safety and community resilience.

(9) R-Factors, or Response Modification Factors, that are used in design become less reliable in ascertaining/predicting the "end result" (or the building's actual performance in an earthquake). And, "an earthquake" really needs to explicitly consider the full suite of earthquake possibilities that the regional tectonics forewarn us can occur (including MCE = Maximum Credible Earthquake, or Maximum Possible Earthquake). "R-Factors" have become less reliable primarily because:

(a) quite a lot of the "ductility" or building "toughness" that the code relies upon to: (i) ride out the earthquake (by bending, not breaking, and absorbing energy); and (ii) remain standing (without killing the occupants) -- is due to "overstrength;" and.

(b) when the code design "strength" is systematically diminished (weakened) or reduced (over several-to-many iterations of seismic hazard mapping --by lowering (yo-yo effect) the "numerator" quantity in the design strength

equation; then when dividing this numerator (now smaller number) by the same "large" number (R-Factor in denominator) - we have now "lost" perhaps a good portion of our "over-strength" - that was implicit in selecting the weights of the various R-Factors in the first place!

Basically, with RISK-TARGETED (MCE_R), the code is now dividing an ever-decreasing and now smaller number (perhaps by 30%) by the same "large" number (R-Factor denominator) -- with the result that the buildings' performances and outcomes are really now much less certain . . . and also now much more problematical. (10) The psha methodology has been shown in dramatic and tragic fashion to be not only "misleading", but also deadly, in the last decade or so of the "Eleven of the World's Deadliest Earthquakes." (Panza et. al. 2011, Table 1) In example after example, and all across the globe (where now more than 700, 000 people have perished); the pshamethodology "prescribed" seismic hazard: was determined to be either low or very low - but was "disproved" in these many cases by earthquakes that were "surprises" from what psha had determined could be expected. In too many of these deadly "surprises", the actual intensities of ground shaking experienced were greater by factors of 2X to 4X than what psha had predicted. (Bela 2010; Bela and Mualchin, 2011;

Kossobokov and Nekrasova, 2010;)

It is clear that this is an unsafe situation (to general public) that must not continue; but it does continue for some of these following main reasons:

(a) the psha methodology is "anonymous," so when there is clear evidence (> 700,000 casualties) that it is "not working;" no one is accountable for its: (i) external failures (mass casualties); and/or (ii) internal failures (very real errors in its "applied mathematics" derivations).

(b) the psha methodology has a hierarchial and powerful elite behind its influence and continued use. (c) the psha methodology has a pedigree of high sounding terms (like "quantifying uncertainty," "logic-tree", "expert opinion," "best science," etc.) -- all purporting to increase the method's "precision." But the end result, as these Eleven Deadliest Earthquakes" have shown us, is, unfortunately, still too "inaccurate" and "too deadly" for protecting the public safety. And in this regard, it is clearly missing its target!

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Mualchin, L. (2010) History of Modern Earthquake Hazard Mapping and Assessment in California Using a Deterministic or Scenario Approach, Pure and Applied Geophysics Volume 168, Numbers 3-4, 383-407, DOI: 10.1007/s00024-010-0121-1 [From the issue entitled "Special Issue: Advanced Seismic Hazard Assessment. Part II: Regional Seismic Hazard and Seismic Microzonation Case Studies"]

http://www.springerlink.com/content/p301955r53nk5xl2/

Panza, G.F.; Irikura, K.; Kouteva-Guentcheva, M.; Peresan, A.; Wang, Z.; Saragoni, R. (Eds.) (2011). Advanced Seismic Hazard Assessment, 320 p.

http://www.springer.com/earth+sciences+and+geography/book/978-3-0348-0091-4

Table 1 List of the top eleven deadliest earthquakes occurred during the period 2000-2011 and the corresponding intensity differences (ΔI) among the observed values and those predicted by the Global Seismic Hazard Assessment Program, or GSHAP.

Allesandro Martelli, Paolo Clemente, Massimo Forni, Giuliano F. Panza, Antonello Salvatori (2011).

RECENT DEVELOPMENT AND APPLICATION OF SEISMIC ISOLATION AND ENERGY DISSIPATION SYSTEMS, IN PARTICULAR IN ITALY, CONDITIONS FOR THEIR CORRECT USE AND RECOMMENTATIONS FOR CODE IMPROVEMENTS, IN 12TH WORLD CONFERENCE ON SEISMIC ISOLATION, ENERGY DISSIPATION AND ACTIVE CONTROL OF STRUCTURES Sept. 20-23, 2011 Sochi-city, Russia

S110-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				F1613-S-BELA doc

S111–12 1613.5 (NEW), 1613.5.1 (NEW)

Proponent: Kelly Cobeen, representing self; Dana Deke Smith and Steve Winkel, Building Seismic Safety Council, representing FEMA/Code Resource Support Committee (dsmith@nibs.org) (swinkel@preview-group.com)

Add new text as follows:

1613.5 Amendments to ASCE 7. The provisions of Section 1613.5 shall be permitted as an amendment to the relevant provisions of ASCE 7.

1613.5.1 Transfer of anchorage forces into diaphragm. Modify ASCE 7 Section 12.11.2.2.1 as follows:

<u>12.11.2.2.1 Transfer of anchorage forces into diaphragm</u>. Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorages forces into the diaphragms. Diaphragm connections shall be positive, mechanical, or welded. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of a wood, wood structural panel, or untopped steel deck sheathed structural subdiaphragm that serves as part of the continuous tie system shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

Reason: The subdiaphragm aspect ratio is indicated in this proposal as only applying to wood sheathed diaphragms, wood structural panel sheathed diaphragms, and untopped metal deck diaphragm. When limitation of subdiaphragms was first submitted as a proposed change to the 1997 UBC by Kariotis [code change proposal 1631.2.8-95-1 K.A.S.E.] in the form of an allowable shear limitation, the reason focused on tilt-up buildings with nailed diaphragms and contemporary designs not meeting the intent of provisions written after observed poor performance in the 1973 Sylmar Earthquake. When approved for inclusion in the 1997 UBC [code change proposal 16-96-2 SEAOC/ Seismology] the approved wording for the aspect ratio limitation specifically applied only to wood structural subdiaphragms. In the process of being included in the IBC and ASCE 7, the wording designating wood subdiaphragms was dropped, making the requirement applicable to all subdiaphragms. This code change proposes to reintroduce the limit to wood subdiaphragms due to the similarities in construction and perceived structural behavior. This aspect ratio limit is not perceived to be necessary for good performance for other diaphragm types; once this aspect ratio limit is removed for concrete, composite deck, and other diaphragm types, other diaphragm limitations within the referenced material standards will govern design.

Cost Impact: The code change proposal will not increase the cost of construction and may reduce cost for some structural systems.

S111-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1613.5.1-S-COBEEN-SMITH-WINKEL.doc

S112–12 1701.3

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Delete without substitution:

1701.3 Used materials. The use of second-hand materials that meet the minimum requirements of this code for new materials shall be permitted.

Reason: This is nearly identical to 104.9.1 and is thus redundant here.

S112-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1701.3-S-KERR.doc

S113-12

1703.1.3, 1703.5.2, 1703.6, 1703.6.2, 1704.1, 1704.2, 1704.2.1, 1704.2.2, 1704.2.4, 1704.3, 1704.3.1, 1704.3.2, 1705.1, 1705.1.1, Table 1705.2.2, 1705.3, Table 1705.3, 1705.3.1, 1705.4, 1705.4.1, 1705.4.2, 1705.6, Table 1705.6, 1705.7, Table 1705.7, 1705.8, Table 1705.8, 1705.9, 1705.11.1, 1705.13, 1705.13.1, 1705.13.2, 1705.14, 1901.4, [F] 909.18.8, [F] 909.18.8.1, [F] 909.21.7[F] 1705.17, [F] 1705.17.1

Proponent: Phillip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

THIS IS A TWO PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IFC COMMITTEE, AS TWO SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES

PART I – IBC STRUCTURAL

Revise as follows:

1703.1.3 Personnel. An *approved agency* shall employ experienced personnel educated in conducting, supervising and evaluating tests and/or *special* inspections.

1703.5.2 Inspection and identification. The *approved agency* shall periodically perform an <u>a special</u> inspection, which shall be in-plant if necessary, of the product or material that is to be *labeled*. The *inspection special inspector* shall verify that the labeled product or material is representative of the product or material tested.

1703.6 Evaluation and follow-up inspection services. Where structural components or other items regulated by this code are not visible for <u>special</u> inspection after completion of a prefabricated assembly, the applicant shall submit a report of each prefabricated assembly. The report shall indicate the complete details of the assembly, including a description of the assembly and its components, the basis upon which the assembly is being evaluated, test results and similar information and other data as necessary for the *building official* to determine conformance to this code. Such a report shall be *approved* by the *building official*.

1703.6.2 Test and inspection records. Copies of necessary test and <u>special</u> inspection records shall be filed with the *building official*.

SECTION 1704 SPECIAL INSPECTIONS AND TESTS, CONTRACTOR RESPONSIBILITY AND STRUCTURAL OBSERVATIONS

1704.1 General. This section provides minimum requirements for special inspections <u>and tests</u>, the statement of special inspections, contractor responsibility and structural observations.

1704.2 Special inspections and tests. 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 <u>special</u> inspections <u>and tests</u> during construction on the types of work listed under Section 1705. These <u>special</u> inspections <u>and tests</u> are in addition to the inspections <u>by the building official that are</u> identified in Section 110.

Exceptions:

1. Special inspections <u>and tests</u> are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as *approved* by the *building official*.

- 2. Unless otherwise required by the *building official*, *special inspections* <u>and tests</u> 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.
- 3. Special inspections and tests are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.

1704.2.1 Special inspector qualifications. 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 where the documented experience or training is related in complexity to the same type of *special inspection* or testing activities for projects of similar complexity and material qualities. These qualifications are in addition to qualifications specified in other sections of this code. 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 inspectors.

1704.2.2 Access for special inspection. The construction or work for which special inspection <u>or testing</u> is required shall remain accessible and exposed for special inspection <u>or testing</u> purposes until completion of the required special inspections <u>or tests</u>.

1704.2.4 Report requirement. Special inspectors shall keep records of <u>special</u> inspections <u>and tests</u>. The special inspector shall furnish submit reports of <u>special</u> inspections reports and tests to the building official, and to the registered design professional in responsible charge. Reports shall indicate that work inspected <u>or tested</u> was or was not completed in conformance to <u>approved construction documents</u>. 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 <u>or tests</u>, and correction of any discrepancies noted in the inspections <u>or tests</u>, shall be submitted at a point in time agreed upon prior to the start of work by the applicant and the *building official*.

1704.3 Statement of special inspections. Where *special inspections* or testing is tests are required by Section 1705, the *registered design professional in responsible charge* shall prepare a statement of special inspections in accordance with Section 1704.3.1 for submittal by the applicant in accordance with Section 1704.2.3.

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*.

1704.3.1 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 inspections* or testing tests 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 inspections* or testing tests for seismic or wind resistance as specified in Sections 1705.10, 1705.11 and 1705.12.
- 5. For each type of *special inspection*, identification as to whether it will be continuous *special inspection* or periodic *special inspection*.

1704.3.2 Seismic requirements in the statement of special inspections. Where Section 1705.11 or 1705.12 specifies special inspection, testing or qualification for seismic resistance, the statement of special inspections shall identify the designated seismic systems and seismic force-resisting systems that are subject to <u>the special inspections or tests</u>

SECTION 1705 REQUIRED VERIFICATION AND SPECIAL INSPECTIONS AND TESTS

1705.1 General. Verification and <u>Special</u> inspections and tests of elements of buildings and structures shall be as required by meet the applicable requirements of this section.

1705.1.1 Special cases. Special inspections and tests 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.

TABLE 1705.2.2

REQUIRED VERIFICATION AND SPECIAL INSPECTIONS OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION TYPE	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
1. Material verification of cold-formed steel deck:			••••••
 a. Identification markings to conform to ASTM standards specified in the approved construction documents 	—	Х	Applicable ASTM material standards
 Manufacturers' certified test reports. 		Х	
2. <u>Special</u> inspection of welding:			
a. Cold–formed steel deck			
 Floor and roof deck welds 	—	Х	AWS D1.3
b. Reinforcing steel:			
 Verification of weldability of reinforcing steel other than ASTM A706 	—	Х	
 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.	Х		
4. Other reinforcing steel		Х	

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspections for seismic resistance.

1705.3 Concrete construction. The Special inspections and verifications for tests of concrete construction shall be as required by performed in accordance with this section and Table 1705.3.

Exception: Special inspections and tests shall not be required for:

(Portions of section not shown remain unchanged)

TABLE 1705.3 REQUIRED VERIFICATION AND SPECIAL INSPECTIONS AND TESTS OF CONCRETE CONSTRUCTION

VERIFICATION	CONTINUOUS	PERIODIC		
AND INSPECTION	<u>SPECIAL</u>	SPECIAL		IBC REFERENCE
TYPE	INSPECTION	INSPECTION	STANDARD	

(Portions of table not shown remain unchanged)

1705.3.1 Materials <u>tests</u>. 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.

1705.4 Masonry construction. <u>Special inspections and tests of</u> masonry construction shall be inspected and verified in accordance with the quality assurance program requirements of TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6 quality assurance program requirements.

Exception: Special inspections and tests shall not be required for:

- Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, where they are part of <u>a</u> structures classified as *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.

1705.4.1 Empirically designed masonry, glass unit masonry and masonry veneer in Risk Category IV. The minimum *special inspection* program *Special inspections* and tests for empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, in <u>where they are part of a</u> structures classified as *Risk Category* IV, in accordance with Section 1604.5, shall comply be performed in accordance with TMS 402/ACI 530/ASCE 5, Level B Quality Assurance.

1705.4.2 Vertical masonry foundation elements. Special inspections and tests of vertical masonry foundation elements shall be performed in accordance with Section 1705.4 for vertical masonry foundation elements.

1705.6 Soils. Special inspections for and tests of existing site soil conditions, fill placement and loadbearing requirements shall be as required by performed in accordance with this section and Table 1705.6. 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 verify 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.

TABLE 1705.6

VERIFICATION AND INSPECTION TASK TYPE CONTINUOUS DURING TASK LISTED SPECIAL INSPECTION	PERIODICALLY DURING TASK LISTED SPECIAL INSPECTION PERIODIC
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(Portions of table not shown remain unchanged)

1705.7 Driven deep foundations. Special inspections and tests shall be performed during installation and testing of driven deep foundation elements as required by specified in Table 1705.7. The approved instruction documents prepared by the registered design professionals, shall be used to determine compliance.

TABLE 1705.7 REQUIRED VERIFICATION AND SPECIAL INSPECTIONS AND TESTS OF DRIVEN DEEP FOUNDATION ELEMENTS

VERIFICATION AND INSEPCTION TASK TYPE	CONTINUOUS DURING TASK LISTED <u>SPECIAL</u> INSPECTION	PERIODICALLY DURING TASK LISTED PERIODIC SPECIAL INSPECTION
 For steel elements, perform additional <u>special</u> inspections in accordance with Section 1705.2. 	-	-
 For concrete elements and concrete-filled elements, perform <u>tests and</u> additional <u>special</u> inspections in accordance with Section 1705.3. 	_	-

(Portions of table not shown remain unchanged)

1705.8 Cast-in-place deep foundations. Special inspections and tests shall be performed during installation and testing of cast-in place deep foundation elements as required by specified in Table 1705.8. The approved geotechnical report, and the construction documents prepared by the registered design professionals, shall be used to determine compliance.

TABLE 1705.8 REQUIRED VERIFICATION AND SPECIAL INSPECTIONS AND TESTS OF CAST-IN-PLACE DEEP FOUNDATION ELEMENTS

VERIFICATION AND INSPECTION TASK TYPE	CONTINUOUS	PERIODICALLY
	DURING TASK	DURING TASK
	LISTED	LISTED
	SPECIAL	SPECIAL
	INSPECTION	INSPECTION
3. For concrete elements, perform tests and additional special		
inspections in accordance with Section 1705.3.	_	_

1705.9 Helical pile foundations. <u>Continuous</u> 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 construction documents prepared by the registered design professional shall be used to determine compliance.

1705.11.1 Structural steel. Special inspection for <u>of</u> structural steel shall be <u>performed</u> in accordance with the quality assurance requirements of AISC 341.

Exception: 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.

1705.13 Sprayed fire-resistant materials. *Special inspections* for and tests of sprayed fire-resistant materials applied to floor, roof and wall assemblies and structural members shall be <u>performed</u> in accordance with Sections 1705.13.1 through 1705.13.6. *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* and tests shall be performed after the rough installation of electrical, automatic sprinkler, mechanical and plumbing systems and suspension systems for ceilings, where applicable.

1705.13.1 Physical and visual tests. The *special inspections* <u>and tests</u> 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.

1705.13.2 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 <u>by the *special inspector*</u> before the application of the sprayed fire-resistant material.

1705.14 Mastic and intumescent fire-resistant coatings. Special inspections <u>and tests</u> for mastic and intumescent fire-resistant coatings applied to structural elements and decks shall be <u>performed</u> in accordance with AWCI 12-B. Special inspections <u>and tests</u> shall be based on the fire-resistance design as designated in the *approved construction documents*.

Revise as follows:

1901.4 Special inspections and tests. The Special inspections and tests of concrete elements of buildings and structures and concreting operations shall be as required by Chapter 17.

PART II - IFC

Revise as follows:

[F] 909.18.8 Special inspections <u>Testing</u> for smoke control. Smoke control systems shall be tested by a special inspector.

[F] 909.18.8.1 Scope of testing. Special inspections <u>Testing</u> shall be conducted in accordance with the following:

- 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.

909.21.7 Special inspection <u>Testing</u>. Special inspection <u>Testing</u> for performance shall be required in accordance with Section 909.18.8. System acceptance shall be in accordance with Section 909.19.

[F] 1705.17 Special inspection <u>Testing</u> for smoke control. Smoke control systems shall be tested by a special inspector.

Reason: The proposal has several purposes. It distinguishes between inspections by the building official and special inspections by special inspectors by adding "special" after "inspection" where special inspections by special inspectors are intended. It adds "tests" after "special inspectors" to recognize that the requirements of Chapter 17 distinguish between (1) special inspections by the special inspector, and (2) tests by the special inspector or other individuals employed or retained by the approved agency at the construction site or testing facilities. It deletes references to "verification," which is considered superfluous given that a primary purpose for inspection, including special inspection, is to verify that the construction complies with the building code and the approved construction documents. It also changes the charging language in several places to state that special inspections and tests shall be "performed" rather than be "as required by" for consistency with the charging language elsewhere in Chapter 17.

tests shall be "performed" rather than be "as required by" for consistency with the charging language elsewhere in Chapter 17. The titles of Tables 1705.3, 1705.6, 1705.7 and 1705.8 are revised to specify tests as well as special inspections due to the tests that are specified in the first column of each table. The columns labeled "continuous" and "periodic" are changed to "continuous special inspection" and "periodic special inspection" because these distinctions apply to special inspections but not to tests. These changes are not made to Table 1705.2.2 because there are no tests specified in the table.

In Section 1705.4.1, "where they are part of" a structure is added for consistency with similar language in Section 1705.4, Exception, Item #1. In Section 1705.17, the title is changed from "special inspection" to "testing" because there are requirements for testing in the section but there are none for special inspection.

An additional benefit of the proposal is that replacement of Table 1705.4 in the 2009 IBC with a reference to TMS 402/ACI 530/ASCE 5 in the 2012 IBC effectively eliminated requirements for special inspection by continuing the use of "inspected." The changes above clarify the intended requirements for special inspection.

Changes to Sections 1705.2 through 1705.2.2 were included in early drafts of this proposal but they were deleted after the changes were incorporated into separate proposals, which was the result of collaboration with the steel industry.

- Note that separate proposals:
 - 1. Further modify Section 1704.2 by changing the title from "special inspections" to "approved agency"

 - Further modify Section 1704.3.2 by deleting "qualification"
 Change "inspection" to "inspections" in Sections 1705.10.1, Exception; 1705.10.2, Exception; 1705.11.2, Exception; and 1705.11.3, Exception
 - Further modify Section 1704.3.1 by deleting Item #1 and 4.
 - Change "inspection" to "inspections" in Sections 1705.2.2 and 1705.2.2.1.2. 5.

S113-12					
PART I - INTER	RNATIONAL	BUILDING CODE	- STRUCTURA	L	
Public Hearing:	Committee	AS	AM	D	
C	Assembly:	ASF	AMF	DF	
PART II – INTE	RNATIONAL	FIRE CODE			
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1703.1.3-S-BRAZIL.doc

S114–12 1703.1, 1703.1.1, 1703.3

Proponent: Phillip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1703.1 Approved agency. An *approved agency* shall provide all information as necessary for the *building official* to determine that the agency meets the applicable requirements <u>specified in Sections</u> <u>1703.1.1 through 1703.1.4</u>.

1703.1.1 Independence. An *approved agency* shall be objective, competent and independent from the contractor responsible for the work being inspected. The agency shall also disclose to the *building official* and the *registered design professional in responsible charge* possible conflicts of interest so that objectivity can be confirmed.

1703.3 Approved Record of approval. For any material, appliance, equipment, system or method of construction that has been *approved*, a record of such approval, including the conditions and limitations of the approval, shall be kept on file in the *building official's* office and shall be open to available for public inspection review at appropriate times.

Reason: Section 1703.1 requires approved agencies to provide the information necessary for the building official to verify that the agency meets the applicable requirements but these requirements are not identified. The proposal specifies the sections containing the requirements.

Section 1703.1.1 requires approved agencies to disclose possible conflicts of interest so that objectivity can be confirmed but the recipient of the disclosure is not identified. The proposal specifies the building official and the registered design professional in responsible charge as the recipients.

Section 1703.3 is clarifies the requirement of the building official to provide access to the public for records of approval.

S114-12			
Public Hearing: Committee:	AS	D	
Assembly.	701	Ы	1703.1-S-BRAZIL.doc

S115-12

104.11.3 (NEW), 104.11.4 (NEW), 104.12 (NEW), 104.12.1 (NEW), 104.12.2 (NEW), 104.12.3 (NEW), 104.12.4 (NEW), 1703.2, 1703.3, 1703.4, 1703.4.1, 1703.4.2, 1703.5, 1703.5.1, 1703.5.2, 1703.5.3, 1703.5.4

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.

Add new text as follows:

104.11.3 Written approval. Any material, design, equipment, or method of construction that has been shown to meet the requirements of this code shall be *approved* in writing after satisfactory completion of the required tests and submission of required test reports.

104.11.4 Approved record. For any material, design, equipment, or method of construction that has been *approved*, a record of such approval, including the conditions and limitations of the approval, shall be kept on file in the *building official's* office and shall be open to public inspection at appropriate times.

104.12 Labeling. Where materials or assemblies are required by this code to be *labeled*, such materials and assemblies shall be *labeled* by an *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 104.11.5.1 through 104.11.5.4.

104.12.1 Testing. An *approved agency* shall test 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.

104.12.2 Inspection and identification. The *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.

104.12.3 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 *approved agency's* identification.

104.12.4 Method of labeling. Information required to be permanently identified on the product shall be acid etched, sand blasted, ceramic fired, laser etched, embossed or of a type that, once applied, cannot be removed without being destroyed.

Delete without substitution:

1703.2 Written approval. Any material, appliance, equipment, system or method of construction meeting the requirements of this code shall be *approved* in writing after satisfactory completion of the required tests and submission of required test reports.

1703.3 Approved record. For any material, appliance, equipment, system or method of construction that has been *approved*, a record of such approval, including the conditions and limitations of the approval, shall be kept on file in the *building official's* office and shall be open to public inspection at appropriate times.

1703.4 Performance. Specific information consisting of test reports conducted by an *approved* testing agency in accordance with the appropriate referenced standards, or other such information as necessary, shall be provided for the *building official* to determine that the material meets the applicable code requirements.

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 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.

1703.5 Labeling. Where materials or assemblies are required by this code to be *labeled*, such materials and assemblies shall be *labeled* by an *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 1703.5.4.

1703.5.1 Testing. An *approved agency* shall test 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 *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 *approved agency's* identification.

1703.5.4 Method of labeling. Information required to be permanently identified on the product shall be acid etched, sand blasted, ceramic fired, laser etched, embossed or of a type that, once applied, cannot be removed without being destroyed.

Reason: Chapter 17 is titled "Special Inspections and Tests" and as such, should be reserved for the special inspection and testing associated with construction projects.

This proposal moves paragraphs from SECTION 1703 APPROVALS to Section 104.11 Alternate materials, design and methods of construction and equipment, which is under SECTION 104 DUTIES AND POWERS OF THE BUILDING OFFICIAL, as these paragraphs deal with the approval of materials and systems not covered by the Code. The language of new Sections 104.11.3, and 104.11.4 has been modified slightly to align with Section 104.11, and old sections 1703.4, 1703.4.1 and 1703.4.2 have been deleted without being moved to Section 104.11, as existing sections 104.11.1 and 104.11.2 already cover research reports and the testing associated with approval of materials and systems not covered by the Code.

Paragraphs dealing with products that require labeling have been moved to a new Section 104.12, without modification. Existing Sections 1703.6, 1703.6.1, and 1703.6.2 have been deleted without being moved to Chapter 1 as the requirements for fabricated assemblies are already covered in Section 1704.2.5, 1704.2.5.1, and 1704.2.5.2.

S115-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1703.2-S-HARMAN.doc

S116–12 1703.4, 1703.4.1, 1703.4.2, 1703.5, 1703.5.1, 1703.5.2, 1703.5.3, 1703.5.4

Proponent: Phillip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1703.4 Performance. Specific information consisting of test reports conducted by an *approved* testing agency in accordance with the appropriate referenced standards, or other such information as necessary, shall be provided for the *building official* to determine that the <u>product</u>, material <u>or assembly</u> meets the applicable code requirements.

1703.4.1 Research and investigation. Sufficient technical data shall be submitted to the *building official* to substantiate the proposed use of any <u>product</u>, 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 <u>product</u> material or assembly subject to the requirements of this code. The costs, reports and investigations required under these provisions shall be paid by the applicant.

1703.4.2 Research reports. Supporting data, where necessary to assist in the approval of <u>products</u>, materials or assemblies not specifically provided for in this code, shall consist of valid research reports from *approved* sources.

1703.5 Labeling. Where materials or assemblies are required by this code to be labeled, such materials and assemblies shall be labeled by an approved agency in accordance with Section 1703. Products, and materials or assemblies required to be labeled shall be labeled in accordance with the procedures set forth in Sections 1703.5.1 through 1703.5.4.

1703.5.1 Testing. An approved agency shall test a representative sample of the product<u>.</u> or material <u>or</u> <u>assembly</u> 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 *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 <u>or assembly</u> is representative of the product, or material <u>or assembly</u> 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 <u>of the product, material or assembly</u> and <u>the</u> *approved agency's* identification.

1703.5.4 Method of labeling. Information required to be permanently identified on the product, <u>material</u> <u>or assembly</u> shall be acid etched, sand blasted, ceramic fired, laser etched, embossed or of a type that, once applied, cannot be removed without being destroyed.

Reason: The purpose for the proposal is to update the language in Sections 1703.4 and 1703.5 by correlating the references to "product," "material" and "assembly" for internal consistency. In Section 1703.5, the first sentence is deleted because it is superfluous given that the requirements for labeling in this section are specified in its subsections and the second sentence is sufficient to serve as charging language for the section.

In Section 1703.5.3, the reference to the distributor is deleted for consistency with the definition of "label" in Section 202, which specifies that the label is applied by the manufacturer. Note that Section 1703.5.3 requires the label to contain the identifications of the manufacturer and the approved agency and this is consistent with the definition of "label" that specifies the same identifications. Note that separate proposals:

1. Delete "testing" from Section 1703.4 so that it reads "...approved agency..." and

Change "the applicant" to "the owner or the owner's authorized agent" in Section 1703.4.1.

S116-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1703.4 #1-S-BRAZIL.doc

S117–12 202, 1703.4, 1704.2.5.2, 1705.16.1, 1705.16.2, [F]909.18.8.2, [F]909.18.8.3, [F]1705.17.2

Proponent: Phillip Brazil P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

THIS IS A THREE PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IBC ADMINISTRATION COMMITTEE. PART III WILL BE HEARD BY THE IFC COMMITTEE AS SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCUTRAL

Revise as follows:

1703.4 Performance. Specific information consisting of test reports conducted by an *approved* testing agency in accordance with the appropriate referenced standards, or other such information as necessary, shall be provided for the *building official* to determine that the material meets the applicable code requirements.

1704.2.5.2 Fabricator approval. Special inspections required by Section 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*.

1705.16.1 Penetration firestops. Inspections of penetration fire-stop systems that are tested and listed in accordance with Sections 714.3.1.2 and 714.4.1.2 shall be conducted by an approved inspection agency in accordance with ASTM E 2174.

1705.16.2 Fire-resistant joint systems. Inspection of fire-resistant joint systems that are tested and listed in accordance with Sections 715.3 and 715.4 shall be conducted by an approved-inspection agency in accordance with ASTM E 2393.

PART II – IBC ADMINISTRATION

Revise as follows:

[A] LABELED. Equipment, materials or products to which has been affixed a *label*, seal, symbol or other identifying *mark* of a nationally recognized testing laboratory, inspection <u>approved</u> agency or other organization concerned with product evaluation that maintains periodic inspection of the production of the above-labeled items and whose labeling indicates either that the equipment, material or product meets identified standards or has been tested and found suitable for a specified purpose.

PART III – IFC

Revise as follows:

[F] 909.18.8.2 Qualifications. *Special inspection* <u>Approved</u> agencies for smoke control shall have expertise in fire protection engineering, mechanical engineering and certification as air balancers.

[F] 909.18.8.3 Reports. A complete report of testing shall be prepared by the special inspector or special inspection <u>approved</u> agency. The report shall include identification of all devices by manufacturer, nameplate data, design values, measured values and identification tag or *mark*. The report shall be reviewed by the responsible *registered design professional* and, when satisfied that the design intent has been achieved, the responsible *registered design professional* shall seal, sign and date the report.

[F] 1705.17.2 Qualifications. *Special inspection agencies Special inspectors* for smoke control shall have expertise in fire protection engineering, mechanical engineering and certification as air balancers.

Reason: The purpose for the proposal is to update references to "approved agency" throughout the building code. Approved agencies (defined in Section 202) are regularly engaged in conducting tests and employ or retain special inspectors (also defined in Section 202) who are qualified to perform inspections, including special inspections.

In Section 1704.2.5.2, "registered" is deleted because no purpose is served by requiring a fabricator who is approved by the building official to also be registered with the same building official.

Note that a separate proposal changes "special inspections" to "testing" in the title of Section 909.18.8 and in Section 909.18.8.1.

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

S117-12

PART I - INTERNATIONAL BUILDING CODE - STRUCTURAL

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

PART II – INTERNATIONAL BUILDING CODE - ADMINISTRATION

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
PART III – INTERNATIONAL	FIRE COD	θE		
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	

1703.4 #2-S-BRAZIL.doc

S118–12 1704.1, 1704.2.5.2, 1704.5 (New), 1705.12.3, 1910.5, 2207.5

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.1 General. This section provides minimum requirements for special inspections, the statement of special inspections, contractor responsibility, <u>submittals to the *building official*</u> and structural observations.

1704.2.5.2 Fabricator approval. *Special inspections* required by Section 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 owner or the owner's authorized agent for submittal to the *building official* as specified in Section 1704.5 stating that the work was performed in accordance with the *approved construction documents*.

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design professional* and prior to the construction or work being performed for each of the following:

- Certificates of compliance for the fabrication of structural, load-bearing or lateral load-resisting members or assemblies on the premises of an approved fabricator in accordance with Section 1704.2.5.2
- 2. Certificates of compliance for the seismic qualification of nonstructural components, supports and attachments in accordance with Section 1705.12.3
- 3. Certificates of compliance for designated seismic systems in accordance with Section 1705.12.4
- 4. Reports of preconstruction tests for shotcrete in accordance with Section 1910.5
- 5. Certificates of compliance for open web steel joists and joist girders in accordance with Section 2207.5

(Renumber subsequent sections)

1705.12.3 Seismic certification of nonstructural components. The *registered design professional* shall specify on the construction documents the 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 1705.12. <u>Certificates of compliance shall be submitted to the *building official* as specified in Section 1704.5.</u>

Revise as follows:

1910.5 Preconstruction tests. When Where preconstruction tests are required by the *building official* <u>Section 1910.4</u>, a test panel shall be shot, cured, cored or sawn, examined and tested prior to commencement of the project. The sample panel shall be representative of the project and simulate job conditions as closely as possible. The panel thickness and reinforcing shall reproduce the thickest and most congested area specified in the structural design. It shall be shot at the same angle, using the same nozzleman and with the same concrete mix design that will be used on the project. The equipment used in preconstruction testing shall be the same equipment used in the work requiring such testing, unless substitute equipment is *approved* by the *building official*. Reports of preconstruction tests shall be submitted to the *building official* as specified in Section 1704.5.

Revise as follows:

2207.5 Certification. At completion of manufacture, the steel joist manufacturer shall submit a *certificate* of compliance in accordance with to the owner or the owner's authorized agent for submittal to the <u>building official as specified in</u> Section 1704.2.5.2 1704.5 stating that work was performed in accordance with approved construction documents and with SJI standard specifications.

Reason: The purpose for the proposal is to provide a new section (Section 1704.5) in the building code that comprehensively specifies the requirements for the submittal of reports and certificates related to construction that is subject to special inspections and tests required by Chapter 17 of the building code. Typically, these documents certify or otherwise verify that a material or product meets certain special requirements, or are alternatives to the general requirements, of the building code.

The items in new Section 1704.5 are typically references to provisions elsewhere in the building code or a referenced standard. The charging language of the new section specifies the requirements for submittal to the building official (e.g., by whom, after review and acceptance, and before the work begins) and the requirements apply equally to each listed submittal. The referenced provisions, however, contain additional requirements unique to each situation. The proposal modifies these provisions to be consistent with the submittal requirements in new Section 1704.5. For example, Item 2 requires submittal of the certificate of conformance "in accordance with Section 1705.12.3." Section 1705.12.3, in turn, requires submittal of the certificate of conformance "to the building official as specified in Section 1704.5." Similar language is found in Item 4 and corresponding Section 1910.5.

Item 1 is similar to Item 2 in that it requires submittal of the certificate of conformance "in accordance with Section 1704.2.5.2." Section 1704.2.5.2, however, requires submittal of the certificate of conformance to "the owner or the owner's authorized agent for submittal to the building official as specified in Section 1704.5...". This is because of the requirement in Section 1704.2.5.2 for submittal of the certificate of compliance by the approved fabricator and is done to avoid a conflict with new Section 1704.5. Similar language is found in Item 5 of new Section 1704.5 and corresponding Section 2207.5.

The charging statement in new Section 1704.5 states that the submittals are in addition to the submittal of reports of special inspections and tests because also listing them in the new section is not needed since this activity is already covered in Section 1704.2.4. It is also not advisable because the submittal of reports of special inspections and tests is the responsibility of approved agencies but the submittals listed in this new section are the responsibility of the owner or owner's authorized agent. Examples of reports of special inspections and tests of submitted by approved agencies are: tests of concrete for strength, slump and air content (see Table 1705.3); tests of masonry units, grout and mortar (see Section 1705.4); and strength tests of shotcrete (see Table 1705.3).

Item 4 is included in new Section 1704.5 because the preconstruction tests required by Section 1910.4 are not also a requirement in Chapter 17 of the building code and requiring the submittal of test reports to the building official will enable the building official to verify, before construction begins, the validity of structural design assumptions based on the success of the preconstruction tests. Text requiring the submittal of the test reports to the building official is added to Section 1910.5 in conjunction with Item 4.

For Items 2 and 3 of new Section 1704.5, a separate proposal places the provisions of Section 1705.12.3 into two subsections (Sections 1705.12.3 and 1705.12.4) to provide effective charging language for the corresponding provisions in ASCE 7-10. In that proposal, requirements for the submittal of certificates of compliance to the building official are added to each subsection. This proposal for a new Section 1704.5 also adds a similar requirement to Section 1705.12.3 but the only purpose for doing so is to specify Section 1704.5. Should both proposals be approved by the ICC membership, our intent is that Section 1705.12.3 reads: "Certificates of compliance for the seismic qualification shall be submitted to the building official as specified in Section 1704.5;" and Section 1705.12.4 reads: "Certificates of compliance documenting that the requirements are met shall be submitted to the building official as specified in Section 1704.5."

Note that separate proposals:

- 1. Transfer the requirements of Section 1705.12.1 to new Section 1704.5;
- 2. Add additional requirements for submittals that are related to structural steel ;
- 3. Correlate the language in Section 1704.2.5 with the definition of "fabricated item" in Section 202;
- 4. Add additional requirements for submittals that are related to the welding of concrete reinforcement and anchor bolts;
- 5. Add additional requirements for submittals that are related to masonry;
- Change "the owner" to "the owner or the owner's authorized agent";
- 7. Add a new Section 107.1.1 that correlates with this proposal: and
- 8. Add "responsible" before "registered design professional".

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

S118-12

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Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
· · · · ·				1704.1 #1-S-BRAZIL.doc

S119–12 1704.1

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.1 General. This section provides minimum requirements for Special inspections, the statements of special inspections, responsibilities of contractors responsibility and structural observations shall meet the applicable requirements of this section.

Reason: The changes revise the language from being declarative to being mandatory.

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

S119-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1704.1 #2-S-BRAZIL.doc

S120–12 1704.2

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

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, other than the contractor, shall employ one or more *approved agencies* to perform inspections during construction on the types of work listed under Section 1705. These inspections are in addition to the inspections identified in Section 110.

Exceptions:

- 1. Special inspections are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as *approved* by the *building official*.
- 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.
- 3. Special inspections are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.
- 4. The contractor is permitted to employ the approved agencies where the contractor is also the owner.

Reason: The purpose for the proposal is to delete the requirement that only the registered design professional in responsible charge is permitted to serve as the owner's agent for employing an approved agency to perform special inspections and tests required by Section 1705 of the building code. We are not aware of any abilities of registered design professionals in responsible charge that make them uniquely qualified for this role.

The purpose for adding language to prohibit the contractor from employing the approved agencies is to prevent the contractor from serving as the owner's agent. The employment of approved agencies should be the responsibility of the owner. The contractor should not perform this function to avoid potential conflicts of interest. Note that Section 1703.1.1 requires the approved agency to be independent from the contractor responsible for the work being inspected. Exception #4 is added, however, to permit the contractor to do so where the contractor is also the owner.

Note that a separate proposal adds "authorized" before "agent" in Section 1704.2.

S120-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					1704 2 #1-S-BRAZIL doc
S121–12 1704.2, 1704.2.1, 1704.2.4

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

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 provide* inspections during construction on the types of work listed under Section 1705 and identify them to the *building official*. These inspections are in addition to the inspections identified in Section 110.

Exceptions:

- 1. Special inspections are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as *approved* by the *building official*.
- 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.
- 3. Special inspections are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.

1704.2.1 Special inspector qualifications. Prior to the start of the construction, the special inspector approved agencies shall provide written documentation to the building official demonstrating his or her the competence and relevant experience or training of the special inspectors who will perform the special inspections and tests during construction. 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. 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 they qualify as special inspectors.

1704.2.4 Report requirement. Special inspectors <u>Approved agencies</u> shall keep records of inspections. The special inspector <u>approved agency</u> 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*.

Reason: Section 1704.2 requires the owner or owner's agent to employ approved agencies to perform special inspections and tests required by Section 1705. The act of an owner or owner's agent to employ an approved agency for this purpose, however, is a private matter (typically contractual) and not an appropriate subject for a building code that requires compliance with its provisions. The proposal revises the language to require the owner or owner's agent to identify to the building official the approved agencies who will provide the special inspections and tests required by Section 1705 that will be performed by special inspectors and others (e.g., testing lab personnel) employed or retained by the approved agency.

Section 1704.2.1 requires special inspectors to provide documentation of their qualifications to the building official but it does not specify when this is required to occur. Being a subsection of Section 1704.2, Section 1704.2.1 also does not specify the relationship between the special inspector providing documentation of qualifications and the owner or owner's agent employing an approved agency. Special inspectors are employed or retained by an approved agency to perform special inspections (see definition of "special inspector" in Section 202). The proposal revises the language to require the approved agency to provide to the building

official prior to the start of construction documentation of the qualifications for the special inspectors who will perform the special inspections and tests during construction.

An example of written documentation demonstrating the competence and relevant experience of an approved agency would be evidence of accreditation as an approved agency by the International Accreditation Service (IAS), Inc. The requirements for obtaining and maintaining such accreditation from the IAS are in the Accreditation Criteria for Special inspection Agencies, AC291. Notable provisions in AC291 are definitions, many of which are from 2012 IBC Section 202 (Section 2); information required to be submitted by the agency for accreditation (Section 3); requirements for inspection reports issued by the agency, including compliance with the reporting requirements of IBC Chapter 17 (Section 4); requirements for training, supervision and monitoring of special inspectors (Section 5); and minimum qualifications of special inspectors for specific classes of construction, including those in 2012 IBC Section 1705 (Section 6).

Section 1704.2.4 requires special inspectors to keep records of inspections and furnish inspection reports to the building official and the registered design professional in responsible charge. Special inspectors do generate records of their actions but these are typically kept for submittal by the approved agency that employs or retains them. Section 1704.2.4 is changed to require approved agencies to keep records of special inspections and tests and to submit the reports to the building official and the registered design professional in responsible charge.

Note that separate proposals also revise Section 1704.2 to:

- 1. Distinguish between special inspections and tests by approved agencies and inspections by the building official;
- 2. Clarify that the application is made to the building official as specified in Section 105; and
- 3. Update references to "approved agency" throughout the building code, including instances of "special inspection agency".

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

S121-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				1704.2 #2-S-BRAZIL.doc

S122–12 202, 1704.2

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.2 Special inspections Approved agency. Where application is made for to the building official construction as described in this specified in Section 105, 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 specified in Section 1705. These inspections are in addition to the inspections identified in Section 110.

Exceptions:

- 1. Special inspections are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as *approved* by the *building official*.
- 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.
- 3. Special inspections are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.

Revise as follows:

SECTION 202 DEFINITIONS

STRUCTURAL OBSERVATION. The visual observation of the structural system by a *registered design professional* for general conformance to the *approved construction documents*. Structural observation does not include or waive the responsibility for the inspections required by <u>in</u> Section 110, <u>or the special</u> <u>inspections in Section</u> 1705 or other sections of this code.

Reason: The current language in Section 1704.2 references that section for requirements applicable to applications for construction but Section 1704.2 contains no such requirements. The requirements for applications to the building official for permits are located in Section 105.

The definition of structural observation is revised because the current language refers to inspections in Section 110, Section 1705 or other sections of the code but Section 110 specifies inspections to be performed by the building official, Section 1705 specifies special inspections to be performed by an approved agency, and there are no other sections in the *International Building Code* with inspections or special inspections to reference other than for smoke control systems, which are not subject to structural observations. The changes will also make the definition consistent with the last sentence of Section 1704.2 ("in addition to the inspections specified in Section 110").

Note that a separate proposal also revises Section 1704.2 to distinguish between special inspections and tests by approved agencies and inspections by the building official.

S122-12					
Public Hearing:	Committee:	AS	AM	D	
Ū	Assembly:	ASF	AMF	DF	
	,				1704.2 #3-S-BRAZIL.doc

S123–12 1704.2.5, 1704.2.5.1, 1704.2.5.2, 1705.10 (NEW)

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1704.2.5 <u>Special</u> inspection of fabricators <u>fabricated items</u>. Where fabrication of structural loadbearing members and assemblies is being performed <u>conducted</u> 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 performed during fabrication.

Exceptions:

1704.2.5.1 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:

2. Special inspections as required by Section 1704.2.5 shall are not be required where the fabricator is registered and approved in accordance with Section 1704.2.5.2.

1704.2.5.2 <u>**1704.2.5.1**</u> **Fabricator approval.** Special inspections required by Section 1705 during fabrication 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*.

1705.10 Fabricated items. Special inspections of fabricated items shall be performed in accordance with Section 1704.2.5.

(Renumber subsequent sections)

Reason: Section 1704.2.5 requires special inspections to be performed for all structural load-bearing members and assemblies that are fabricated on the premises of a fabricator's shop (e.g., not at the construction site) as specified in the section and elsewhere in the building code. One example of this is the fabrication of metal-plate-connected wood trusses, which is subject to the special inspections required by Section 1704.2.5. Special inspections of the installation of the trusses at the construction site is not required except for trusses spanning 60 feet or greater (Section 1705.5.2).

A second example is the fabrication of precast, prestressed, concrete members (e.g., hollow-core slabs), which is also subject to the special inspections required by Section 1704.2.5 as well as those of Section 1705.3 for concrete construction. Note that Item 9 of Table 1705.3 specifies inspection of prestressed concrete.

Section 1704.2.5 requires special inspections of the fabricated items. Section 1704.2.5.1 specifies duties of the special inspector but these duties are not directly related to special inspections of the fabricated items. Instead, the specified duties are typical of what is conducted by an approved agency for the accreditation of a fabricator by a nationally recognized accreditation service such as the International Accreditation Service. Based on Section 1704.2.5, these duties are required in addition to special inspections of the fabricated items that are required elsewhere in the building code, such as for precast, prestressed, concrete members.

The proposal modifies the provisions in Section 1704.2.5 by requiring special inspections of fabricated items during fabrication. Section 1704.2.5.1 is changed to an exception making it an alternative to the basic requirement for special inspection in Section 1704.2.5.

The other changes in the proposal are made to clarify the language. Section 1705.10 is added because Section 1704.2.5 requires special inspections except where the work is done on the premises of an approved fabricator (Section 1704.2.5.2) and should be included in Section 1705, which specifies required special inspection and tests.

The current provisions in Section 1704.2.5.2 (renumbered to Section 1704.2.5.1 are an acknowledgement that there are fabricators who (1) fabricate products or assemblies with sufficient quality and through the application of documented procedures (e.g., quality management systems), and (2) and are recognized for this through certification, accreditation or qualification by a national recognized organization providing such services, that they should be exempt from further requirements for special inspection of fabrication. Examples are:

- 1. The certification program of steel fabricators and erectors by the American Institute of Steel Construction (AISC), which is audited by the Quality Management Company;
- The accreditation of the fabrication inspection programs for reinforced concrete and precast/prestressed concrete, structural steel and wood wall panels by the International Accreditation Service (IAS) (see AC157, AC172 and AC196, respectively, for accreditation criteria);
- The accreditation of the inspection programs for manufacturers of metal building systems by the International Accreditation Service (IAS) (see AC472 for accreditation criteria); and
- 4. Qualification of prefabricated items such as prefabricated wood shear panels, cold-formed, pin-connected open-web trusses with wood chords and tubular or angular steel webs, and steel lateral-force-resisting vertical assemblies, as alternatives to applicable requirements in the IBC or other codes by the ICC Evaluation Service (ICC-ES) (see AC130, AC306 and AC322, respectively, for acceptance criteria).
- 5. The certification of structural and architectural concrete products by the Precast, Prestressed Concrete Institute (PCI).
- 6. The certification of precast concrete products by the National Precast Concrete Association (NPCA).

Note that separate proposals:

- 1. Revise Section 1704.2.5.2 to specify that the approved fabricator is required to submit the certificate of compliance to the owner or the owner's authorized agent in conjunction with the requirement in proposed Section 1704.5 for submittal of the certificate to the building official;
- Revise Sections 1704.2.5 and 1704.2.5.1 for consistency with and to correlate with the definition of "fabricated item" in Section 202: and
- 3. Revise Section 1704.2.5.2 and other sections to update references to "approved agency" throughout the building code.

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

S123-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1704.2.5 #1-S-BRAZIL.doc

S124–12 202, 1704.2.5, 1704.2.5.1

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.2.5 Inspection of fabricators. Where fabrication of structural, load-bearing <u>or lateral load-resisting</u> members <u>and or</u> assemblies is being <u>performed conducted</u> on the premises of a fabricator's shop, *special inspection* of the fabricated items shall be <u>performed as</u> required by this section and as required elsewhere in this code.

1704.2.5.1 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 this code. The special inspector shall review the procedures for completeness and adequacy relative to the code requirements for <u>applicable to</u> the fabricator's scope of work.

Exception: Special inspections as required by Section 1704.2.5 shall not be required where the fabricator is *approved* in accordance with Section 1704.2.5.2.

Revise as follows:

SECTION 202 DEFINITIONS

FABRICATED ITEM. Structural, load-bearing or lateral load-resisting <u>members or</u> 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 standards <u>specifications</u> 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 which that provides requirements for quality control done under the supervisions of a third-party quality control agency, shall not be considered <u>are not</u> "fabricated items."

Reason: The purpose for the proposal is to correlate the provisions for fabrication on the premises of a fabricator's shop. Section 1704.2.5 and the definition of "fabricated item" in Section 202 are revised for internal consistency. The change from "shall not be" to "are not" in the definition of "fabricated item" eliminates mandatory language, which is not appropriate in a definition. Also, "specifications" is deleted because the building code references standards, not specifications.

In Section 1704.2.5.1, "referenced standards" is replaced with "this code" for consistency with Section 102.4, which establishes that standards referenced by the building code are considered part of the code's requirements to the prescribed extent of the standard. The other changes are made because there are no requirements in the building code for the fabricator's scope of work and the requirements applicable to the fabricator are not limited to the requirements in the building code but also include what is specified in the approved construction documents.

S124-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1704.2.5 #2-S-BRAZIL.doc

S125–12 1704.2.5, 1704.2.5.1, 1704.2.5.2

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1704.2.5 Inspection of fabricators<u>fabricated and pre-fabricated items</u>. 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.

1704.2.5.1 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 1704.2.5 shall not be required where the fabricator is approved in accordance with Section 1704.2.5.2.

1704.2.5.2 <u>**1704.2.5.1**</u> **Fabricator approval.** Special inspections required by <u>Section 1704.2.5 and</u> Section 1705, <u>except Sections 1705.10</u>, <u>1705.11</u> and <u>1705.12</u> are not required where the work is done on the premises of a fabricator registered and *approved* to perform such work without *special inspection*. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an *approved special inspection* agency. At completion of fabrication, the *approved* fabricator shall submit a *certificate of compliance* to the *building official* stating that the work was performed in accordance with the *approved construction documents*.

Reason: The proposed change in 1704.2.5 makes it clear that the special inspection is of the fabricated item, not the fabricator. The addition of the word "pre-fabricated" is needed due to the use of the word in 1703.6, and 1705.5. A related code change proposal adds a definition of "prefabricated item", equating it with "fabricated item."

Section 1704.2.5.1 is deleted because it is often confused with the review of the fabricator's quality control procedures that is done as part of the process of approving fabricators to perform work without special inspection. That task should only be done by a qualified auditor when the fabricator is seeking "approved fabricator" status in accordance with 1704.2.5.2 (here renumbered as 1704.2.5.1). As 1704.2.5 requires special inspection of the items being fabricated, verification of the fabricator's quality processes is not needed.

The exception to 1704.2.5.1 is deleted because the exception is adequately covered in 1704.2.5.2 (now renumbered as 1704.2.5.1).

The revision to the first sentence of 1704.2.5.2 is needed because the reorganization of Chapter 17 effected in the last codechange cycle merged all the special inspection requirements, including those for wind and seismic resistance, into Section 1705. The reference to Section 1705 would then allow the waiver of special inspection when work is performed in an approved fabricator's shop to be applicable to the wind-force resisting system in high wind areas and to the seismic force-resisting system. Code Change Proposal S109 07/08 specifically changed this section to clarify that the waiver would not apply to seismic.

Cost Impact: The code change proposal will not increase the cost of construction and may decrease the cost of construction in jurisdictions where the special inspector was performing the verification required by 1704.2.5.1.

S125-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1704.2.5-S-KERR.doc

S126–12 1704.2.5, 1704.2.5.1, 1704.2.5.2

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute, representing American Institute of Steel Construction (bmanley@steel.org)

Revise as follows:

1704.2.5 <u>**1704.3**</u> **Inspection of fabricators** <u>and fabricated items</u>. Where fabrication of structural loadbearing 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.

Exception: Special inspections as required by Section 1704.3.1 and Section 1705, except Sections 1705.10, 1705.11 and 1705.12, are not required where the work is done on the premises of a fabricator approved in accordance with Section 1704.3.2 to perform such work without special inspection.

1704.2.5.1 1704.3.1 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 1704.2.5 shall not be required where the fabricator is approved in accordance with Section 1704.2.5.2.

1704.2.5.2 <u>1704.3.2</u> Fabricator approval. *Special inspections* required by Section 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*.

(Renumber subsequent sections)

Reason: This modification corrects the unintended consequences of modifications made by Proposal S116-09/10, effective with IBC 2012, which reorganized Chapter 17 and combined all special inspections and tests into Section 1705, including requirements for additional inspection and testing for wind resistance and seismic resistance. Previously, special inspections for wind resistance and seismic resistance had not been subject to the waiver of special inspections under the approved fabricators provisions, as demonstrated by the modifications made under Proposal S109-07/08, which appears in IBC 2009.

This modification also corrects the unintended consequences of modifications made by Proposal S116-09/10, effective with IBC 2012, which reorganized Section 1704, combining the provisions on inspection of fabricators, approved fabricators, and waiver of special inspections into the same numbering set as the general special inspection provisions.

This is first shown to be an unintended consequence with a modification made through Proposal S109–07/08 regarding 1704.2.2, which added the specific reference to Section 1704 into 1704.2.2, with the reason stated as follows:

"This modification attempts to clarify exactly which inspections are permitted to be waived when work is done by a registered and approved fabricator. As written now, it could be interpreted to mean that the special inspections for seismic resistance required by Section 1707.2 could be waived. This is not appropriate and needs to be corrected." This is also shown to be an unintended consequence in that IBC 2009 stated:

"Section 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 ...

Section 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

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: ... "

In IBC 2009, Section 1704.1 did not include the Approved Fabricator provisions, which were located in 1704.2.

With the reorganization for IBC 2012, the Approved Fabricator provisions were combined into the same number sequence as the previous 1704.1, with new provisions stated as follows:

"1705.10 Special inspections for wind resistance. Special inspections itemized in Sections 1705.10.1 through 1705.10.3, unless exempted by the exceptions to Section 1704.2, are required for buildings and structures constructed in the following areas: ...

1705.11 Special inspections for seismic resistance. Special inspections itemized in Sections 1705.11.1 through 1705.11.8, unless exempted by the exceptions of Section 1704.2, are required for the following: ...

1705.12 Testing and qualification for seismic resistance. The testing and qualification specified in Sections 1705.12.1 through 1705.12.4, unless exempted from special inspections by the exceptions of Section 1704.2 are required as follows: ... "

Therefore, for clarity, the Approved Fabricator provisions that were once distinct need to be renumbered separately from 1704.2 to avoid confusion with the provisions of 1705.10, 1705.11 and 1705.12.

The language in 1704.2.5 led to confusion about whether the waiver of special inspections applied only to the review of the fabricator's procedures, the fabricated items, or both, as the exception appeared in Section 1704.2.5.1. Secondly, the exception was repeated in a different manner in Section 1704.2.5.2. The title of 1704.2.5 addressed fabricators only, and not the fabricated items. The term "registered" is not used for such purposes within the code, and therefore is deleted. The new organization and the combination of statements regarding waiver of special inspections is intended to resolve this confusion.

Cost Impact: This change will increase cost of fabricated items that fall under the requirements for additional inspection and testing for wind resistance and seismic resistance.

S126-12

Public Hearing: Committee:	46	A N /	П	
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Assembly:	ASF	AMF	DF	
				1704.2.5-S-MANLEY.doc

S127–12 1704.3.1

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.3.1 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.

(Renumber remaining items)

Reason: The purpose for the proposal is to delete the requirement that the statement of special inspections specify the special inspection or testing required to be performed by the building official or the registered design professional responsible for each portion of the work. The building official is required to perform inspections, not special inspections or tests, which are required to be performed by special inspectors employed or retained by approved agencies. IBC Section 104.4 requires the building official to perform all required inspections but the building official is permitted to accept certified reports of inspections by approved agencies or responsible individuals.

Section 1704.2 requires the owner or owner's agent to employ approved agencies to perform special inspections and tests required by Section 1705. There is no requirement in the building code for registered design professionals to perform special inspections or tests but Section 1704.2.1 permits them to act as special inspectors, provided they demonstrate in writing their "competence and relevant experience or training" to the building official. Section 1704.2.1 also permits the registered design professional in responsible charge and engineers of record involved in the design of the project to act as the approved agency and their personnel to act as special inspectors for the work they designed, provided they qualify as special inspectors. Qualification as special inspectors requires the same demonstration of "competence and relevant experience or training" as noted above.

The language in Section 1704.2.1 serves as an alternative to the requirement in Section 1704.2 for the owner or owner's agent to employ approved agencies to perform special inspections and tests required by Section 1705. Based on its definition in Section 202, an approved agency is "established and recognized" as being "regularly engaged in conducting tests or furnishing inspection services." Registered design professionals in responsible charge and engineers of record involved in the design of the project may not be so "established and recognized" but they are permitted to serve as an approved agency and their personnel are permitted to act as special inspectors for the work they designed, provided they demonstrate their qualifications to the building official. However, this does not establish a requirement for registered design professionals to perform special inspections or tests as specified in Item 1 of Section 1704.3.1.

Note that a separate proposal modifies current Item #4 of Section 1704.3.1 by changing "special inspection or testing" to "special inspections or tests".

S127-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1704 3 1-S-BRAZIL doc

S128-12

1704.3.1

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1704.3.1 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 1705.10, 1705.11 and 1705.12.
- 5. For each type of *special inspection*, identification as to whether it will be continuous *special inspection*, or periodic *special inspection*, or performed at a frequency in accordance with the notation used in the reference standard where the inspections are defined.

Reason: The quality assurance requirements of AISC 360 and AISC 341, which are referenced as the standard for special inspections and testing for structural steel, do not describe the frequency of the inspections as "periodic" or "continuous." Rather, detailed inspection tasks are defined, and the level of effort for each task is described by the terms "Observe" and "Perform". This proposal accommodates this alternate approach to the frequency of special inspection.

S128-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1704.3.1-S-KERR.doc

S129–12 1704.3.2, 1705.11.4, 1705.12, 1705.12.3, 1705.12.4 (New)

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.3.2 Seismic requirements in the statement of special inspections. Where Section 1705.11 or 1705.12 specifies special inspections, testing or qualification or tests for seismic resistance, the statement of special inspections shall identify the designated seismic systems and seismic force-resisting systems that are subject to the special inspections or tests.

1705.11.4 Designated seismic systems. The special inspector shall examine designated seismic systems requiring seismic qualification in accordance with Section 1705.12.3 <u>13.2.2 of ASCE 7</u> and verify that the *label*, anchorage or <u>and</u> mounting conforms to the *certificate of compliance*.

1705.12 Testing and qualification for seismic resistance. The Testing and qualification for seismic resistance is required as specified in Sections 1705.12.1 through 1705.12.4 <u>1705.12.5</u>, unless exempted from *special inspections* by the exceptions of Section 1704.2 are required as follows:

- 1. The seismic force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F shall meet the requirements of Sections 1705.12.1 and 1705.12.2, as applicable.
- Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F and subject to the certification requirements of ASCE 7 Section 13.2.2 shall comply with Section 1705.12.3.
- 3. Architectural, mechanical and electrical components in structures assigned to *Seismic Design Category* C, D, E or F and where the requirements of ASCE 7 Section 13.2.1 are met by submittal of manufacturer's certification, in accordance with Item 2 therein, shall comply with Section 1705.12.3.
- 4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1705.12.4.

1705.12.3 Seismic certification of Nonstructural components. For structures assigned to Seismic Design Category B, C, D, E or F, where the requirements of Section 13.2.1 of ASCE 7 for nonstructural components, supports or attachments are met by seismic qualification as specified in Item 2 therein, the registered design professional shall specify on the construction documents the requirements for certification seismic qualification 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 1705.12 Certificates of compliance for the seismic qualification shall be submitted to the building official.

1705.12.4 Designated seismic systems. For *structures* assigned to *Seismic Design Category* C, D, E or F and with *designated seismic systems* that are subject to the requirements of Section 13.2.2 of ASCE 7 for certification, the *registered design professional* shall specify on the *construction documents* the requirements to be met by analysis, testing or experience data as specified therein. *Certificates of compliance* documenting that the requirements are met shall be submitted to the *building official*.

(Renumber subsequent sections)

Reason: The provisions in Section 1705.12.3 are placed in two sections to provide effective charging language for the corresponding provisions in ASCE 7-10 for nonstructural components meeting special requirements and designated seismic systems, which differ substantially from each other. References to "certification" and "qualification" in this section as well as other sections in the proposal are also revised for consistency with the corresponding provisions of ASCE 7-10. Seismic qualification and certification are technical requirements that are covered by the provisions in ASCE 7-10 (Sections 13.2.1 and 13.2.2). What is relevant in the building code is the submittal of certificates of compliance (manufacturer's certification in ASCE 7-10) to the building

official for verification that the requirements for seismic qualification and certification are met and language is added to both sections for this purpose.

The requirement to submit certificates of compliance to the building official is also added to both sections for consistency with corresponding language in ASCE 7-10. Items #1 and #2 in Section 13.2.2 of ASCE 7-10 both specify submittal "for approval to the authority having jurisdiction after review and acceptance by a registered design professional." Item #1 in Section 13.2.1 of ASCE 7-10 contains similar language. Item #2 in Section 13.2.1, however, specifies submittal but not to whom. This has been judged to be an oversight on the part of the ASCE 7 Committee whose membership includes two members of the WABO Technical Code Development Committee. This has been brought to the attention of the ASCE 7 Committee and a proposal that addresses the issue will be submitted for consideration in the next development cycle for the standard.

The current language in Section 1705.12.3 for the registered design professional to specify on the construction documents the requirements to be met by analysis, testing or experience data is not substantively changed by this proposal.

Also in Section 1705.12.3, the scope is expanded to include structures assigned to Seismic Category B. For a nonstructural component in a structure where the option of seismic qualification by analysis, testing or experience data in Section 13.2.1, Item 2, of ASCE 7-10 is chosen, the requirements to document the parameters for seismic qualification on the construction documents and to submit the certificate of compliance for seismic qualification to the building official will apply. These requirements, however, are the consequence of the owner, design team or construction team choosing to comply with Section 13.2.1 of ASCE 7-10 through seismic qualification rather than the design option in Section 13.2.1, Item 1 of ASCE 7-10.

Note that a separate proposal modifies the requirement in Sections 1705.12.3 and 1705.12.4 to submit certificates of compliance for consistency with the changes in that proposal by stating that they shall be submitted to the building official "as specified in Section 1704.5".

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

S129-12

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Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1704.3.2-S-BRAZIL.doc

S130–12 1704.3.3, 1705.10

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1704.3.3 Wind requirements in the statement of special inspections. Where Section 1705.10 specifies special inspection for wind requirements resistance, the statement of special inspections shall identify the main windforce-resisting systems and wind-resisting components that are subject to special inspection.

1705.10 Special inspections for wind resistance. Special inspections itemized for wind resistance specified in Sections 1705.10.1 through 1705.10.3, unless exempted by the exceptions to Section 1704.2, are required for buildings and structures constructed in the following areas:

- In wind Exposure Category B, where V_{asd} as determined in accordance with Section 1609.3.1 is 120 miles per hour (52.8 m/sec) or greater.
- 2. In wind Exposure Category C or D, where V_{asd} as determined in accordance with Section 1609.3.1 is 110 mph (49 m/sec) or greater.

Reason: The purpose for the proposal is to correlate the language that specifies special inspections for wind resistance with separate proposals that make similar changes to Section 1705.11 on special inspections for seismic resistance and to Section 1705.12 on testing for seismic resistance.

S130-12					
Public Hearing: Co	ommittee:	AS	AM	D	
As	sembly:	ASF	AMF	DF	
	•				1704.3.3-S-BRAZIL.doc

S131–12 202, 1704.5

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

Revise as follows:

SECTION 202 DEFINITIONS

STRUCTURAL OBSERVATION. The visual observation of the structural system by a *registered design professional* for general conformance to the *approved construction documents*. Structural observation does not include or waive the responsibility for the inspection required by Section 110, 1705 or other sections of this code.

Revise as follows:

1704.5 Structural observations. Where required by the provisions of Section 1704.5.1 or 1704.5.2, the owner shall employ a *registered design professional* to perform structural observations as defined in Section 202 Chapter 2. Structural observation does not include or waive the responsibility for the inspections in Section 110 or the *special inspections* in Section 1705.

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

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

Reason: The last sentence of the definition in section 202 is moved to section 1704.5 because rules and relationships to other requirements should not be in the definition. The sentence is slightly revised to distinguish between "inspections" and *special inspections* and the reference to "other sections of this code" is deleted as there are no other sections that deal with inspections. The first sentence in 1704.5 is revised to make reference to Chapter 2 where definitions are now located.

S131-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1704 5 #1-S-HARMAN doc

S132–12 1704.5, 1704.5.1, 1704.5.2

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

Revise as follows:

1704.5 Structural observations. Where required by the provisions of Section 1704.5.1 or 1704.5.2, the owner shall employ a *registered design professional* to perform structural observations as defined in Section 202 Chapter 2.

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.

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 where one or more of the following conditions exist:

- 1. The structure is classified as *Risk 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 Risk 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*.

1704.5.2 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where V_{ast} as determined in accordance with Section 1609.3.1 exceeds 110 mph (49 m/sec), where one or more of the following conditions exist:

- 1. The structure is classified as 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.

1704.5.1 Structural observations of the structural system. *Structural observations* shall be provided where one or more of the following conditions exist:

- 1. The *building height*, or the height above the *grade plane* to the uppermost structural level of a non-building structure, is greater than 75 feet (22860 mm).
- 2. The structure has an occupant load greater than 500.
- 3. The structure is classified as *Risk Category* III in accordance with Table 1604.5, and is assigned to *Seismic Design Category* D, E, or F.
- The structure is classified as *Risk Category* III in accordance with Table 1604.5, and is sited where V_{asd} as determined in accordance with Section 1609.3.1 exceeds 110 mph (49m/sec).
- 5. The structure is classified as *Risk Category* I or II in accordance with Table 1604.5, is assigned to Seismic Design Category E, or F and is greater than two stories above the grade plane.
- 6. The structure is classified as Risk Category IV in accordance with Table 1604.5.
- 7. Where required by the registered design professional responsible for the structural design.
- 8. Where such observation is specifically required by the building official.

Reason: Currently the code requires structural observation only in the limited situations of tall buildings or higher risk category structures located in high seismic and wind areas. It is the opinion of the National Council of Structural Engineers Associations that structural observation should be required for all large, or important, buildings anywhere in the country. It is well established that the quality of construction is increased when the engineer who designed the structure can verify that key construction conditions are in conformance with the design intent. Structural observation is meant to augment the detailed inspection provided by the special inspectors. It should be required wherever the consequence of structural failure is greater by virtue of complexity, size, occupancy, or risk.

Currently, a 7 story office building in San Francisco would require structural observation but a 60 story highrise or a 40000 seat stadium in New York would not. This proposal is intended to increase public safety by requiring that all similar structures are afforded the benefit of structural observation, not just the ones at risk of earthquakes or hurricanes.

Cost Impact: The code change proposal will not increase the cost of construction. It is generally held by many structural engineers that requirements stipulated by the building code will viewed as within the normal scope of services therefore it is not anticipated that there will be a general increase in engineering fees resulting from this proposal.

S132-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1704.5 #2-S-HARMAN.doc

S133–12 1704.5 (NEW), Chapter 35 (NEW)

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Add new text as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design* professional and prior to the construction or work being performed for each of the following:

- <u>1.</u> Welding procedure specifications in accordance with Section 6.1.2 of AWS D1.4 for the welding of concrete reinforcement other than by fillet welds.
- 2. <u>Test reports for Grade 55 anchor bolts verifying compliance with Supplementary Requirement S1</u> of ASTM F 1554 for weldability.
- 3. <u>Test reports for Grade A and B anchor bolts verifying compliance with Supplementary</u> <u>Requirement S1 of ASTM A 307 for weldability.</u>

Add new standard to Chapter 35 as follows:

ASTM

F1554-07a Standard Specification for Anchor Bolts, Steel, 36, 55 and 105-ksi Yield Strength

Reason: This proposal is a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official. This proposal adds three items to those in the separate proposal and the charging language in new Section 1704.5 is identical in both proposals.

Item 1 is added to new Section 1704.5 because Section 6.1.2 of AWS D1.4 requires qualification testing for the welding procedure specifications (WPS) of all types of welded joints that include reinforcing bars except for those consisting of fillet welds, which are deemed to be prequalified and, thus, exempt from testing. Section 6.1.2.3 of the standard requires the WPS to be made available to those authorized to examine them. The requirement for availability means that welding procedure specifications are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the welded joints are adequately designed to meet applicable requirements. Note that the 1998 edition of AWS D1.4 is a referenced standard of the 2012 IBC (see Chapter 35) but the 2011 edition is the current edition.

Item 2 is added to new Section 1704.5 because Grade 55 anchor bolts complying with ASTM F 1554-07a are not suitable for welding but weldable steel is possible, provided the material for the bolts meets Supplementary Requirement S1 of the standard. In ASTM F 1554-07a, Section 4.2 classifies Grade 55 anchor bolts complying with Supplementary Requirement S1 as weldable, Section 5.1 requires orders for anchor bolts to include required test reports (Section 5.1.13), and Section 17.1 requires the purchaser to be furnished with a test report that includes the carbon equivalent in accordance with Supplementary Requirement S1 (Section 17.1.1). The requirement that the purchaser be furnished with the test reports means that they are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the anchor bolts meet the applicable requirements for weldability.

Grade 36 bolts complying with ASTM F 1554-07a are weldable because of the limits on carbon in Table 1 ("Chemical Requirements for Grade 36") of the standard, which are 0.26%-0.28% by heat analysis and 0.29%-0.31% by product analysis depending on the bolt diameter. Grade 55 anchor bolts not complying with Supplementary Requirement S1 are not weldable because of the lack of limits on carbon in Table 2 ("Chemical Requirements for Grades 55 and 105") of the standard. In Supplementary Requirement S1, Section S1.2 assumes that suitable welding procedures for the steel being welded and the intended service will be selected, Section S1.5.1 specifies limits on carbon of 0.30% by heat analysis and 0.33% by product analysis, Section S1.5.2 requires an analysis of the carbon equivalent (CE) verifying that limits on CE are met (0.45% for alloy and low-alloy steel and 0.40% for carbon steel), and Section S1.6 requires the anchor bolts to be designated by a white paint mark on the side of the bar to be encased in concrete.

Of the ASTM standards applicable to other commonly used anchor bolts, Table 2 ("Chemical Requirements") of ASTM A 36 for carbon steel shapes, plates and bars of structural quality limits carbon in bars to 0.26%-0.29% depending on nominal diameter; and Table 1 ("Chemical Requirements for Grades A and B Bolts and Studs") of ASTM A 307 for carbon steel bolts and studs limits carbon in Grade A and B bolts and studs to 0.29% by heat analysis and 0.33% by product analysis. ASTM A 307 Grade C bolts and studs are specified as having properties complying with ASTM A 36 (Section 1.1). The effect of these provisions is that anchor bolts with properties complying with ASTM A 307, Grade C) are weldable but anchor bolts complying with ASTM A 307, Grade A or B, may not be weldable and the standard specifies additional requirements (Section 1.5) to ensure weldability (Supplementary Requirement S1) that are similar to those in ASTM F 1554-07a. Item 3 is added to new Section 1704.5 because of this.

Note that separate proposals:

- 1. Transfer the requirements of Section 1705.12.1 to new Section 1704.5;
- Add additional requirements for submittals that are related to structural steel;
 Add additional requirements for submittals that are related to masonry; and
- 4. Add a new Section 107.1.1 that correlates with this proposal.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S133-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1704.5 (NEW) #1-S-BRAZIL.doc

S134–12 1704.5 (NEW), Chapter 35 (NEW)

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, representing self (pbrazil@reidmiddleton.com)

Add new text as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design* professional and prior to the construction or work being performed for each of the following:

- <u>Test reports verifying compliance with Supplementary Requirement S30 of ASTM A6 for W-shaped and WT-shaped elements of structural steel with flange thicknesses of 1-1/2 inches (38 mm) or greater that are required to have a Charpy V-notch toughness as specified in Section A3.3 of AISC 341;</u>
- Test reports verifying compliance with Supplementary Requirement S5 of ASTM A6 for structural steel plates of 2 inches (51 mm) in thickness or greater that are required to have a Charpy Vnotch toughness as specified in Section A3.3 of AISC 341;
- <u>Certificates of compliance for verification that welds at elements of structural steel and their</u> connections that are in the seismic force-resisting system are made with filler metal having a Charpy V-notch toughness as specified in Section A3.3a of AISC 341;
- 4. <u>Certificates of compliance for verification that demand critical welds are made with filler metal</u> having a Charpy V-notch toughness as specified in Section A3.3b of AISC 341;
- Test reports verifying compliance with Supplementary Requirement S30 of ASTM A6 for hotrolled shapes of structural steel with flange thicknesses greater than 2 inches (51 mm) that are required to have a Charpy V-notch toughness as specified in Section A3.1c of AISC 360;
- 6. <u>Certificates of compliance for the fabrication of steel buckling-restrained braces on the premises</u> of an approved fabricator in accordance with Section 1704.2.5.2.

Add new standard to Chapter 35 as follows:

ASTM

<u>A 6-11</u> Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes and Sheet Piling

Reason: This proposal is a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official. This proposal adds six items to those in the separate proposal and the charging language in new Section 1704.5 is identical in both proposals. The parenthetic references to AISC 341-05 below are provided for reference and correspond to the referenced provisions of AISC 341-10. Similarly, there are parenthetic references to AISC 360-05 that correspond to the referenced provisions of AISC 360-10.

Items 1 and 2 are added to new Section 1704.5 because of the requirements in Section A3.3 of AISC 341-10 (Section 6.3 of AISC 341-05) for minimum Charpy V-notch (CVN) toughness in (1) hot rolled shapes of structural steel with flange thicknesses of 1-1/2 inches or greater, and (2) structural steel plates 2 inches in thickness or greater and meeting the condition specified therein, where they are elements of the seismic force-resisting system in structures within the scope of AISC 341-10 (or AISC 341-05) for verification by the building official (authority having jurisdiction) that the requirements are met.

The condition specified in Section A3.3 of AISC 341-10 for steel plates is that Charpy V-notch (CVN) toughness is limited for (1) members built up from plate, (2) connection plates where inelastic strain under seismic loading is expected, and (3) the steel core of buckling-restrained braces. Note that there is apparently an error in Section A3.3 of AISC 341-10 for hot-rolled shapes in that the minimum flange thickness is specified as 1/2 inch (38 mm) but, given the stated thickness in millimeters, 1-1/2 inches is intended.

Section A3.3 of AISC 341-10 (Section 6.3 of AISC 341-05) requires the structural steel to comply with Section A3.1c of AISC 360-10 (Section A3.1c of AISC 360-05). For hot rolled shapes of structural steel with flange thicknesses greater than 2 inches and meeting the conditions specified therein, Section A3.1c of AISC 360-10 requires the construction documents (structural design documents) to specify that such shapes shall be supplied with CVN impact test results in accordance with ASTM A6, Supplementary Requirement S30. Assuming that it is not the intent for the shapes to supply the test results, it is assumed that the intent is for tests in accordance with ASTM A6, Supplementary Requirement S30 to be conducted on the shapes. Section A3.3 of AISC 341-10 also requires the structural steel to be tested for CVN toughness as specified in ASTM A6, Supplementary Requirement S30, for hot-rolled shapes and in accordance with ASTM A 673 for steel plate. This has the effect of modifying the requirement in Section A3.1c of AISC 360-10 to lower the threshold for CVN impact testing of hot-rolled shapes of structural steel to those with flange thicknesses of 1-1/2 inches or greater and to also require CVN impact testing for structural steel plates that are 2 inches in thickness or greater. The requirement for test results means that test reports are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the structural steel meets the applicable requirements for CVN toughness.

In ASTM A 6-11, Section 1.8 indicates that the supplementary requirements therein are for use where additional testing or restrictions are required by the purchaser in the purchase order, Section 14.1 requires test reports for each heat supplied, and Section 14.1.6 requires the test reports to report the results of tests required by the purchase order. As for Section A3.1c of AISC 360-10 (discussed above), the requirement for test reports means that they are available for submittal to the building official, and requiring their submittal to the building official will enable the building official to verify whether the structural steel meets the applicable requirements for CVN toughness.

Supplementary Requirement S5 of ASTM A 6-11 requires CVN impact tests to be conducted in accordance with ASTM A 673 (Section S5.1). Supplementary Requirement S30 of ASTM A 6-11 requires CVN impact tests to be conducted in accordance with ASTM A 673 using specimens taken from the alternate core location (Section S30.1). This means that the supplementary requirements are identical in that both require impact testing in accordance with ASTM A 673 to determine CVN toughness except that Supplementary Requirement S30 imposes an additional condition on the testing, which is to take specimens from the alternate core location. Section A3.3 of AISC 341-10 references ASTM A 673 for steel plate but the proposal references Supplementary Requirement S5 of ASTM A 6-11 for consistency with the reference to Supplementary Requirement S30 of ASTM A 6-11 for hot-rolled shapes of structural steel.

Item 1 is limited in scope to W-shaped and WT-shaped structural members because the requirement in Section A3.3 of AISC 341-10 (Section 6.3 of AISC 341-05) for minimum CVN toughness is limited to hot-rolled shapes of structural steel with flange thicknesses of 1-1/2 inches or greater, which occur only in W-shaped and WT-shaped elements of structural steel. Section 3.1.2 of ASTM A 6-11 defines "shapes" as including "W" shapes, "HP" shapes, "S" shapes, "M" shapes, "C" shapes, "MC" shapes and "L" shapes. Of these shapes, the *AISC Steel Construction Manual* (thirteenth edition) only lists W-shaped and WT-shaped elements of structural steel with flange thicknesses of 1-1/2 inches or greater (Tables 1-1 and 1-8). Note that the *Manual* also does not list any "MT" shapes or "ST" shapes with flange thicknesses of 1-1/2 inches or greater.

The provisions in Section A3.3 of AISC 341-10 (Section 6.3 of AISC 341-05) and Section A3.1c of AISC 360-10 (Section A3.1c of AISC 360-05) are limited to hot-rolled shapes of structural steel but are not limited by type of shape. In Items 1 and 2 of this proposal, however, the requirement for submittal of test reports is limited by type of shape but is not limited to hot-rolled shapes of structural steel. The type of shape is limited to eliminate extraneous shapes for which the requirement for submittal does not apply. Limiting the requirement for submittal to shapes that are hot-rolled is not included because "hot-rolled" is a manufacturing process and is not relevant to the requirement for submittal. The "hot-rolled" limit is also not included for consistency with ASTM A 6-11 whose scope specifies the standard as applying to "rolled structural steel bars, plates, shapes and sheet piling" (Section 1.1).

Section A3.3 of AISC 341-10 and Section A3.1c of AISC 360-10 do specify hot-rolled shapes and the same is true of Section 6.3 of AISC 341-05 and Section A3.1c of AISC 360-05. None of these standards, however, define "hot-rolled" nor, to my knowledge, does any referenced standard of the 2012 IBC or any other standard referenced in the AISC standards listed above.

Items 3 and 4 are added to new Section 1704.5 because of the requirements in Sections A4.4a and A4.4b of AISC 341-10 (Sections 7.3a and 7.3b of AISC 341-05) for minimum CVN toughness of welds that are used in elements of structural steel and their connections that are in the seismic force-resisting system of structures within the scope of AISC 341. AISC 341-05 directly specifies the requirements. AISC 341-10 indirectly specifies them by referencing the requirements in Section (Clause) 6.3 of AWS D1.8. As for Items 1 and 2 of the proposal (discussed above), there are no provisions in AISC 341-10 (or AISC 341-05) for verification by the building official (authority having jurisdiction) that the requirements are met.

Section (Clause) 6.3 of AWS D1.8 (2009 edition) contains requirements for filler and weld metal of welds, including demand critical welds, that are within the scope of the standard. Among those requirements, Sections 6.3.1 and 6.3.5 specify mechanical properties for filler metals, including minimum CVN toughness, of welds and demand critical welds, respectively, which are listed in corresponding Tables 6.1 and 6.2. Note that AWS D1.8 is not a referenced standard of the 2012 IBC.

Section (Clause) 6.1.1 of AWS D1.8 requires welding procedure specifications to be prequalified, or to be qualified by testing in accordance with applicable AWS D1.1 requirements. Note that Section 1.1 of AWS D1.8 (1) establishes the applicability of AWS D1.8 as supplementing AWS D1.1 and (2) states that the provisions in AWS D1.1 apply to the welds governed by the provisions AWS D1.1 except where modified in AWS D1.8.

Section (Clause) 4.0 of AWS D1.1 (2008 edition) contains requirements for qualification testing of welding procedure specifications (WPS's). Section 3.1, however, exempts prequalified welding procedure specifications from requirements for qualification testing. A WPS is required to meet the provisions of Chapter 3 of AWS D1.1 in order to be prequalified. However, there are no provisions in Chapter 3 for minimum CVN toughness. Section 4.1.1.3 requires CVN tests to be included in the WPS qualification where required by the construction (contract) documents. Section 1.4.1(5) requires the Engineer to specify in the construction (contract) documents the CVN toughness criteria for weld metal (and base metal). Where notch toughness of welds used in elements of structural steel or their connections (welded joints) is required, Section 2.2.2 requires the Engineer to specify in the construction (contract) documents the minimum absorbed energy and corresponding test temperature for the filler metal (e.g., prequalified) or to specify that the WPS shall be qualified by CVN tests.

The effect of these provisions in AWS D1.1 is that the standard specifies CVN impact testing for qualification of welded joints to meet specified requirements for minimum CVN toughness. The standard does not prevent a prequalified WPS from being qualified to meet requirements for minimum CVN toughness but verification is only possible through review of the WPS. Section 3.1 of the standard requires all prequalified welding procedure specifications to be written. This requirement means that prequalified welding procedure specifications or equivalent documents for minimum CVN toughness, requiring the submittal of welding procedure specifications or equivalent documents (see below) to the building official will enable the building official to verify whether the welded joints meet the applicable requirements for CVN toughness.

Given the discussion above on the provisions in AWS D1.8 and D1.1, it would appear that the submittal of welding procedure specifications is needed to verify CVN toughness where required by Section A4.4a or A4.4b of AISC 341-10. AISC 341-10,

however, presents another approach. Section J2 contains requirements for documents to be submitted or made available to the engineer of record. Section J2.1 requires the submittal of welding procedure specifications (Item 1); certificates of conformance from the manufacturer for electrodes, fluxes and shielding gases (Item 2); and, for demand critical welds, applicable manufacturer's certifications that the filler metal meets supplemental notch toughness requirements (Item 3). Given these requirements and for consistency with Section 1704.2.5.2 and other sections of the 2012 IBC, the submittal of certificates of compliance instead of welding procedure specifications is specified in Items 3 and 4. Note that Section J2 does not specify that the documents required to be submitted or made available to the engineer of record are also required to be submitted or made available to the authority having jurisdiction (building official).

Item 5 is added to new Section 1704.5 because of the requirement in Section A3.1c of AISC 360-10 (Section A3.1c of AISC 360-05) for minimum Charpy V-notch (CVN) toughness of heavy structural steel shapes (e.g., with flange thicknesses greater than 2 inches) and meeting several conditions specified therein. Section A3.1c requires the construction documents (structural design documents) to specify that such shapes shall be supplied with CVN impact test results in accordance with ASTM A6, Supplementary Requirement S30. The requirement for test results means that test reports are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the structural steel meets applicable requirements for CVN toughness.

Item 6 is added to new Section 1704.5 to enable the building official to verify that fabrication of the steel buckling-restrained braces, where it is conducted at a location other than the construction site, was performed in accordance with the building code, its referenced standards (e.g., AISC 341) and the approved construction documents. Otherwise, special inspection at the fabricator's shop should be conducted (see IBC Section 1704.2.5).

Note that separate proposals:

- 1. Transfer the requirements of Section 1705.12.1 to new Section 1704.5;
- 2. Add additional requirements for submittals that are related to the welding of concrete reinforcement and anchor bolts;
- 3. Add additional requirements for submittals that are related to masonry; and
- 4. Add a new Section 107.1.1 that correlates with this proposal.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S134-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1704.5 (NEW) #2-S-BRAZIL.doc

S135–12 1704.5 (NEW)

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Add new text as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of special inspections and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a registered design professional and prior to the construction or work being performed for each of the following:

- 1. Reports of preconstruction tests for masonry where the prism test method of Section 2105.2.2 is used to determine the *compressive strength of masonry* in accordance with Section 1.19.3 of TMS 402/ACI 530/ASCE 5.
- 2. Reports of preconstruction tests of grout where the unit strength method of Section 2105.2.2 is used to determine the *compressive strength of masonry* in accordance with Section 1.19.3 of TMS 402/ACI 530/ASCE 5.

Reason: This proposal is a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official. This proposal adds two items to those in the separate proposal and the charging language in new Section 1704.5 is identical in both proposals.

The items are added to new Section 1704.5 because Section 1.19.3 of TMS 402/ACI 530/ASCE 5 requires compliance with a Level C quality assurance program for engineered masonry in structures classified as Risk Category IV. Table 1.19.3 for Level C quality assurance requires the verification of the specified compressive strength of masonry, f_m , prior to construction. Section 1.19.6.2 requires the compressive strength of masonry to be determined in accordance with TMS 602/ACI 530.1/ASCE 6. Article 1.4.B.1 of TMS 602/ACI 530.1/ASCE 6 requires the determination to be done by the unit strength method or the prism test method. Determination by the prism test method is, therefore, not required but when it is chosen for the verification of f_m prior to construction test. Item 1 is added because of this. When the unit strength method is chosen for the same purpose, the grout is required to be tested for compressive strength in accordance with ASTM C 1019 (Article 1.4.B.2b (3b), which also becomes a preconstruction test. Item 2 is added because of this. In each case, requiring the submittal of test reports to the building official will enable the building official to verify, before construction begins, the validity of structural design assumptions based on the success of the preconstruction tests.

Neither TMS 402/ACI 530/ASCE 5 nor TMS 602/ACI 530.1/ASCE 6 specifies submittals to applicable regulatory officials (e.g., building official or authority having jurisdiction). In TMS 402/ACI 530/ASCE 5, Section 1.19.4 requires the quality assurance program to set forth the procedures for reporting and review, and Item 1 in Tables 1.19.2 (Level B Quality Assurance) and 1.19.3 (Level C Quality Assurance) specifies verification of compliance with the approved submittals ("approved" is not defined in Section 1.6, Definitions). In TMS 602/ACI 530.1/ASCE 6, (1) Section 1.5.A specifies that written acceptance of submittals be obtained prior to use of the materials or methods requiring acceptance; (2) Section 1.5.B specifies the submittals; (3) Section 1.2 defines "acceptable/accepted" as being done by the architect/engineer and "architect/engineer" as the individual or firm that issues, or administers the work under, the drawings and specifications ("approved" is not defined); and (4) Sections 1.6.A and 1.6.B specify the services and duties of testing agencies and inspection agencies, respectively, including requirements for the owner to retain the agencies to report results and submit final reports to the architect/engineer and contractor.

Note that separate proposals:

- 1. Transfer the requirements of Section 1705.12.1 to new Section 1704.5 ;
- 2. Add additional requirements for submittals that are related to structural steel ;
- Add additional requirements for submittals that are related to the welding of concrete reinforcement and anchor bolts and
 Add a new Section 107.1.1 that correlates with this proposal.

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

S135-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
, ,				1704.5 (NEW) #3-S-BRAZIL.doc

S136–12 1704.5 (NEW), 1705.3.1, 1705.12.1

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design* professional and prior to the construction or work being performed for each of the following:

- Reports of material properties verifying compliance with the requirements of AWS D1.4 for weldability as specified in Section 3.5.2 of ACI 318 for reinforcing bars in concrete complying with a standard other than ASTM A 706 that are to be welded; and
- 2. Reports of mill tests in accordance with Section 21.1.5.2 of ACI 318 for reinforcing bars complying with ASTM A 615 and used to resist earthquake-induced flexural or axial forces in the special moment frames, special structural walls, or coupling beams connecting special structural walls, of seismic force-resisting systems in structures assigned to Seismic Design Category B, C, D, E or F.

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.

1705.12.1 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, 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.

Reason: This proposal is a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official. This proposal adds two items to those in the separate proposal and the charging language in new Section 1704.5 is identical in both proposals.

The requirement in Section 1705.12.1 to provide certified mill test reports for reinforcement in special moment frames, special structural walls and coupling beams is relocated to Item 2 of new Section 1704.5 because the subject of Section 1705.12 is testing and qualification for seismic resistance but there is no testing specified in Section 1705.12.1. The submittal of certified mill test reports is specified but there is no corresponding requirement in ACI 318-11 that the reports be certified or that the act of submittal amounts to a "qualification." Also ACI 318 has consistently specified "mill tests" since the alternative to reinforcement complying with ASTM A 706 first appeared in the 1983 edition. The limitation in Section 1705.12.1 to reinforcement complying with ASTM A 615 is retained in Item 2 for consistency with the same limitation in the referenced section of ACI 318-11 (Section 21.1.5.2).

Relocating the requirement in Section 1705.12.1 to Item 2 of new Section 1704.5 has an additional benefit that is provided by the charging language in the new section. Section 1705.12.1 requires mill test reports to be provided with each shipment of reinforcement but that does not ensure the reports will be available to the owner, design team, construction team or building official. New Section 1704.5, however, requires the owner or authorized agent to submit the reports to the building official after review and acceptance by a registered design professional and prior to the construction or work begin performed. Also, the current requirement in Section 1705.12.1 that the reports be provided for each shipment means that they are available for submittal to the building official.

The charging language in Section 21.1.5.2 of ACI 318-11 specifies deformed reinforcement but Item 2 specifies reinforcing bars for consistency with (1) the basic requirement in Section 21.1.5.2 for compliance with ASTM A 706, which is limited in scope to "deformed and plain low-alloy steel bars...for concrete reinforcement" (Section 1.1), and (2) the alternative of compliance with ASTM A 615, which is limited in scope to "deformed and plain carbon steel bars for concrete reinforcement," provided the special requirements of Section 21.1.5.2 are also met.

The source document for some of the language in Section 1705.12.1 is the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (Section 3.4.1.2 of FEMA 368 and Section 2.4.1.2 of FEMA 450-1).

In Item 1 of new Section 1704.5, the requirement in the last sentence of Section 1705.1.2.1 for chemical tests of reinforcement complying with ASTM A 615 that is to be welded is replaced with a requirement to submit reports of material properties for reinforcing bars complying with a standard other than ASTM A 706 that verify compliance with the requirements of AWS D1.4 for weldability. These changes correct several errors. First, the current language in Section 1705.1.2.1 is limited in scope to Seismic Design Categories B through F by that section, and to Seismic Design Categories C through F by the charging language in Section 1705.12 (Item 1), but verification of weldability is not a seismic issue. Verifying weldability is important for concrete reinforcement designed to resist all load effects, not merely seismic load effects.

Second, the current language in Section 1705.1.2.1 requires chemical tests of reinforcement be performed to determine weldability in accordance with Section 3.5.2 of ACI 318 but Section 3.5.2 of ACI 318 does not require chemical tests to be performed. Instead, it requires the ASTM specification to be supplemented by specifying a "report of material properties."

Third, Section 1705.12.1 requires the chemical tests for reinforcement complying with ASTM A 615 but Section 3.5.2 of ACI 318 specifies the report of material properties for reinforcement complying with a standard other than ASTM A 706. In ACI 318-11, specified standards other than ASTM A 615 and A 706 include A 955, A 996 and A 1035 (see Section 3.5.3.1).

Fourth, Section 1705.12.1 specifies concrete reinforcement but Section 3.5.2 of ACI 318 specifies reinforcing bars, which is done to exclude other types of concrete reinforcement such as plain reinforcement, headed shear studs, structural steel, steel pipe and steel tubing. Refer to Section 3.5, and the definition of "reinforcement" in Section 2.2, in ACI 318-11 for further information.

The language in Item 1 of new Section 1704.5 is consistent with the provisions in Section 3.5.2 of ACI 318 as discussed above. Section 3.5.2 of ACI 318 has consistently specified (1) a report of material properties, (2) a standard other than ASTM A 706 and (3) reinforcing bars, ever since the section first appeared in the 1977 edition. Section 3.5.2 also requires the applicable ASTM specifications for reinforcing bars to be "supplemented to require a report of material properties necessary to conform to the requirements in AWS D1.4." The requirement means that reports of material properties are available for submittal to the building official will enable the building official to verify whether the reinforcing bars meet the applicable requirements for weldability.

For Items 1 and 2, neither ACI 318-11 nor ACI 301 ("Specifications for Structural Concrete," not an IBC referenced standard) specifies submittals to applicable regulatory officials (e.g., building official or authority having jurisdiction). In ACI 318, (1) Section 1.2.2 specifies the filing of calculations pertinent to the design with the contract documents when required by the building official, (2) Section 1.3.1 specifies inspection as required by the legally adopted general building code, and (3) Sections 1.3.2 through 1.3.4 specify requirements for the keeping and retention of inspection records, but (4) reports of mill tests and material properties are not included. In ACI 301-05, (1) Section 1.5.1 specifies that submittals required by the standard be submitted for review and acceptance; (2) Section 1.2 defines "submitted" as being provided to the architect/engineer for review or acceptance and "architect/engineer" as the individual or firm that issues the project drawings and specifications or administers the work under the contract documents ("approved" is not defined); (3) Section 1.5.2 specifies requirements for testing agency of test results to the owner, architect/engineer and contractor; and (4) Section 1.6.2 specifies requirements for testing agencies, including acceptance by the architect/engineer before performing any work.

Note that Section 1.3.4 of AWS D1.4-98 requires the calculation of carbon equivalent for all reinforcing bars, including those complying with ASTM A 706. If mill test reports are not available to enable the calculation, chemical analysis is permitted to be performed. If the chemical composition is not known, special preheat temperatures are required (see Section 1.3.4.3).

Also, the likely source document for the current requirement to perform chemical tests, the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (Section 3.4.1.3 of FEMA 368 and Section 2.4.1.3 of FEMA 450-1) did not require chemical tests to be performed. It required verification "that chemical tests have been performed to determine weldability in accordance with Section 3.5.2 of ACI 318."

Note that separate proposals:

- 1. Add additional requirements for submittals that are related to structural steel (Sxx-12/13);
- Add additional requirements for submittals that are related to the welding of concrete reinforcement and anchor bolts (Sxx-12/13);
- 3. Add additional requirements for submittals that are related to masonry (Sxx-12/13); and
- 4. Add a new Section 107.1.1 that correlates with this proposal (Sxx-12/13).

S136-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1704.5 (NEW) #4-S-BRAZIL.doc

S137–12 1704.5.1, 1705.11, 1705.11.7, 1905.1.8, 2209.1

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

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 where one or more of the following conditions exist:

- 1. The structure is classified as Risk 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 <u>as defined in</u> <u>Section 11.2 of ASCE 7</u>.
- 3. The structure is assigned to Seismic Design Category E, is classified as Risk 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.

1705.11 Special inspections for seismic resistance. *Special inspections* itemized in Sections 1705.11.1 through 1705.11.8, unless exempted by the exceptions of Section 1704.2, are required for the following:

- 1. The seismic force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Sections 1705.11.1 through 1705.11.3, as applicable.
- 2. Designated seismic systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Section 1705.11.4.
- 3. Architectural, mechanical and electrical components in accordance with Sections 1705.11.5 and 1705.11.6.
- 4. Storage racks <u>as defined in Section 11.2 of ASCE 7 that are</u> in structures assigned to *Seismic Design Category* D, E or F in accordance with Section 1705.11.7.
- 5. Seismic isolation systems in accordance with Section 1705.11.8.

Exception: Special inspections itemized in Sections 1705.11.1 through 1705.11.8 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, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 35 feet (10 668 mm).
- The seismic force-resisting system of the structure consists of reinforced masonry or reinforced concrete; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 25 feet (7620 mm).
- 3. The structure is a detached one- or two-family dwelling not exceeding two *stories above grade plane* and does not have any of the following horizontal or vertical irregularities in accordance with Section 12.3 of ASCE 7:
 - 3.1. Torsional or extreme torsional irregularity.
 - 3.2. Nonparallel systems irregularity.
 - 3.3. Stiffness-soft story or stiffness-extreme soft story irregularity.
 - 3.4. Discontinuity in lateral strength-weak story irregularity.

1705.11.7 Storage racks. Periodic *special inspection* is required during the anchorage of storage racks <u>as defined in Section 11.2 of ASCE 7 that are</u> 8 feet (2438 mm) or greater in height in structures assigned to *Seismic Design Category* D, E or F.

Revise as follows:

1905.1.8 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 <u>as defined in</u> <u>Section 11.2 of ASCE 7</u> 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 Seismic Design Categories A, B and C, detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls, are permitted to have plain concrete footings without longitudinal reinforcement.
- 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.

Revise as follows:

2209.1 Storage racks. The design, testing and utilization of industrial steel storage racks as defined in Section 11.2 of ASCE 7 and 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 provisions of Section 15.5.3 of ASCE 7, except that the mapped acceleration parameters, S_s and S_1 , shall be determined in accordance with Section 1613.3.1.

Reason: The purpose for the proposal is to clarify the meaning of "base" and "storage rack," which are defined in ASCE 7-10 but are not also defined in the building code. Both of these terms have meanings that necessitate knowing their definitions to fully understand the technical provisions related to them. Therefore, the proposal adds references to Section 11.2 of ASCE 7-10 for their

definitions. The only instances of these terms in the 2012 IBC where they are directly related to their corresponding definitions in ASCE 7-10 are in this proposal.

For storage racks, adding a reference to the definition in ASCE 7-10 in Section 1705.11.7 also has the effect of narrowing the scope to those that are defined. Note that "storage rack" is defined in ASCE 7-10 as including "industrial pallet racks, moveable shelf racks and stacker racks made of cold-formed or hot-rolled structural members;" but excluding "other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks or racks made of materials other than steel."

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

S137-12

Public Hearing: Comm	nittee:	AS	AM	D	
Assen	nbly:	ASF	AMF	DF	
	•				1704.5.1-S-BRAZIL.doc

S138–12 1704.5, 1705.4, 1705.4.1

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.5 Structural observations. Where required by the provisions of Section 1704.5.1 or 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.

1705.4 Masonry construction. Masonry construction shall be inspected and verified in accordance with TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6 quality assurance program requirements.

Exception: Special inspections shall not be required for:

- Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, where they are part of structures classified as *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.

1705.4.1 Empirically designed masonry, glass unit masonry and masonry veneer in 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, in structures classified as *Risk Category* IV, in accordance with Section 1604.5, shall comply with TMS 402/ACI 530/ASCE 5 Level B Quality Assurance.

Reason: The purpose for the proposal is to delete language considered superfluous given the definitions in Section 202 for "structural observation" and "risk category." These are the only instances of such language in the structural chapters of the 2012 IBC.

S138-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1704 5-S-BRAZIL doc

S139–12 202, 1705.1, 1705.10.1, 1705.10.2, 1705.11, 1705.11.2, 1705.11.3, 1705.11.6

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Delete without substitution:

SECTION 202 DEFINITIONS

MECHANICAL SYSTEMS. For the purposes of determining seismic *loads* in ASCE 7, mechanical systems shall include plumbing systems as specified therein.

Revise as follows:

1705.1 General. Verification and inspection of elements <u>and nonstructural components</u> of buildings and structures shall be as required by this section.

1705.10.1 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main windforce-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within <u>elements of</u> the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: Special inspections is <u>are</u> not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components <u>elements</u> of the main windforce-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

1705.10.2 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within elements of the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: Special inspections is <u>are</u> 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.).

1705.11 Special inspections for seismic resistance. *Special inspections* itemized in Sections 1705.11.1 through 1705.11.8, unless exempted by the exceptions of Section 1704.2, are required for the following:

- 1. The seismic force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Sections 1705.11.1 through 1705.11.3, as applicable.
- 2. Designated seismic systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Section 1705.11.4.
- 3. Architectural, mechanical and electrical <u>Nonstructural</u> components in accordance with Sections 1705.11.5 and 1705.11.6.
- Storage racks in structures assigned to Seismic Design Category D, E or F in accordance with Section 1705.11.7.

5. Seismic isolation systems in accordance with Section 1705.11.8.

Exception: Special inspections itemized in Sections 1705.11.1 through 1705.11.8 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, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 35 feet (10 668 mm).
- 2. The seismic force-resisting system of the structure consists of reinforced masonry or reinforced concrete; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 25 feet (7620 mm).
- 3. The structure is a detached one- or two-family dwelling not exceeding two *stories above grade plane* and does not have any of the following horizontal or vertical irregularities in accordance with Section 12.3 of ASCE 7:
 - 3.1. Torsional or extreme torsional irregularity.
 - 3.2. Nonparallel systems irregularity.
 - 3.3. Stiffness-soft story or stiffness-extreme soft story irregularity.
 - 3.4. Discontinuity in lateral strength-weak story irregularity.

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 <u>elements of</u> the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

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

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 elements of the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: Special inspections is <u>are</u> not required for coldformed 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.

1705.11.6 <u>Plumbing</u>, mechanical and electrical components. *Special inspection* for <u>plumbing</u>, mechanical and electrical components shall be as follows:

- 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 anchorage of other electrical equipment in structures assigned to *Seismic Design Category* E or F;
- Periodic special inspection is required during the installation and anchorage of piping systems designed to carry 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 ductwork 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 the proposal is to correlate the provisions of the building code related to nonstructural components with the corresponding provisions for nonstructural components in ASCE 7-10. Essentially, the seismic chapters of ASCE 7-10 apply to the seismic force-resisting system except for Chapter 13, which applies to nonstructural components. The language in these chapters consistency refers to the seismic force-resisting system in terms of structural members or elements, and to other materials or products that are required to be designed for resistance to seismic load effects as "nonstructural components." Chapter 13 consistently uses the term "nonstructural component" until later in the chapter where there are individual requirements for groups of nonstructural components. Materials and products subject to the requirements of Chapter 13 are grouped according to whether they are architectural, mechanical or electrical components, and "nonstructural" is dropped because it is, by then, considered redundant. The proposal revises the corresponding provisions in the building code for consistency with this phraseology.

The definition of "mechanical system" is deleted because it isn't a definition but a requirement, which is incorporated into the building code by adding "plumbing" to Section 1705.11.6. Also, the requirement in the definition that mechanical systems include plumbing systems for "the purposes of determining seismic loads in ASCE 7" serves no purpose in the building code. Section 1613.1 references ASCE 7 for the design and construction of structures to resist the effects of earthquake motions. Chapter 13 of ASCE 7-10 clearly indicates that plumbing systems are included in the provisions for mechanical systems.

In Item 3 of Section 1705.11, "architectural, mechanical and electrical" is replaced with "nonstructural" for consistency with Chapter 13 of ASCE 7-10 and because distinguishing among the groups of nonstructural components in Section 1705.11 serves no purpose but it does serve a purpose in Sections 1705.11.5 and 1705.11.6 where the requirements for architectural components differ from those for plumbing, mechanical and electrical components.

S139-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1705.1-S-BRAZIL.doc

S140–12 1705.2, Table 1705.2.2, 1705.2.2.1.1, 1705.2.2.2, 1705.11.1, 1705.11.1.1 (NEW), 1705.11.1.2 (NEW), 1705.12.2, 1705.12.2.1 (NEW), 1705.12.2.2 (NEW)

Proponent: Bonnie Manley, P.E. American Iron and Steel Institute, representing American Institute of Steel Construction (bmanley@steel.org)

Revise as follows:

1705.2 Steel construction. The *special inspections* for <u>and nondestructive testing of</u> steel elements of <u>construction in</u> buildings, and structures, <u>and portions thereof</u> shall be as required in <u>accordance with</u> this section.

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

1705.2.1 Structural steel. Special inspections and nondestructive testing for of structural steel <u>structural</u> <u>steel elements</u> in buildings, structures, and portions thereof shall be in accordance with the quality assurance inspection requirements of AISC 360.

Exception: Special inspection of railing systems composed of *structural steel elements* shall be limited to welding inspection of welds at the base of cantilevered rail posts.

1705.2.2 <u>Cold-formed</u> steel construction other than structural steel <u>deck and reinforcing</u>. Special inspections for steel construction other than structural steel <u>of cold-formed steel deck and reinforcing</u> <u>steel in buildings, structures, and portions theeof</u> shall be in accordance with Table 1705.2.2 and this section.

1705.2.2.1.1 Cold-formed steel <u>deck</u>. Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks shall be in accordance with AWS D1.3.

1705.2.2.2 1705.2.3 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 1705.2.2 REQUIRED VERIFICATION AND <u>SPECIAL</u> INSPECTION<u>S</u> OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL <u>COLD-FORMED STEEL DECK AND REINFORCING STEEL</u>

VERIFICATION AND INSPECTION TYPE	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
1. Material verification of cold-formed steel deck:			
 a. Identification markings to conform to ASTM standards specified in the approved construction documents. 	—	X	Applicable ASTM material standards
b. Manufacturers' certified test reports.	—	Х	
 Special inspection of welding: a. Cold-formed steel deck: 			

VERIFICATION AND INSPECTION TYPE	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
1) Floor and roof deck welds.	_	Х	AWS D1.3
b. Reinforcing steel:			
1) Verification of weldability of reinforcing steel other than ASTM A 706.	-	X	AWS D1.4 or ACI 318: Section 3.5.2
2) Reinforcing steel-resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special reinforced concrete shear walls and shear reinforcement.	X	_	
3) Shear reinforcement.	Х	—	1
4) Other reinforcing steel.	—	Х	1

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspections for seismic resistance.

1705.11.1 Structural steel. Special inspections for seismic resistance shall be in accordance with Sections 1705.11.1.1 or 1705.11.1.2, as applicable.

1705.11.1.1 Special inspections for structural steel of structural steel seismic-force resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Special inspections of structural steel are not required in the seismic-force resisting systems of buildings and structures assigned to *Seismic Design Category* <u>B or</u> C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.

1705.11.1.2 Special inspections of structural steel elements in seismic-force resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F other than those covered in Section 1705.11.1.1, including struts, collectors, chords and foundation elements, shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Special inspections of structural steel elements are not required in the seismic-force resisting systems of buildings and structures assigned to Seismic Design Category B or C with a response modification coefficient, *R*, less than 3.

1705.12.2 Structural steel. <u>Nondestructive testing for seismic resistance shall be in accordance with</u> <u>Sections 1705.12.2.1 or 1705.12.2.2, as applicable.</u>

<u>**1705.12.2.1**</u> Nondestructive testing for <u>of</u> structural steel <u>seismic-force resisting systems in buildings and</u> <u>structures assigned to *Seismic Design Category* B, C, D, E or F shall be <u>performed</u> in accordance with the quality assurance requirements of AISC 341.</u>

Exception: <u>Nondestructive</u> testing for structural steel is not required in the seismic-force resisting systems of buildings and structures assigned to *Seismic Design Category* <u>B or</u> C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.

1705.12.2.2 Nondestructive testing of *structural steel elements* in seismic-force resisting systems of buildings and structures assigned to *Seismic Design Category* B, C, D, E or F other than those covered in Section 1705.12.2.1, including struts, collectors, chords and foundation elements, shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Nondestructive testing of *structural steel elements* is not required in the seismic-force resisting systems of buildings and structures assigned to *Seismic Design Category* B or C with a response modification coefficient, *R*, less than 3.

Reason: This comprehensive proposal not only makes a number of editorial modifications for clarification purposes, it also introduces into Chapter 17 the term and associated requirements for "structural steel elements", which is handled in a companion proposal for Chapter 22. In that companion proposal, the definition of "structural steel member" is recommended for replacement by "structural steel element", which is defined as follows:

STEEL ELEMENT, STRUCTURAL. Any steel structural member of a building or structure consisting of rolled shapes, pipe, hollow structural sections, plates, bars, sheets, rods, or steel castings other than cold-formed steel or steel joist members. The Chapter 22 companion proposal includes a comprehensive discussion in the reason statement – please refer to it for additional background. Building on that proposal's reason statement, this proposal coordinates the existing special inspection and nondestructive testing requirements with the new terminology for structural steel elements. In Section 1705.2.1, changes clarify that structural steel elements in buildings, structures and portions thereof are to be inspected and tested in accordance with the quality assurance requirements in AISC 360. Current code requirements limit the special inspections to "structural steel." The change to "structural steel elements" was made to explicitly include steel construction that is typically designed, fabricated, and constructed in accordance with AISC 360, but that does fall within the definition of structural steel in AISC 360 and the AISC Code of Standard Practice for Buildings and Bridges. An exception is provided for railing systems to reflect what is currently done for these systems and prevent the implementation of excessive requirements.

In Section 1705.11.1 on special inspections for seismic resistance the distinction is drawn between structural steel seismicforce resisting systems, which include the sixteen structural steel systems currently listed in ASCE 7-10, Table 12.2-1, and structural steel elements that work as struts, collectors, chords and foundation elements in seismic-force resisting systems composed of other structural materials. These structural steel elements should be inspected in accordance with the quality assurance requirements of AISC 341, if they are used in a seismic-force resisting system that relies heavily on non-elastic energy dissipation, in this case chosen as a system with a response modification coefficient, R, greater than 3. A parallel change is made in Section 1705.12.2 on nondestructive testing for seismic resistance.

- Finally, the proposal includes a number of editorial modifications, including the following:
- It adds reference to "nondestructive testing" to clarify that the quality assurance provisions of AISC 360 and AISC 341covers not only special inspections but also testing of welds. The use of "nondestructive" is the appropriate industry terminology.
- It modifies "steel elements" to "steel construction" in order to match the terminology used in Chapter 22.
- It recognizes that special inspections and testing may be required in buildings, structures or portions thereof.
- It changes the title in Section 1705.2.2 to specifically recognize the types of steel construction covered cold-formed steel deck and reinforcing steel and to get away from the use of "structural steel". Since the section is limited to cold-formed steel deck, Section 1705.2.2.2 on cold-formed steel trusses is shifted to a new sub-section, 1705.2.3.
- It clarifies that the requirements in Sections 1705.11.1 and 1705.12.2 apply to the seismic-force resisting systems of buildings and other structures.

Finally, it clarifies the appropriate SDCs for the requirements and exceptions in both Sections 1705.11.1 and 1705.12.2.

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

S140-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1705.2#1-S-MANLEY.doc

S141–12 1705.2, 1705.2.1, 1705.2.2, Table 1705.2.2, 1705.2.2.1.1, 1705.2.2.2, 1705.11.1, 1705.12.2

Proponent: Bonnie Manley, P.E. American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

1705.2 Steel construction. The Special inspections for and nondestructive tests of steel elements of construction in buildings, and structures, and portions thereof shall be as required in accordance with this section.

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

1705.2.1 Structural steel. Special inspections for <u>and nondestructive testing of</u> structural steel <u>in</u> <u>buildings, structures, and portions thereof</u> shall be in accordance with the quality assurance inspection requirements of AISC 360.

1705.2.2 <u>Cold-formed</u> steel construction other than structural steel <u>deck and reinforcing steel</u>. Special inspections for steel construction other than structural steel <u>of cold-formed steel deck and</u> <u>reinforcing steel in buildings, structures, and portions thereof</u> shall be in accordance with Table 1705.2.2 and this section.

1705.2.2.1.1 Cold-formed steel <u>deck</u>. Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks shall be in accordance with AWS D1.3.

1705.2.22 1705.2.3 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 1705.2.2 REQUIRED VERIFICATION AND SPECIAL INSPECTIONS OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL COLD-FORMED STEEL DECK AND REINFORCING STEEL

VERIFICATION AND INSPECTION TYPE	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
1. Material verification of cold-formed steel deck:			
 a. Identification markings to conform to ASTM standards specified in the approved construction documents. 	_	х	Applicable ASTM material standards
b. Manufacturers' certified test reports.	_	Х	
 Special ilnspection of welding: a. Cold-formed steel deck: 			
1) Floor and roof deck welds.		Х	AWS D1.3
b. Reinforcing steel:			
VERIFICATION AND INSPECTION TYPE	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
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 Verification of weldability of reinforcing steel other than ASTM A 706. 	_	х	
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.	х	_	AWS D1.4 or ACI 318: Section 3.5.2
3) Shear reinforcement.	Х		
4) Other reinforcing steel.	—	Х	

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspections for seismic resistance.

1705.11.1 Structural steel. Special inspections for of structural steel in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Special inspections of structural steel are not required in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category <u>B or</u> C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.

1705.12.2 Structural steel. <u>Nondestructive</u> testing for <u>of</u> structural steel <u>in the seismic force-resisting</u> <u>systems of buildings and structures assigned to *Seismic Design Category* B, C, D, E or F shall be <u>performed</u> in accordance with the quality assurance requirements of AISC 341.</u>

Exception: <u>Nondestructive</u> testing for <u>of</u> structural steel <u>is not required</u> in <u>the seismic-force resisting</u> <u>systems of buildings and</u> structures assigned to *Seismic Design Category* <u>B or</u> 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 proposal is primarily editorial in nature and makes the following modifications:

- It adds reference to "nondestructive testing" to clarify that Chapter 17 covers not only special inspections but also testing. The use of "nondestructive" is the appropriate industry terminology.
- It modifies "steel elements" to "steel construction" in order to match the terminology used in Chapter 22.
- It adds recognition that special inspections and testing may be required in buildings, structures or portions thereof.
- It changes the title in Section 1705.2.2 to specifically recognize the types of steel construction covered cold-formed steel deck and reinforcing steel. Since the section is limited to cold-formed steel deck, Section 1705.2.2.2 on cold-formed steel trusses is shifted to a new sub-section, 1705.2.3.
- It adds reference to "special" inspections in Table 1705.2.2 and coordinates the title with the changes in the charging text.
- It clarifies that the requirements in Sections 1705.11.1 and 1705.12.2 apply to the seismic-force resisting systems of buildings and other structures.

Finally, it clarifies the appropriate SDCs for the requirements and exceptions in both Sections 1705.11.1 and 1705.12.2.

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

S141-12

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Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.2#2-S-MANLEY.doc

S142–12 1705.2.2 (NEW), Table 1705.2.2, Chapter 35 (NEW)

Proponent: Thomas Sputo, Ph.D., P.E., S.E., Steel Deck Institute

Revise as follows:

1705.2.2 Cold-formed steel deck. Special inspections and qualification of welding special inspectors for cold-formed steel floor and roof deck shall be in accordance with the quality assurance inspection requirements of SDI QA/QC,

1705.2.2 <u>**1705.2.3**</u> **Steel construction other than structural steel** <u>**Reinforcing steel**</u>. <u>Reinforcing steel</u> special inspections for steel construction other than structural steel</u> shall be in accordance with Table <u>1705.2.2</u> <u>1705.2.3</u> and this section.

TABLE 1705.2.2 1705.2.2.3 REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL REINFORCING STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a				
-1. Material verification of cold-formed steel deck:							
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	_	×	Applicable ASTM material standards				
<u>b. Manufacturer's</u> certified test reports.	_	×	-				
<u>1.</u> 2. Inspection of w	elding:						
- a. Cold-formed stee	el deck:						
1) Floor and roof deck welds.		×	AWS D1.3				
<u>a.</u> 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				

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
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		
3) Shear reinforcement.	х	_	
4) Other reinforcing steel.	_	x	

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspections for seismic resistance.

1705.2.2.1 <u>1705.2.3.1</u> Welding. Welding inspection and welding inspector qualification <u>for reinforcing</u> <u>steel</u> shall be in accordance with <u>AWS D1.4 AND ACI 318</u>.

1705.2.2.1.1 Cold-formed steel. Welding inspection and welding inspector qualification for coldformed steel floor and roof decks shall be in accordance with AWS D1.3.

1705.2.2.1.2 Reinforcing steel. Welding inspection and welding inspector qualification for reinforcing steel shall be in accordance with AWS D1.4 and ACI 318.

Add new standard to Chapter 35 as follows:

Steel Deck Institute

SDI QA/QC-2011, Standard for Quality Control and Quality Assurance for Installation of Steel Deck.

Reason: The SDI QA/QC-2011 Standard contains provisions for quality assurance inspection of steel floor and roof deck, and is intended to coordinate with the requirements of AISC 360, as contained in Section 1705.2.1.

The Standard complies with the Special Inspection requirements of the 2012 IBC Chapter 17, and clarifies the scope of required inspections and responsibilities of both the installer's quality control personnel and the quality assurance inspector. The Standard contains tables of inspection tasks that specifically list inspection requirements for material verification, deck installation, welding, and mechanical fastening. These tables amplify and clarify the basic special inspection requirements for steel deck that were contained in the 2012 IBC, and bring all special inspection requirements for steel deck into one place.

This Standard contains the 2012 IBC requirements of using AWS D1.3 for weld quality and requiring material verification. This Standard was developed and approved through a consensus process under ANSI guidelines, and complies with ICC CP 28. This Standard, along with all other Steel Deck Institute (SDI) Standards, will be available for free download from the SDI website for all parties.

For review purposes, the SDI QA/QC-2011 Standard that is being proposed is available for download and review from this website: http://www.sputoandlammert.com/standard.html

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S142-11

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.2.2 (NEW)-S-SPUTO

S143–12 1705.2.2.2, 1705.5.2, 2211.3.3, 2203.4.1.3

Proponent: Mark Gilligan, P.E., S.E., representing self (mark@gilligan.name)

Revise as follows:

1705.2.2.2 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.

1705.5.2 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.

Delete without substitution:

2211.3.3 Trussses spanning 60 feet or greater. The owner shall contract with a *registered design professional* for the design of the temporary installation restraint/bracing and the permanent individual truss member restraint/ bracing for trusses with clear spans 60 feet (18 288 mm) or greater. Special inspection of trusses over 60 feet (18 288 mm) in length shall conform to Section 1705.

2303.4.1.3 Trusses spanning 60 feet or greater. The owner shall contract with any qualified *registered design professional* for the design of the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing for all trusses with clear spans 60 feet (18 288 mm) or greater.

Reason: The provisions for temporary bracing need to be deleted since building departments do not have authority to enforce safety provisions during construction. The existing provisions deal with contractor's means and methods of construction. The ability to regulate in this area is pre-empted by Federal of State OSHA regulations thus local agencies do not have legal authority to regulate in this area and thus the model code should not contain these requirements.

There is no disagreement about the need for temporary bracing only with it being addressed in the building code. In addition to the legal argument, it is suggested that temporary bracing is an integral part of the installation procedures, thus separating the responsibility for design of temporary bracing from the responsibility for installation procedures will have a negative impact on construction safety.

Deleting these provisions does not alter the code requirements for permanent bracing nor the need to inspect the permanent bracing.

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

S143-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				1705.2.2.2-S-GILLIGAN.doc

S144-12 1705.2.2, Table 1705.2.2, 1705.2.2.1, 1705.2.2.1.2, Table 1705.3

Proponent: Philip Brazil, P.E., S.E., Senior Structural Engineer, Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1705.2.2 Steel construction other than structural Cold-formed steel deck. Special inspections for steel construction other than structural of cold-formed steel deck shall be in accordance with Table 1705.2.2 and this section.

TABLE 1705.2.2 REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL COLD-FORMED STEEL DECK

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
1. Material verification of cold-formed steel deck:			
a. Identification markings to conform to ASTM			Applicable ASTM
standards specified in the approved	_	Х	material standards
construction documents.			
b. Manufacturers' certified test reports.	_	Х	
2. Inspection of welding:	—	—	
a. Cold-formed steel deck			
4 <u>a</u> . Floor and roof deck welds		Х	AWS D1.3
b. Reinforcing steel:			
1. Verification of weldability of reinforcing steel			
other than ASTM A 706.	_	×	
2. Reinforcing steel resisting flexural and axial	X	-	
forces in intermediate and special moment			AWS D1.4
frames, and boundary elements of special			ACI 318 Section
structural walls of concrete and shear			3.5.2
reinforcement.			
3. Shear reinforcement.	×	_	
4. Other reinforcing steel.	_	X	

For SI 1 inch = 25.4 mm

a. Where applicable, see also Section 1705.11, Special inspection for seismic resistance.

1705.2.2.1 Welding. Welding inspection and welding inspector qualification shall be in accordance with this section. 1705.2.2.1.1 Cold-formed steel. Welding inspection and welding inspector gualification for cold-formed steel floor and roof decks shall be in accordance with AWS D1.3.

REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION IBC REFERENCE **D STANDARD** REFERENC **VERIFICATION AND INSPECTION** CONTINUOUS PERIODIC Ε 1. Inspection of reinforcing steel, including ACI 318: 3.5, Х 1910.4 prestressing tendons, and placement. 7.1-7.7 2. Inspection of reinforcing steel welding in AWS D1.4 accordance with Table 1705.2.2. Item ACI 318: 3.5.2 2b.

TABLE 1705.3

	CONTINUOUS		REFERENCE D STANDARD	IBC REFERENC
VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	-	E
<u>2. Inspection of reinforcing bar weiding:</u> <u>a. Verification of weldability of reinforcing</u> <u>bars other than ASTM A 706.</u>	=	X	<u>AWS D1.4</u> ACI 318: 3.5.2	
<u>axial forces in intermediate and</u> <u>special moment frames, and boundary</u> <u>elements of special structural walls of</u> <u>concrete and shear reinforcement.</u>	X	=		
c. Shear reinforcement.	X	=		
d. Other reinforcing bars.	=	<u>X</u>		
 Inspection of anchors cast in concrete where allowable loads have been increased or where strength design is used. 		х	ACI 318: 8.1.3, 21.2.8	1908.5, 1909.1
4. Inspection of anchors post-installed in hardened concrete members. ^b		Х	ACI 318: 3.8.6, 8.1.3, 21.2.8	1912.1
5. Verifying use of required design mix.	_	х	ACI 318: Ch. 4, 5.2–5.4	1904.2.2, 1910.2, 1910.3
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.	х	_	ASTM C 172 ASTM C 31 ACI 318: 5.6, 5.8	1910.10
 Inspection of concrete and shotcrete placement for proper application techniques. 	х	_	ACI 318: 5.9, 5.10	1910.6, 1910.7, 1910.8
8. Inspection for maintenance of specified curing temperature and techniques.	_	Х	ACI 318: 5.11– 5.13	1910.9
 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	Ι
10. Erection of precast concrete members.	_	Х	ACI 318: Ch. 16	_
11. Verification of in-situ concrete strength, prior to stressing of tendons in post- tensioned concrete and prior to removal of shores and forms from beams and structural slabs.	_	Х	ACI 318: 6.2	_
12. Inspect formwork for shape, location and dimensions of the concrete member being formed.		х	ACI 318: 6.1.1	

For SI: 1 inch = 25.4 mm.

a.

Where applicable, see also Section 1705.11, Special inspection for seismic resistance. Specific requirements for special inspection shall be included in the research report for the anchor issued by an approved b. source in accordance with ACI 355.2 or other qualification procedures. Where specific requirements are not provided, special inspection requirements shall be specified by the registered design professional and shall be approved by the building official prior to the commencement of the work.

1705.2.2.1.2 1705.3.1 Welding of reinforcing steel bars. Welding Special inspections of welding and welding inspector qualifications of *special inspectors* for reinforcing steel bars shall be in accordance with the inspection requirements of AWS D1.4 and ACI 318 for special inspection and the qualification requirements of AWS D1.4 for special inspector qualification.

Reason: This proposal is a continuation of a separate proposal that correlates Tables 1705.2.2 and 1705.3.3 with ACI 318-11. The purpose for this proposal is to relocate the requirements for special inspection of reinforcing bar welding in concrete from Item 2b of Table 1705.2.2 for steel construction to Item 2 of Table 1705.3 for concrete construction. Reinforcing bars are related to concrete construction, not steel construction. Note that the referenced standard listed in Table 1705.2.2 for reinforcing bar welding is ACI 318 for structural concrete (e.g., not also for TMS 402/ACI 530/ASCE 5 for masonry structures).

The other changes in the proposal are a consequence of the relocation, which reduces the scope of Table 1705.2.2 to specifying special inspections of cold-formed steel deck. These other changes eliminate language that becomes superfluous with the relocation.

Note that separate proposals:

- 1. Make several modifications to the titles and column headings of Tables 1705.2.2 and 1705.3 that are related to special inspections and tests as well as continuous and periodic special inspection; and
- 2. Further modify Item 2b of Table 1705.2.2 by replacing the last three listings under the item .

The final language in the titles and column headings of Tables 1705.2.2 and 1705.3 from this proposal and the proposal in Item #1 above is shown below for reference.

REQUIRED SPECIAL INSPECTION	5.2.2 S OF COLD-FORMED	DECK			
ТҮРЕ	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a		

REFERENCED

IBC

REFERENCE

TABLE 1705.3 REQUIRED SPECIAL INSPECTIONS AND TESTS OF CONCRETE CONSTRUCTION CONTINUOUS PERIODIC DEFENSES

ТҮРЕ	SPECIAL INSPECTION	SPECIAL INSPECTION	STANDARD

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

S144-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1705.2.2-S-BRAZIL.doc

S145–12 1705.2.2, Table 1705.2.2, 1705.2.2.1.1, 1705.5, Table 1705.5 (NEW), 1705.10.1, 1705.10.2, 1705.11.2, 1705.11.3

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:

1705.2.2 Steel construction other than structural steel. Special inspection for steel construction other than structural steel shall be in accordance with Table 1705.2.2 and this section.

Exceptions:

- 1. Special inspection of cold-formed steel light-frame construction for buildings and structures in Risk Category I shall not be required.
- Special inspection of cold-formed steel light-frame construction for buildings and structures in Risk Category II that are 3 stories or less in height above grade plane and that are not included in Sections 1705.10 or 1705.11, shall not be required.

TABLE 1705.2.2 REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

	VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a		
1.	Material verification of cold-formed steel deck and cold-formed steel light-frame construction:					
	 a. Identification markings to conform to ASTM standards specified in the approved construction documents. 	_	х	Applicable ASTM material standards		
	 Manufacturer's certified test reports. 	—	х			
2.	Inspection of welding:					
	a. Cold-formed steel deck and cold-formed steel light-frame construction:					
	 Floor and roof deck welds. 	_	Х	AWS D1.3		
	2) Cold-formed steel light- frame construction welds.		X	<u>AWS D1.3</u>		
	b. Reinforcing steel:					
	 Verification of weldability of reinforcing steel other than ASTM A 706. 	_	х	AWS D1.4		
	 Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and 	x	_	Section 3.5.2		

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
boundary elements of special structural walls of concrete and shear reinforcement.			
3) Shear reinforcement.	Х	—	
4) Other reinforcing steel.		Х	
3. Inspection of cold-formed steel light-fra and shear panels for conformance wit	me construction th the approved	including fran construction (ning, shear walls, diaphragms documents:
a. Inspect member locations and sizes.		X	
b. <u>Inspect bracing, strap</u> <u>bracing, drag strut and</u> <u>stiffener locations and sizes.</u>		X	
c. <u>Verify mechanical</u> <u>connectors including screws,</u> <u>powder actuated fasteners,</u> <u>bolts, anchor bolts, hold</u> <u>downs, anchors and other</u> <u>fastening components.</u>		X	<u>Applicable ASTM</u> <u>Standards</u>
d. <u>Inspect material thickness,</u> <u>grade and fastening of</u> <u>diaphragms, and sheathing</u> <u>for the lateral force resisting</u> <u>system.</u>		X	
e. Inspect connections including plates and components; screw quantity, size and spacing; powder actuated fastener quantity size and location; bolt size and location; anchor bolt size, spacing and location; hold down size location and configuration; beam hangers and framing.		X	

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also <u>Section 1705.10 Special inspections for wind resistance and</u> Section 1705.11, Special inspections for seismic resistance.

1705.2.2.1.1 Cold-formed steel. Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks <u>and cold-formed steel light-frame construction</u> shall be in accordance with AWS D1.3.

1705.5 Wood construction. *Special inspections* of the fabrication process of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2.5. *Special inspections* of site-built assemblies shall be in accordance with this section <u>and Table 1705.5</u>.

Exceptions:

- 1. <u>Special inspection of wood construction for buildings and structures in Risk Category I shall</u> not be required.
- 2. Special inspection of wood construction for buildings and structures in Risk Category II that are 3 stories or less in height above grade plane and that are not included in Sections 1705.10 or 1705.11 shall not be required.

TABLE 1705.5 REQUIRED VERIFICATION AND INSPECTION OF WOOD CONSTRUCTION						
VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARDª			
1. <u>Inspection of wood construction including framing, shear walls, diaphragms and shear panels for</u> conformance with the approved construction documents:						
a. <u>Verify grade stamp on</u> framing lumber, plywood and OSB.		X				
b. Inspect wood framing including layout, member sizes, blocking, bridging and bearing lengths.		X				
<u>c.</u> <u>Verify mechanical</u> <u>connectors including</u> <u>screws, powder actuated</u> <u>fasteners, bolts, anchor</u> <u>bolts, hold downs, anchors</u> <u>and other fastening</u> <u>components.</u>		X	<u>Applicable ASTM</u> <u>Standards</u>			
<u>d.</u> Inspect diaphragms, shear walls and wood structural panel sheathing size and thickness; sizes of framing members at adjoining panel edges and nail or staple size and spacing.		X				
e. Inspect wood connections including plates and components; nail quantity, size and spacing; bolt size and location; anchor bolt		X				

VERIFICATION AND INSPECTION	PERIODIC	REFERENCED STANDARD ^a
size, spacing and location; hold down size location and configuration; beam hangers and framing.		

 <u>a.</u> Where applicable, see Section 1705.10, Special inspections for wind resistance and Section 1705.11, Special inspections for seismic resistance.

1705.10.1 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main windforce-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

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

1705.10.2 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

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

- 1. The sheathing is gypsum board or fiberboard.
- 2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

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

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

- 1. The sheathing is gypsum board or fiberboard.
- 2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) o.c.

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. There is a large group of buildings constructed with light frame construction that is not subject to the same requirements for Special Inspection as the same buildings constructed with structural steel, concrete or masonry. This proposal seeks to correct this deficiency in the Code.

This proposal provides requirements to be consistent across both wood and cold-formed steel systems to avoid any competitive advantage of one system over the other. This proposal will improve the consistency of special inspections across all of the major structural materials.

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 unless these minor structures are subject to the existing requirements of 1705.10 and 1705.11.

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.

The proposed revisions to 1705.2 and 1705.5 improve the Special Inspection requirements for both wood and cold-formed steel light-frame construction in a manner consistent with Special Inspection requirements for structural steel, concrete and masonry.

The proposed revisions to 1705.10 and 1705.11 are to coordinate between the additional requirements for Special Inspections in high seismic and high wind conditions and the proposed provisions. The proposed changes to 1705.10 and 1705.11 do not reduce the requirements of these sections they only prevent the exceptions for these sections from conflicting with the new requirements. In addition, notes are added to the tables to refer to 1705.10 and 1705.11 for additional requirements.

There will be no increase in construction cost due to the increased Special Inspection that will take place. Currently structural engineers provide for these inspections in project specifications. However, individual requirements vary greatly and there is not a consistent level of requirements. Standardization of these requirements in the Code will reduce delays and added costs due to confusion created by varying specifications. The improved field quality assurance will improve safety and reduce field errors resulting in a savings in construction cost and schedule. The improved public safety and potential reduction in construction cost support adoption of this proposal.

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

S145-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				1705.2.2-SHARMAN.doc

S146–12 Table 1705.2.2

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing myself (pbrazil@reidmiddleton.com)

Revise as follows:

TABLE 1705.2.2 REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
1. Material verification of cold-formed steel deck:			
a. Identification markings to conform to ASTM			Applicable ASTM
standards specified in the approved construction	—	Х	material standards
documents.			
b. Manufacturers' certified test reports.	_	Х	
2. Inspection of welding:	_	_	
a. Cold-formed steel deck			
1. Floor and roof deck welds		Х	AWS D1.3
b. Reinforcing steel:			
1. Verification of weldability of reinforcing steel			
other than ASTM A 706.	-	Х	
Reinforcing steel resisting flexural and axial	Х	—	
forces in intermediate and special moment			AWS D1.4
frames, and boundary elements of special			ACI 318 Section 3.5.2
structural walls of concrete and shear			
reinforcement.			
Shear reinforcement.	Х	—	
Other reinforcing steel.		Х	
3. Installation of open web steel joists and joist girders		X	
in accordance with the approved construction			
documents and steel joist placement plans			

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspection for seismic resistance.

Reason: The purpose for this proposal is to require special inspections for the installation of open web steel joists and joist girders. Their structural design is sufficiently complex to warrant inspection from a person with the expertise of a special inspector who is approved by the building official as having the competence necessary to inspect the installation of the joists. Refer to the definitions of "special inspection" and "special inspector" for further information. Examples of the complexity of the structural design that warrant special inspection of the installation are the bearing seat attachments, field splices and bridging attachments.

The standard specifications for open web steel joists (SJI-K-2010 and SJI-LH/DLH-2010), joist girders (SJI-JG-2010) and composite steel joists (SJI-CJ-2010) by the Steel Joist Institute contain provisions for inspections but these are limited to inspections by the manufacturer before shipment to verify compliance and workmanship with the requirements of the specifications. Refer to Section 5.12 of SJI-K-2010, Section 104.13 of SJI-LH/DLH-2010, Section 1004.10 of SJI-G-2010 and Section 104.13 of SJI-CJ-2010. The sections of the SJI standards noted above are also referenced in Section 4 of the codes of standard practice for steel joists and joist girders (no identifier) and composite steel joists (SJI-CJCOSP-2010). The identifiers cited above match those from the published documents but they are abbreviated in Chapter 35 of the 2012 IBC to K-10, LH/LDH-10, JG-10 and CJ-10, respectively; and are specified as SJI-K-1.1, SJI-LH/LDH-1.1, SJI-JG-1.1 and SJI-CJ-1.0, respectively, in Section 2207.1. Note that the codes of standard practice published by the Steel Joist Institute are not referenced standards of the 2012 IBC.

S146-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				T1705.2.2 #1-S-BRAZIL.doc

S147–12 Table 1705.2.2, Table 1705.3

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

TABLE 1705.2.2 REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
1. Material verification of cold-formed steel deck:			
 Identification markings to conform to ASTM standards specified in the approved construction documents. 	_	х	Applicable ASTM material standards
 Manufacturers' certified test reports. 	—	Х	
2. Inspection of welding:	—	—	
a. Cold-formed steel deck			
 Floor and roof deck welds 		Х	AWS D1.3
b. Reinforcing steel <u>bars</u>:			
 Verification of weldability of reinforcing steel bars other than ASTM A 706. 	_	х	
 Reinforcing steel bars resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement. 	Х	Ι	AWS D1.4 ACI 318 Section 3.5.2
3. Shear reinforcement.	Х	_	
4. Other reinforcing steel bars.	—	Х	

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspection for seismic resistance.

TABLE 1705.3 REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

	VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
1.	Inspection of reinforcing steel <u>reinforcement</u> , including prestressing tendons, and placement.	_	х	ACI 318: 3.5, 7.1–7.7	1910.4
2.	Inspection of reinforcing steel <u>bar</u> welding in accordance with Table 1705.2.2, Item 2b.	_	_	AWS D1.4 ACI 318: 3.5.2	_

(Portions of table not shown remain unchanged)

Reason: The purpose for the proposal is to update Tables 1705.2.2 and 1705.3 for consistency with ACI 318-11, which does not use the term "reinforcing steel" but does use "(concrete) reinforcement" and "reinforcing bars." In Section 2.2 of ACI 318-11, "deformed reinforcement" is defined as including bar mats, deformed wire and welded wire reinforcement as well as deformed reinforcing bars. Section 3.5.1 requires reinforcement in concrete to be deformed reinforcement except that plain reinforcement is permitted for spirals and prestressing steel and reinforcement consisting of headed shear studs, structural steel, steel pipe or steel tubing is also permitted. Section 3.5.2 on welding, however, only specifies reinforcing bars. Note that Section 2.2 of ACI 318-11 also defines "reinforcement," "plain reinforcement," "headed deformed bars," "prestressing steel" and "tendon."

Note that separate proposals:

- 1. Make several modifications to the titles and column headings of Tables 1705.2.2 and 1705.3 that are related to special inspections and tests as well as continuous and periodic special inspection; and
- 2. Further modify Item 2b of Table 1705.2.2 by relocating the language to Table 1705.3; and replacing the last three listings under the item.

The final language in the titles and column headings of Tables 1705.2.2 and 1705.3 from the proposal in Item #1 above is shown below for reference.
TABLE 1705.2.2

REQUIRED SPECIAL INSPECTIONS OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL						
ТҮРЕ	CONTINUOUS		PERIODIC	C REFERENCED STANDARD ^a		
TABLE 1705.3 REQUIRED SPECIAL INSPECTIONS AND TESTS OF CONCRETE CONSTRUCTION						
ТҮРЕ	CONTINUOUS SPECIAL INSPECTION	PERIODI SPECIAL INSPECTIO	C REFERE	INCED ARD ^ª	IBC REFERENCE	
	•				•	

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

S147-12

••••				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				T1705.2.2 #2-S-BRAZIL.doc

S148–12 Table 1705.2.2

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

TABLE 1705.2.2 REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a
1. Material verification of cold-formed steel deck:	•		·
a. Identification markings to conform to ASTM			Applicable ASTM
standards specified in the approved construction	_	Х	material standards
documents.			
b. Manufacturers' certified test reports.	—	Х	
2. Inspection of welding:	—	—	
a. Cold-formed steel deck	-		
1. Floor and roof deck welds		Х	AWS D1.3
b. Reinforcing steel <u>bars</u>:			
 Verification of weldability of reinforcing steel bars 			
other than ASTM A 706.	_	Х	
Reinforcing steel resisting flexural and axial	×	—	
forces in intermediate and special moment			
frames, and boundary elements of special			
structural walls of concrete and shear			AWS D1.4
reinforcement.			ACI 318 Section 3.5.2
3. Shear reinforcement.	×	-	
4. Other reinforcing steel.	_	×	
2. Single-pass fillet welds, maximum 5/16"		<u>X</u>	
3. <u>All other welds</u>	<u>X</u>		

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspection for seismic resistance.

Reason: This proposal is a continuation of separate proposals that correlate Tables 1705.2.2 and 1705.3.3 with ACI 318-11 and relocate the requirements for special inspection of reinforcing bar welding from Table 1705.2.2 to Table 1705.3.3. The purpose for this proposal is to simplify the required extent (continuous or periodic) of special inspection for the welding of reinforcing bars, which is currently based on the structural design (e.g., resisting flexural, axial or shear forces). The proposal changes the extent to continuous special inspection of all welding of reinforcing bars except for single-pass fillet welds that are a maximum of 5/16-inch where periodic special inspection is permitted. This will also be consistent with the historical approach taken by the building code for the extent of special inspections related to welding.

Should this proposal and the proposal to relocate the requirements for special inspection of reinforcing bar welding from Table 1705.2.2 to Table 1705.3.3 both be approved by the ICC membership, our intent is that the language in this proposal at Item 2b of Table 1705.2.2 be placed in Item 2 of Table 1705.3 and that Item 2 of Table 1705.3.3 read as follows:

2.	Inspection of reinforcing bar welding:				
а.	Verification of weldability of reinforcing bars			AWS D1.4	
	other than ASTM A 706.	-	Х	ACI 318: 3.5.2	
b.	Single-pass fillet welds, maximum 5/16"	_	Х		
С.	All other welds	Х	-		

Note that a separate proposal also makes several modifications to the title and column headings of Table 1705.2.2 that are related to special inspections and tests as well as continuous and periodic special inspection.

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

S148-12

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

T1705.2.2 #3-S-BRAZIL.doc

S149–12 202, 1705.3, 1705.11.6, 1705.12.3, 2105.1, 2105.2.2.2.1, 2204.2.1, 2207.4

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

SECTION 202 DEFINITIONS

SPECIFIED COMPRESSIVE STRENGTH OF MASONRY, f'_m . Minimum compressive strength, expressed as force per unit of net cross-sectional area, required of the *masonry* used in construction by the *approved construction documents*, and upon which the project design is based. Whenever the quantity f'_m is under the radical sign, the square root of numerical value only is intended and the result has units of pounds per square inch (psi) (MPa).

Revise as follows:

1705.3 Concrete construction. The *special inspections* and verifications for concrete construction shall be as required by this section and Table 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 <u>approved</u> 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.

1705.11.6 Mechanical and electrical components. *Special inspection* for mechanical and electrical components shall be as follows:

- 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 anchorage of other electrical equipment in structures assigned to *Seismic Design Category* E or F;
- Periodic special inspection is required during the installation and anchorage of piping systems designed to carry 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 ductwork 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 <u>approved</u> construction documents require a nominal clearance of 1/4 inch (6.4 mm) or less between the equipment support frame and restraint.

1705.12.3 Seismic certification of nonstructural components. The *registered design professional* shall specify on the *approved* construction documents the 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 1705.12.

Revise as follows:

2105.1 General. A quality assurance program shall be used to ensure that the constructed masonry is in compliance with the <u>approved</u> construction documents. The quality assurance program shall comply with the inspection and testing requirements of Chapter 17.

2105.2.2.1 General. The compressive strength of clay and concrete masonry shall be determined by the prism test method:

- 1. Where specified in the <u>approved</u> construction documents.
- 2. Where masonry does not meet the requirements for application of the unit strength method in Section 2105.2.2.1.

Revise as follows:

2204.2.1 Anchor rods. Anchor rods shall be set in accordance with the <u>approved</u> construction documents. The protrusion of the threaded ends through the connected material shall fully engage the threads of the nuts, but shall not be greater than the length of the threads on the bolts.

2207.4 Steel joist drawings. Steel joist placement plans shall be provided to show the steel joist products as specified on the <u>approved</u> construction documents and are to be utilized for field installation in accordance with specific project requirements as stated in Section 2207.2. Steel placement plans shall include, at a minimum, the following:

- 1. Listing of all applicable loads as stated in Section 2207.2 and used in the design of the steel joists and joist girders as specified in the *approved* construction documents.
- 2. Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder configurations are as indicated in the SJI catalog).
- 3. Connection requirements for:
 - 3.1. Joist supports;
 - 3.2. Joist girder supports;
 - 3.3. Field splices; and
 - 3.4. Bridging attachments.
- 4. Deflection criteria for live and total loads for non-SJI standard joists.
- 5. Size, location and connections for all bridging.
- 6. Joist headers.

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

Reason: The purpose for the proposal is to update references to "construction documents" in the building code. Section 107.1 contains the requirements for the submittal of construction documents with each permit application and Section 107.3 requires the building official to approve the construction documents for permit issuance. The building code typically specifies "construction documents" before permit issuance and "approved construction documents" after permit issuance but there are exceptions and this proposal adds "approved" for those cases.

The instances of "construction documents" not preceded by "approved" in the building code typically occur in provisions that require the designers to specify information in the construction documents or to design the building or structure to meet specified requirements. Compliance with these provisions is only possible before the construction documents are approved. These are located in Sections 104.2, 105.3(4), 105.3.1, 105.4, 107.1, 107.1, 107.2, 107.2.1, 107.2.2, 107.2.3, 107.2.4, 107.2.5, 107.3.1, 107.3.2, 107.3.3, 107.3.4.1, 414.1.3, 907.1.1, 909.2, 909.3, 909.4, 909.21.2, 1603.1, 1603.1.6, 1603.1.9, 1607.5, 1705.11.6(5), 1705.12.3, 1901.3, 2101.3, 2101.3.1, 2207.2, 2403.2, 3103.2, 3303.2, G104.2, H105.2, K104.1, K104.2 and K105.5.

The instances of "approved construction documents" in the building code are located in Sections 107.4, 107.5, 114.4, 202 ("certificate of compliance" and "structural observation"), 1704.2.4, 1704.2.5.1, 1704.2.5.2, 1705.2-Exc., 1705.6, 1705.8, 1705.9, 1705.13, 1705.14, 1810.3.5.2.2-Exc., 1910.7, 2207.5, and 2403.1; and Table 1705.2.2. Note that "approved" precedes "geotechnical report and the construction documents" in Sections 1705.6, 1705.8 and 1705.9. In Section 1705.7, however, "*approved*" precedes "*instruction documents*," which is apparently an advertent error made during the development of the 2012 IBC. On August 9, 2011, I submitted a request to the ICC that this be posted as errata but, as of January 3, 2012, the submittal deadline for Group A change proposals, a posting for the 2012 IBC had not yet been made on the ICC website.

All instances of "construction documents" in the building code were considered and are either in the proposal or are listed in the paragraphs immediately above.

A separate proposal places the provisions of Section 1705.12.3 into two subsections (Sections 1705.12.3 and 1705.12.4) to provide effective charging language for the corresponding provisions in ASCE 7-10. Should both proposals be approved by the ICC membership, our intent is that Sections 1705.12.3 and 1705.12.4 both read: "...the *registered design professional* shall specify on the *approved construction documents* the requirements..."

A separate proposal deletes the definition of "specified" in Section 202. Should both proposals be approved by the ICC membership, our intent is that the definition be deleted.

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

S149-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.3-S-BRAZIL.doc

S150–12 1705.3

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1705.3 Concrete construction. The *special inspections* and verifications for concrete construction shall be as required by this section and Table 1705.3. <u>The following exceptions shall not apply where Section</u> <u>1705.10 or 1705.11 invoke *special inspections* or where special inspection of column anchor bolts for structural steel lateral force resisting frames is required by Section 1705.11.1.</u>

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.
- 21. Isolated spread concrete footings and continuous concrete footings supporting walls of buildings three stories or less above grade plane that are fully supported on earth or rock and where any of the following conditions apply:
- 2.1. <u>1.1</u> The footings support walls of light-frame construction;
- 2.2. <u>1.2</u> The footings are designed in accordance with Table 1809.7; or
- **2.3.** <u>1.3</u> 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.
- <u>32.</u> 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).
- 43. Concrete foundation walls constructed in accordance with Table 1807.1.6.2.
- 54. Concrete patios, driveways and sidewalks, on grade.

Reason: Special inspections for concrete include such items as proper mix, reinforcing steel, bolts installed in concrete, postinstalled anchors, formwork, concrete placement, curing, etc. Under Exception 1, the building could be of any type (concrete, masonry, steel, light frame), utilize high-strength concrete, and have heavily-loaded "isolated" footings. This change proposal makes the exception for isolated spread footings subject to the same limitations as those for continuous footings.

Note also that there are no additional inspection requirements for concrete under 1705.10 (wind), 1705.11 (seismic) and 1705.12 (testing for seismic). Therefore, anchorage elements such as anchor bolts for holdowns or steel frames used in the lateral system would not require special inspection when used in conjunction with light-frame construction or at isolated footings. The proposed change ensures that, when special inspection for light-frame construction is required by Section 1705.10 or 1705.11, the placement of anchor bolts will require special inspection, and that the placement of anchor bolts for steel frames resisting seismic loads will also require special inspection.

S150-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					1705.3-S-KERR.doc

S151–12 Table 1705.3, Table 1705.6, Table 1705.7, Table 1705.8

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION							
VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE			
 Inspection of Inspect reinforcing steel, including prestressing tendons, and <u>verify</u> placement. 	_	х	ACI 318: 3.5, 7.1–7.7	1910.4			
 Inspection of <u>R</u>einforcing steel <u>bar</u> welding: in accordance with Table 1705.2.2, Item 2b. 	_	_	AWS D1.4 ACI 318: 3.5.2	_			
 a. <u>Verify weldability of reinforcing</u> bars other than ASTM A 706; b. <u>Inspect single-pass fillet welds</u>, maximum 5(16"), and 	=	=					
maximum 5/16; and	~	<u>~</u>					
3.Inspection of Inspect anchors cast in concrete where allowable loads have been increased or where strength design is used.	<u> </u>	х	ACI 318: 8.1.3, 21.1.8	1908.5, 1909.1			
4. Inspection of Inspect anchors post- installed in hardened concrete members. ^b		Х	ACI 318: 3.8.6, 8.1.3, 21.1.8	1909.1			
5.Verify ing use of required design mix.	_	Х	ACI 318: Ch. 4, 5.2–5.4	1904.2.2, 1910.2, 1910.3			
6. 7 <u>.At the time fresh concrete is</u> <u>sampled to During concrete</u> <u>placement.</u> fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete.	Х	_	ASTM C 172 ASTM C 31 ACI 318: 5.6, 5.8	1910.10			
 Inspection of Inspect concrete and shotcrete placement for proper application techniques. 	Х		ACI 318: 5.9, 5.10	1910.6, 1910.7, 1910.8			
 Inspection for <u>Verify</u> maintenance of specified curing temperature and techniques. 	_	Х	ACI 318: 5.11–5.13	1910.9			
 9. Inspection of Inspect prestressed concrete for: a. Application of prestressing forces-; and b. Grouting of bonded prestressing tendons in the seismic force-resisting system. 	x x	_	ACI 318: 18.20 ACI 318: 18.18.4	_			

TABLE 1705.3 EQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
10. <u>Inspect e</u> rection of precast concrete members.	—	х	ACI 318: Ch. 16	—
11. Verification of Verify in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs.	_	х	ACI 318: 6.2	_
12. Inspect formwork for shape, location and dimensions of the concrete member being formed.	_	Х	ACI 318: 6.1.1	_

(No change to footnotes)

TABLE 1705.6

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.	—	Х
2.	Verify excavations are extended to proper depth and have reached proper material.	_	Х
3.	Perform classification and testing of compacted fill materials.	—	Х
4.	Verify use of proper materials, densities and lift thicknesses during placement and compaction of compacted fill.	х	_
5.	Prior to placement of compacted fill, observe inspect subgrade and verify that site has been prepared properly.	_	Х

TABLE 1705.7

REQUIRED VERIFICATION AND INSPECTION OF DRIVEN DEEP FOUNDATION ELEMENTS

	VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
1.	Verify element materials, sizes and lengths comply with the requirements.	Х	_
2.	Determine capacities of test elements and conduct additional load tests, as required.	Х	_
3.	Observe Inspect driving operations and maintain complete and accurate records for each element.	Х	_
4.	Verify placement locations and plumbness, confirm type and size of hammer, record number of blows per foot of penetration, determine required penetrations to achieve design capacity, record tip and butt elevations and document any damage to foundation elements.	Х	
5.	For steel elements, perform additional inspections in accordance with Section 1705.2.	_	_

	VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
6.	For concrete elements and concrete-filled elements, perform additional inspections in accordance with Section 1705.3.	_	_
7.	For specialty elements, perform additional inspections as determined by the registered design professional in responsible charge.	_	_

TABLE 1705.8 REQUIRED VERIFICATION AND INSPECTION OF CAST-IN-PLACE DEEP FOUNDATION FLEMENTS

	VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
1.	Observe Inspect drilling operations and maintain complete and accurate records for each element.	Х	_
2.	Verify placement locations and plumbness, confirm element diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end-bearing strata capacity. Record concrete or grout volumes.	Х	_
3.	For concrete elements, perform additional inspections in accordance with Section 1705.3.	—	—

Reason: This proposal is a continuation of separate proposals that:

1. Correlate Tables 1705.2.2 and 1705.3.3 with ACI 318-11;

2. Relocate the requirements for special inspection of reinforcing bar welding from Table 1705.2.2 to Table 1705.3.3; and

3. Simplify the required extent of special inspection for the welding of reinforcing bars.

The primary purpose for this proposal is to revise Tables 1705.6, 1705.7 and 1705.8 for consistency with Table 1705.3 and to clarify the scope of special inspections in Table 1705.3. The changes from "inspection" to "inspect" in Table 1705.3 are made to reduce confusion with the provisions in the building code for inspections by building inspectors and special inspections by special inspectors. The changes from "observe" to "inspect" in Tables 1705.6, 1705.7 and 1705.8 are made to reduce confusion with the provisions in the building code for inspections by building inspectors and special inspections by special inspectors. The changes from "observe" to "inspect" in Tables 1705.6, 1705.7 and 1705.8 are made to reduce confusion with the provisions in the building code for structural observation, which have been reported to us by several code users. The changes are also made for consistency with the definitions of "special inspection" and "special inspector" in Section 202.

The current language In Item 6 of Table 1705.3 specifies the performance of slump and air content tests "at the time fresh concrete is sampled to fabricate specimens for strength tests." The effect of this is that Table 1705.3 does not specify the sampling of fresh concrete for the purpose of performing strength tests and the proposal changes Item 6 to do so.

The current language in Item 9 of Table 1705.3 limits special inspections of the grouting of bonded prestressing tendons to those that are elements of the seismic force-resisting system. In our judgment, it is equally important that the grouting of these tendons be subject to special inspections where they are elements of gravity or wind force-resisting systems and the proposal changes Item 9(b) to do so.

For Item 2 in Table 1705.3, our intent for the dashes opposite the charging language is that they be deleted.

Note that a separate proposal also makes several modifications to the title and column headings of Table 1705.3 that are related to special inspections and tests as well as continuous and periodic special inspection and the final language the title and column headings from that proposal is shown below.

TABLE 1705.3 REQUIRED SPECIAL INSPECTIONS AND TESTS OF CONCRETE CONSTRUCTION

ТҮРЕ	CONTINUOUS SPECIAL INSPECTION	PERIODIC SPECIAL INSPECTION	REFERENCED STANDARD ^a	IBC REFERENCE
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S151-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				T1705.3-S-BRAZIL.doc

S152–12 1705.5.1

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1705.5.1 High-load diaphragms. High-load diaphragms designed in accordance with Section 2306.2 shall be installed with *special inspections* as indicated in Section 1704.2. 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 <u>construction documents</u>. 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 <u>construction documents</u>.

Reason: The purpose for the proposal is to replace the term "building plans," which is not defined in the building code, with "construction documents," which is defined in Section 202. The instances of "building plans" in the proposal are the only ones in the 2012 *International Building Code* other than in Section 911.1.5(12) where the context is such that changing the term would not be appropriate.

S152-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1705.5.1-S-BRAZIL.doc

S153–12 1705.5

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1705.5 Wood construction. Special inspections of the fabrication process of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2.5. Special inspections of site-built assemblies shall be in accordance with this section.

Reason: Special inspection should be of the item, not the "process". The last sentence is not necessary and confuses the issues. The next two sections of 1705.5 (regarding high-load diaphragms and metal-plate-connected wood truss bracing) state the special inspections required and do not need to be invoked by the deleted language.

S153-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.5-S-KERR.doc

S154–12 1705.6, 1705.8, 1705.9

Proponent: Mark Gilligan, representing self (mark@gilligan.name)

Revise as follows:

1705.6 Soils. Special inspections for existing site soil conditions, fill placement and load-bearing requirements shall be as required by this section and Table 1705.6. 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.

1705.8 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 1705.8. The *approved* geotechnical report, and the *construction documents* prepared by the *registered design professionals*, shall be used to determine compliance.

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* approved construction documents. The *approved* geotechnical report and the *construction documents* prepared by the *registered design professional* shall be used to determine compliance.

Reason: This change makes it clear that special inspections will be based on the approved construction documents as is the rest of the work and not on the geotechnical report. The geotechnical report is not written to be enforceable as a document to direct the contractor or inspector. Thus the use of a geotechnical report by the contractor to construct the work or the inspectors to inspect it will create the potential for confusion.

The proper role of the geotechnical report is to communicate initial recommendations to the design team and as a resource document that provides information to the Contractor on existing ground conditions. Any criteria or direction needed by the contractor, that exists in the geotechnical report, must be included in the construction documents. The code does not say how the information will be incorporated in the construction documents but it is expected that the geotechnical engineer who prepared the report would play an active role in the process.

The last sentence is 1705.6 is not needed as the requirements for special inspection during fill placement are included as Item 4 in Table 1705.6.

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

S154-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1705.6-S-GILLIGAN.doc

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S155–12 1705.7.1 (NEW), 1705.8.1 (NEW)

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Add new text as follows:

1705.7.1 Special inspection by a registered design professional. Where higher allowable stresses are used in the design of driven deep foundations in accordance with Section 1810.3.2.8, special inspections shall be performed under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and deep foundations.

1705.8.1 Special inspection by a registered design professional. Where higher allowable stresses are used in the design of cast-in-place deep foundations in accordance with Section 1810.3.2.8, special inspections shall be performed under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and deep foundations.

Reason: The special requirements of Section 1810.3.2.8 that the installation of piles designed with higher allowable stresses must be done under the supervision of a registered design professional knowledgeable in the field of soil mechanics and deep foundations should be reflected here so that the special requirements for the qualifications of the special inspector are clarified.

A related proposal seeks to eliminate the special allowable stresses used in Chapter 18 for deep foundation elements, and eliminate the special requirements of Section 1810.3.2.8. If that proposal is accepted, this proposal is not necessary, as the special inspection requirements for deep foundation elements are adequate.

S155-12					
Public Hearing:	Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF		
	-				1705.7.1 (NEW)-S-KERR.doc

S156–12 1705.10.1, 1705.10.1.1 (NEW), 1705.10.1.2 (NEW), 1705.10.1.3 (NEW), 1705.10.2, 1705.10.2.1 (NEW), 1705.10.2.2 (NEW), 1705.11.2, 1705.10.11.2.1 (NEW), 1705.10.11.2.2 (NEW), 1705.11.2.3 (NEW), 1705.11.3, 1705.11.3.1 (NEW), 1705.11.3.2 (NEW)

Proponent: Stephen Kerr, S.E., Structural Engineers Association of California (skerr@jwa-se.com)

Revise as follows:

1705.10.1 Structural wood. Special inspection for wood construction within the main windforce-resisting system shall be as required by this section. Special inspection for wood construction in accordance with this section shall also be provided where vertical elements of the main windforce-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls. Continuous special inspection is required during field gluing operations of elements of the main windforce-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, and hold-downs.

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

<u>1705.10.1.1 Field gluing operations</u>. Continuous special inspection is required during field gluing operations of wood elements of the main windforce-resisting system.

1705.10.1.2 Shear walls. Periodic special inspection shall be required for the sheathing fastening, and for other connections within the shear wall. Such connections shall include hold-down or tie-down connections, sill plate and sole plate anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

Exception: Special inspection for wood shear walls is not required where the sheathing is gypsum board or fiberboard or where the fastener spacing along shear wall sheathing edges is more than 4 inches (102 mm) on center,

1705.10.1.3 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, diaphragm chord connections and splices, and collector connections and fastening.

Exception: Special inspection for horizontal wood diaphragms is not required where the sheathing is gypsum board or fiberboard or where the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.10.2 Cold-formed steel light-frame construction. Special inspection for cold-formed light-frame construction within the main windforce-resisting system shall be as required by this section. Special inspection for cold-formed light-frame construction in accordance with this section shall be provided where vertical elements of the main windforce-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls. Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main windforce-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).

1705.10.2.1 Shear walls and strap-braced walls. Periodic special inspection shall be required for the sheathing fastening, the welding or screw attachment of the strap bracing, and for other connections within the shear wall or strap-braced wall. Such connections shall include hold-down or tie-down connections, bottom track anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

Exception: Special inspection for cold-formed light-frame shear walls is not required 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, and the fastener spacing along sheathing edges is more than 4 inches (102 mm) on center.

1705.10.2.2 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, welding or screw attachment of diagonal strap bracing, diaphragm chord connections and splices, and collector connections and fastening.

Exception: Special inspection for cold-formed light-frame horizontal diaphragms is not required where either of the following apply:

- 1. The sheathing is gypsum board or fiberboard.
- The sheathing is wood structural panel or steel sheets on only one side of the framing, and the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.11.2 Structural wood. Special inspection for wood construction within the seismic force-resisting system shall be as required by this section. Special inspection for wood construction in accordance with this section shall be provided where vertical elements of the seismic force-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.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 shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the main seismic force-resisting system where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c).

1705.11.2.1 Field gluing operations. Continuous special inspection shall be required during field gluing operations of wood elements of the seismic force-resisting system.

1705.11.2.2 Shear walls. Periodic special inspection shall be required for the sheathing fastening, and for other connections within the shear wall. Such connections shall include hold-down or tie-down connections, sill plate and sole plate anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

Exception: Special inspection for wood shear walls is not required where the sheathing is gypsum board or fiberboard or where fastener spacing along shear wall sheathing edges is more than 4 inches (102 mm) on center,

1705.11.2.3 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, diaphragm chord connections and splices, and collector connections and fastening.

Exception: Special inspection for horizontal wood diaphragms is not required where the sheathing is gypsum board or fiberboard or where least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.11.3 Cold-formed steel light-frame construction.<u>Special inspection for cold-formed light-frame</u> construction within the seismic force-resisting system shall be as required by this section. <u>Special</u> inspection for cold-formed light-frame construction in accordance with this section shall be provided where vertical elements of the seismic force-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear wallsPeriodic 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) on center (o.c).

1705.11.3.1 Shear walls and strap-braced walls. Periodic special inspection shall be required for the sheathing fastening, the welding or screw attachment of the strap bracing, and for other connections within the shear wall or strap-braced wall. Such connections shall include hold-down or tie-down connections, bottom track anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

Exception: Special inspection for cold-formed light-frame shear walls is not required 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, and the fastener spacing along sheathing edges is more than 4 inches (102 mm) on center.

1705.11.3.2 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, welding or screw attachment of diagonal strap bracing, diaphragm chord connections and splices, and collector connections and fastening.

Exception: Special inspection for cold-formed light-frame horizontal diaphragms is not required 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 framing, and the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

Reason: As currently written, it is not clear how to apply the exceptions to special inspection for wind and seismic as applicable to wood framing and cold-formed steel light frame construction (together "light-frame construction). The exceptions use "fastener spacing of the sheathing" as the trigger for special inspection. However, the following aspects of light-frame construction are not covered adequately by the exception language:

- 1. Fastener spacing for shear walls could vary throughout the building. It is not clear that the exception would only be applicable to the particular shear wall or diaphragm with the larger fastening spacing, and to the other elements of the lateral force-resisting system associated with that shear wall or diaphragm.
- 2. The main elements of the lateral force-resisting system of light-frame buildings are the shear walls and the horizontal diaphragms. Elements associated with the shear walls include hold-downs, and the parts use to make connection to the foundation or the horizontal diaphragms, including sill plates, sole plates, bottom tracks, and blocking and framing clips. Elements associated with the horizontal diaphragms include chords, collectors, and elements used to anchor concrete and masonry walls for out-of-plane forces (such as blocking, straps, and hold-down hardware used horizontally. As written, it is not clear when special inspection would be required for the elements associated with the shear walls and diaphragms.
- 3. Shear wall sheathing is fastened at the sheathing edges, and in the middle of the panel. It is not clear that the reference to sheathing fastening is intended to apply to fastening along sheathing edges.
- 4. Diaphragm sheathing fastening is often specified with different spacing at sheathing edges, and at diaphragm boundaries. It is not clear what fastening (edge or boundary) is being referred to, or what portions of a horizontal diaphragm and associated elements would be affected by the exception.
- 5. Buildings of pre-dominantly light-frame construction often use vertical lateral force-resisting elements made up of other materials, such as steel frames, or concrete shear walls or masonry shear walls. It is not clear under what conditions special inspection would be required for the elements used to connect such vertical lateral force-resisting elements to the light-frame building system.
- 6. Light-frame diaphragms are often used in buildings where all of the vertical lateral force-resisting elements are made up of other materials, such concrete tilt-up shear or masonry shear walls. It is not clear under what conditions special inspection would be required for the wood, light-frame, and/or steel elements used to anchor the concrete or masonry walls for out-of plane forces.

The proposed change includes similar revisions to the provisions for structural wood, and for cold-formed light-frame construction. Shear walls and horizontal diaphragms are handled separately and the elements associated with each are identified. This

makes it clear, once the special inspection is triggered (by fastener spacing, double sided sheathing, or the use of strap bracing) which elements other than the sheathing fastening, require inspection.

The requirements for inspection of anchorage elements in horizontal diaphragms for out-of-plane support of concrete and masonry walls are made explicit.

S156-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1705.10.1-S-KERR.doc

S157–12 1705.10.2, 1705.11.3, 1705.11.5, 1705.11.6, 1705.11.7

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1705.10.2 Cold-formed steel light-frame construction. Periodic special inspection is required during for welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main windforce-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.).

1705.11.3 Cold-formed steel light-frame construction. Periodic special inspection is required during for 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.

1705.11.5 Architectural components. Periodic *special inspection* is required during for 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/m₂) or less.
- 3. Special inspection is not required for interior nonbearing walls weighing 15 psf (73.5 N/m₂) or less.

1705.11.6 Mechanical and electrical components. *Special inspection* for mechanical and electrical components shall be as follows:

 Periodic special inspection is required during for the anchorage of electrical equipment for emergency or standby power systems in structures assigned to Seismic Design Category C, D, E or F;

- Periodic special inspection is required during for the anchorage of other electrical equipment in structures assigned to Seismic Design Category E or F;
- Periodic special inspection is required during for the installation and anchorage of piping systems designed to carry hazardous materials and their associated mechanical units in structures assigned to Seismic Design Category C, D, E or F;
- Periodic special inspection is required during for the installation and anchorage of ductwork designed to carry hazardous materials in structures assigned to Seismic Design Category C, D, E or F; and
- Periodic special inspection is required during for 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.

1705.11.7 Storage racks. Periodic *special inspection* is required during 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.

Reason: The purpose for this proposal is to correlate the requirements for periodic special inspection in Section 1705 with the definition of periodic special inspection in Section 202, which defines it as special inspection "by the special inspector who is intermittently present where the work to be inspected **has been** (emphasis mine) or is being performed." The proposal changes "during," which is consistent with the definition of continuous special inspection, to "for," which is consistent with the definition of periodic special inspection. The proposal also makes the requirements for periodic special inspection in Section 1705 internally consistent in that the other requirements for periodic special inspection state "for" and not "during" (e.g., Sections 1705.10.1, 1705.10.3, 1705.11.2, 1705.11.5.1 and 1705.11.8).

For more information on the intent of the definition of periodic special inspection, as well the definitions of special inspection and continuous special inspection, refer to ICC Proposal S111-09/10-AMPC, notably the reason statement that accompanied the public comment.

S157-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.10.2-S-BRAZIL.doc

S158–12 1705.10.2

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

1705.10.2 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within <u>elements of</u> the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: Special inspections is are not required for cold- formed steel light-frame shear walls, braces, and diaphragms collectors (drag struts) and hold-downs, including screwing, bolting, anchoring, and other fastening to components of the seismic-force resisting system where either of the following apply:

- 1. The sheathing is gypsum board or fiberboard.
- 2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

Reason: This proposal makes minor changes to this section. In the exception, word "braces" is deleted, since Items 1 and 2 of the exception discuss only sheathing used on shear walls and not braced walls. Revisions to the remainder of the section are to ensure consistency with the wood exception in Section 1705.10.1 and eliminate confusion.

S158-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.10.2-S-MANLEY.doc
S159–12 1705.10.3

Proponent: Stephen Kerr, S.E. Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1705.10.3 Wind-resisting components. Periodic special inspection is required for <u>fastening of</u> the following systems and components:

- 1. Roof cladding covering, roof deck, and roof framing connections.
- 2. Wall cladding Exterior covering, and wall connections to roof and floor diaphragms and framing.

Reason: The purpose of this change is to provide clarity and detail for the special inspection requirements for wind-resisting components in high-wind regions. The 2009 IBC identified "roof cladding and roof framing connections" and "wall connections to roof and floor diaphragms and framing" as wind-resisting components that needed to be included in the statement of special inspections, but only referenced "roof cladding" and "wall cladding" in the section describing the actual inspection. However, as part of the reorganization of Chapter 17 approved in the previous code change cycle, the more detailed language was deleted when the inspection requirements were combined with the requirements for inclusion in the statement of special inspections. In addition, "cladding" is not defined.

This proposal restores the more detailed description of the elements requiring special inspection, and uses terms defined in the code to identify the elements.

S159-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1705.10.3-S-KERR.doc

S160–12 1705.11.1, 1705.12.2

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1705.11.1 Structural steel. Special inspection for structural steel in the seismic force-resisting systems of structures assigned to Seismic Design Category B, C, D, E or F shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Special inspections of structural steel are not required in the seismic force-resisting systems of structures assigned to Seismic Design Category <u>B or</u> C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.

1705.12.2 Structural steel. Testing for <u>of</u> structural steel <u>in the seismic force-resisting systems of</u> <u>structures assigned to Seismic Design Category B, C, D, E or F</u> shall be <u>performed</u> in accordance with the quality assurance requirements of AISC 341.

Exception: Testing of structural steel <u>is not required</u> in <u>the seismic force-resisting systems of</u> structures assigned to *Seismic Design Category <u>B or</u>* C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.

Reason: The proposal correlates the requirements for special inspections and testing of structural steel in Section 1705.11.1 and 1705.11.2 with the applicability of AISC 341-10. The proposal is also a continuation of separate proposals that simplify the provisions of Section 1705.11 on required special inspections for seismic resistance and Section 1705.12.2 on required tests for seismic resistance. The changes in this proposal are identical to the changes in those proposals except Seismic Design Category B is added to the charging language and the exception in both sections of this proposal.

Summarizing, AISC 341-10 applies to:

- 1. The seismic force-resisting systems in structures assigned to Seismic Design Category D, E or F; and
- 2. The seismic force-resisting systems designed for a response modification coefficient, *R*, greater than 3 in structures assigned to Seismic Design Category B or C.

This is only a summary because there are additional details affecting the standard's applicability, including nonbuilding structures and cantilever column systems, but these details are not affected by the proposed changes.

S160-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1705.11.1-S-BRAZIL.doc

S161–12 1705.11.3

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

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 <u>elements of</u> the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: Special inspections is are not required for coldformed steel light-frame shear walls, braces, and diaphragms, collectors (drag struts) and hold-downs including screw installation, bolting, anchoring, and other fastening to components of the seismic-force resisting system where either of the following apply:

- 1. The sheathing is gypsum board or fiberboard.
- 2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) o.c.

Reason: This proposal makes minor changes to this section. In the exception, word "braces" is deleted, since Items 1 and 2 of the exception discuss only sheathing used on shear walls and not braced walls. Revisions to the remainder of the section are to ensure consistency with the wood exception in Section 1705.10.1 and eliminate confusion.

S161-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.11.3-S-MANLEY.doc

S162–12 1705.11.5, 1705.11.6

Proponent: Philip Brazil, P.E., S.E., Senior Structural Engineer, Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1705.11.5 Architectural components. Periodic *special inspection* is required 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: Periodic special inspection is not required for the following:

- 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 (24.5N/m²) or less.
- 3. Special inspection is not required for Interior nonbearing walls weighing 15 psf (73.5 N/m²) or less.

1705.11.6 Mechanical and electrical components. <u>*Periodic special inspection* for of</u> mechanical and electrical components shall be as follows shall be required for the following:

- 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 Anchorage of other electrical equipment in structures assigned to Seismic Design Category E or F.
- Periodic special inspection is required during the Installation and anchorage of piping systems designed to carry 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 ductwork designed to carry hazardous materials in structures assigned to *Seismic Design Category* C, D, E or F.
- 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 the proposal is to delete superfluous language.

S162-12				
Public Hearing: Committe	e: AS	AM	D	
Assembly	: ASF	AMF	DF	
				1705.11.5-S-BRAZIL.doc

S163–12 1705.11.5

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1705.11.5 Architectural components. Periodic *special inspection* is required during the erection and fastening of exterior cladding, interior and exterior nonbearing walls, <u>suspended ceiling systems including</u> <u>their anchorage</u> 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/m²) or less.
- 3. Special inspection is not required for interior nonbearing walls weighing 15 psf (73.5 N/m²) or less.

Reason: This proposal restores the needed special inspection for suspended ceiling systems. The 2009 IBC identified "suspended ceiling systems and their anchorage" as components that needed to be included in the statement of special inspections for Seismic Design Category D, E or F, but did not list them in the section that invoked the actual inspection. Then, as part of the reorganization of Chapter 17 approved in the previous code change cycle, the requirement was deleted completely when the inspection requirements were combined with the requirements for inclusion in the statement of special inspections.

Suspended ceiling systems, when not properly anchored and braced, are well known to fail under strong shaking, resulting in debris that can block exits or otherwise impede egress from buildings.

S163-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	5				1705.11.5-S-KERR.doc

S164–12 1705.11.7

Proponent: Victor D. Azzi, PE, Consulting Structural Engineer, representing the Rack Manufacturers Institute (victorazzi@comcast.net)

Revise as follows:

1705.11.7 <u>Steel</u> storage racks. Periodic *special inspection* is required during the anchorage for anchor <u>bolt installation</u> of <u>steel</u> storage racks <u>that are designed in accordance with Section 2209, are</u> 8 feet (2438 mm) or greater in height and are in structures assigned to *Seismic Design Category* D, E or F.

Reason: Additional language ties section back to IBC Section 2209 on steel storage racks, which adopts the RMI MH 16.1 standard, and requires special inspection for the installation of anchor bolts.

S164-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.11.7-S-AZZI.doc

S165–12 1705.11.9 (NEW)

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Add new text as follows:

1705.11.9 Cold-formed steel special bolted moment frames. Periodic special inspection shall be provided for the installation of cold-formed steel special bolted moment frames in the *seismic force-resisting systems* of *structures* assigned to *Seismic Design Category* D, E or F.

Reason: The purpose for this proposal is to require special inspections for the installation of cold-formed steel special bolted moment frames, which are a new type of seismic force-resisting system and are listed in Table 12.2-1 of ASCE 7-10 in the category of moment-resisting frame systems (Item C.12). Their structural design is sufficiently complex to warrant inspection from a person with the expertise of a special inspector who is approved by the building official as having the competence necessary to inspect the installation of the joists. Refer to the definitions of "special inspection" and "special inspector" for further information. Examples of the complexity of the structural design that warrant special inspection of the installation are the beam-to-column connections and the anchorage to the foundation.

The standard for Seismic Design of Cold-formed Steel Structural Systems: Special Bolted Moment Frames with Supplement No. 1, AISI S110-07/S1-09, contain provisions for inspections but these are limited to quality control by the fabricator and inspections by qualified inspectors representing the owner. Refer to Section E of the standard.

S165-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1705.11.9 (NEW)-S-BRAZIL.doc

S166–12 1705.11, 1705.11.1, 1705.11.2, 1705.11.3, 1705.11.4, 1705.11.8

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1705.11 Special inspections for seismic resistance. *Special inspections* itemized for seismic resistance shall be required as specified in Sections 1705.11.1 through 1705.11.8, unless exempted by the exceptions of Section 1704.2, are required for the following:

- 1. The seismic force-resisting systems in structures assigned to Seismic Design Category C, D, E or F in accordance with Sections 1705.11.1 through 1705.11.3, as applicable.
- 2. Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F in accordance with Section 1705.11.4.
- 3. Architectural, mechanical and electrical components in accordance with Sections 1705.11.5 and 1705.11.6.
- 4. Storage racks in structures assigned to Seismic Design Category D, E or F in accordance with Section 1705.11.7.
- 5. Seismic isolation systems in accordance with Section 1705.11.8.

Exception: <u>The</u> special inspections <u>itemized</u> <u>specified</u> in Sections 1705.11.1 through 1705.11.8 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, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 35 feet (10 668 mm).
- The seismic force-resisting system of the structure consists of reinforced masonry or reinforced concrete; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 25 feet (7620 mm).
- 3. The structure is a detached one- or two-family dwelling not exceeding two *stories above grade plane* and does not have any of the following horizontal or vertical irregularities in accordance with Section 12.3 of ASCE 7:
 - 3.1. Torsional or extreme torsional irregularity.
 - 3.2. Nonparallel systems irregularity.
 - 3.3. Stiffness-soft story or stiffness-extreme soft story irregularity.
 - 3.4. Discontinuity in lateral strength-weak story irregularity.

1705.11.1 Structural steel. Special inspection for structural steel in the seismic force-resisting systems of <u>structures assigned to Seismic Design Category C, D, E or F</u> shall be <u>performed</u> in accordance with the quality assurance requirements of AISC 341.

Exception: Special inspections of structural steel <u>are not required</u> in <u>the seismic force-resisting</u> <u>systems of</u> 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.

1705.11.2 Structural wood. For the seismic force-resisting systems of structures assigned to Seismic Design Category C, D, E or F:

1. Continuous special inspection shall be required during field gluing operations of elements of the seismic force-resisting system and

2. Periodic special inspection shall be 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 shear walls, 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.).

1705.11.3 Cold-formed steel light-frame construction. For the seismic force-resisting systems of structures assigned to Seismic Design Category C, D, E or F, periodic special inspection shall be required:

- <u>1.</u> Periodic special inspection is required During welding operations of elements of the seismic force-resisting system and
- <u>2.</u> 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.

1705.11.4 Designated seismic systems. For structures assigned to seismic Design Category C, D, E or <u>F</u>, the special inspector shall examine designated seismic systems requiring seismic qualification in accordance with Section 1705.12.3 and verify that the *label*, anchorage or mounting conforms to the *certificate of compliance*.

1705.11.8 Seismic isolation systems. Periodic special inspection shall be provided for seismic isolation systems in seismically isolated structures assigned to *Seismic Design Category* B, C, D, E or F during the fabrication and installation of isolator units and energy dissipation devices.

Reason:

This proposal is the result of collaboration with the steel industry and makes changes to Section 1705.11 similar to the changes made to Section 1705.12 in a separate proposal.

As in Section 1705.12, determining applicable requirements in Section 1705.11 necessitates combining the governing item in Section 1705.11 with the corresponding subsection that follows. This exercise would be useful if it avoided duplication of the language in several of the subsections and this currently occurs for four sections (Sections 1705.11.1, 1705.11.2, 1705.11.3 and 1705.11.4). Because of the collaboration with the steel industry, however, it will now occur in only three sections and, with that, has outlived its usefulness. Any advantage gained is more than offset by the disadvantage in combining the applicable provisions, which can lead to errors by readers of the code. The proposal simplifies the requirements by transferring the language from the items in Section 1705.11 to the applicable subsections where comprehensive provisions are specified for each instance of required special inspections. Changes to Sections 1705.11.5, 1705.11.5.1, 1705.11.6 and 1705.11.7 are not made because none are needed in that the applicable provisions are already present.

Note that a separate proposal revises Section 1705.11.1 for consistency with the scope of AISC 341-10.

S166-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1705 11-S-BRA7U doc

S167–12 1705.12.1, 1705.3.1, 1705.3.2 (NEW)

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1705.12.1 <u>1705.2.2.1.3 Weldability of</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, 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 concrete reinforcement complying with <u>a standard other than</u> ASTM A 615 <u>706</u> is to be welded <u>and reports of</u> material properties verifying compliance with AWS D1.4 for weldability as specified in Section 3.5.2 of ACI 318 are not available, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.

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.

1705.3.2 Weldability. Weldability of reinforcement complying with a standard other than ASTM A 706, shall be verified in accordance with Section 3.5.2 of ACI 318.

Reason: The proposal retains the requirement in the last sentence of Section 1705.12.1 for chemical tests to determine weldability for concrete reinforcement complying with ASTM A 615 that is to be welded but:

- 1. Limits its scope to when reports of material properties verifying compliance with AWS D1.4 for weldability are not available for consistency with the referenced section of ACI 318-11 (Section 3.5.2);
- 2. Expands the scope to reinforcement designed to resist all load effects, not merely seismic load effects; and
- 3. Expands the scope to reinforcement complying with standards other that ASTM Å 706 for consistency with AWS D1.4 (Section 1.3.4 in the 1998 edition) and well as the referenced section of ACI 318-11.

S167-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	•				1705.12.1-S-BRAZIL.doc

S168–12 1705.12.3, Chapter 35 (NEW)

Proponent: Patrick A. McLaughlin, McLaughlin & Associates, representing Air-Conditioning, Heating & Refrigeration Institute (pmclaugma@aol.com)

Revise as follows:

1705.12.3 Seismic certification of nonstructural components. The *registered design professional* shall specify on the construction documents the requirements for certification by analysis, testing or experience data, or certification by compliance with AHRI 1270 (IP)/1271 (SI) for nonstructural components and designated seismic systems in accordance with Section 13.2 of ASCE 7, where such certification is required by Section 1705.12.

Add new standard Chapter 35 as follows:

AHRI

1270 (I-P)/1271 (SI)-2011 Requirements For Seismic Qualification of HVACR Equipment

Reason: The proposal recognizes compliance with AHRI 1270 (IP)/1271 (SI) Requirements for Seismic Qualification of HVACR Equipment, as a means to confirm seismic qualification of mechanical HVACR equipment. AHRI 1270/1271 was developed to show compliance with both ASCE 7 and IBC seismic requirements. It describes methods for equipment qualification and a process to determine equipment seismic capacity. The standard is available for review and use on AHRI's web site for free download at: http://www.ahrinet.org/search+standards.aspx.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S168-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705 12 3-S-MCI AUGHLIN doc

S169–12 1705.12, 1705.12.1, 1705.12.2, 1705.12.3, 1705.12.4

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1705.12 Testing and qualification for seismic resistance. The Testing and qualification for seismic resistance shall be required as specified in Sections 1705.12.1 through 1705.12.4, unless exempted from *special inspections* by the exceptions of Section 1704.2. are required as follows:

- 1. The seismic force-resisting systems in structures assigned to Seismic Design Category C, D, E or F shall meet the requirements of Sections 1705.12.1 and 1705.12.2, as applicable.
- 2. Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F and subject to the certification requirements of ASCE 7 Section 13.2.2 shall comply with Section 1705.12.3.
- Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F and where the requirements of ASCE 7 Section 13.2.1 are met by submittal of manufacturer's certification, in accordance with Item 2 therein, shall comply with Section 1705.12.3.
- 4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1705.12.4.

1705.12.1 Concrete reinforcement. In the seismic force-resisting systems of structures assigned to <u>Seismic Design Category B, C, D, E or F,</u> 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 <u>or</u> coupling beams connecting special structural walls, in structures assigned to <u>Seismic Design Category</u> B, C, D, E or F, 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 submitted to the *building official*. 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.

1705.12.2 Structural steel. Testing for of structural steel in the seismic force-resisting systems of structures assigned to Seismic Design Category C, D, E or F shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Testing for structural steel <u>is not required</u> in <u>the seismic force-resisting systems of</u> 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.

1705.12.3 Seismic certification of nonstructural components. For *structures* assigned to *Seismic Design Category* C, D, E or F, the *registered design professional* shall specify on the construction documents the 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 1705.12.

1705.12.4 Seismic isolation systems. Seismic isolation systems in seismically isolated structures assigned to *Seismic Design Category* B, C, D, E or F shall be tested in accordance with Section 17.8 of ASCE 7.

Reason: Determining applicable requirements in Section 1705.12 necessitates combining the governing item in Section 1705.12 with the corresponding subsection that follows. This exercise would be useful if it avoided duplication of the language in the items in several of the subsections but this only occurs once (Sections 1705.12.1 and 1705.12.2). Any advantage gained is more than offset by the disadvantage in combining the applicable provisions, which can lead to errors by readers of the code. Also, the applicability

of Section 1705.12.1 to Seismic Design Category B conflicts with corresponding Item 1 of Section 1705.12, which is limited to Seismic Design Categories C, D, E and F. The proposal simplifies the requirements by transferring the language from the items in Section 1705.12 to each of the subsections where comprehensive provisions are specified for each instance of required testing.

In Section 1705.12.1, the language requiring certified mill test reports "be provided for each shipment of such reinforcement" is replaced with "be submitted to the building official" because the details of providing the reports are not relevant to the provisions of the building code but submittal to the building official is relevant and critical to verifying that the reinforcement meets the applicable requirements in Section 21.1.5.2 of ACI 318-11.

The source document for some of the language in Section 1705.12 is the NEHRP Recommended Provisions for Seismic

Regulations for New Buildings and Other Structures (Sections 3.2 and 3.4 of FEMA 368, and Sections 2.2 and 2.4 of FEMA 450-1). Note that separate proposals:

- 1. Delete "qualification" in Section 1705.12 and place the provisions of Section 1705.12.3 into two subsections to provide effective charging language for the corresponding provisions in ASCE 7-10;
- 2. Transfer the requirements of Section 1705.12.1 to a new section on submittals to the building official; and
- 3. Revise Section 1705.12.2 for consistency with the scope of AISC 341-10.

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

S169-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1705.12-S-BRAZIL.doc

S170–12 1708.1, 1710.1

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:

SECTION 1708 TEST SAFE LOAD

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 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.

(Renumber subsequent sections)

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 the applicable referenced standards, the structural adequacy shall be predetermined based on the load test criteria established in this section.

1710.1 General. 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 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.

Reason: Section 1708 is entirely comprised of section 1708.1. Section 1708.1 references section 1710. Therefore, for clarity, the text of section 1708 should be relocated to section 1710. Furthermore, section 1708.1 essentially restates the text of section 1710.1, but in greater detail. Therefore, current section 1710.1 should be deleted after the substitution as it is redundant.

S170-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1708-S-HARMAN.doc

S171–12 1709.3.2

Proponent: Gary R. Searer, Wiss, Janey, Elstner Associates Inc., representing self

Revise as follows:

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 to the minimum of the specified factored design loads. For statically loaded components, the test load shall be left in place for a period of 24 hours. For components such as machine supports or fall arrest anchors that carry dynamic loads, the load shall be left in place for a period consistent with the component's actual function. 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.

Reason: This code change proposal does two things: 1) changes the required static test load from *precisely* "two times the unfactored design load" to a "minimum of the specified factored design loads", and 2) specifies how to test components that carry dynamic loads.

It is essentially not possible for the test load to be precisely two times any particular load, and the requirement to test to two times the unfactored load is arbitrary (i.e., why should you test to 2.0D+2.0L if the commonly accepted and statistically based load combination is 1.2D+1.6L?). By adding the phrase "a minimum of" to the requirement and by referencing factored loads, the intent of the provision is made clear -- that the test load should be *at least* the specified factored design load. Nationally recognized design standards such as the AISC Steel Specifications and ACI 318 have been developed with the intent to ensure that very few elements are unable to carry factored loads. To put it another way, if every element in a structure could carry factored loads, the structure's reliability would be consistent with the intent of such standards. In fact, the load testing provisions in each of the AISC and ACI standards make this clear by requiring proof test loads to essentially the full factored loads. This proposal is in-line with both AISC and ACI standards.

When an element is designed to carry short duration or dynamic loads, there is no need to sustain a proof test load for 24 hours.

S171-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1714.3.2-S-SEARER doc

S172–12 1710.5

Proponent: Julie Ruth, P.E. JRuth Code Consulting, representing American Architectural Manufacturers Association (AAMA) (julruth@aol.com)

Revise as follows:

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 1710.5.1 or 1710.5.2.

Exception: Structural wind load design pressures for window units smaller other than the size tested in accordance with Section 1710.5.1 or 1710.5.2 shall be permitted to be higher different than the design value of the tested unit provided such higher pressures are determined by accepted engineering analysis or validated by an additional test of the window unit to the alternate allowable design pressure in accordance with Section 1710.5.2. All components of the small alternate size unit shall be the same as the tested or labeled unit. Where such calculated design pressures are engineering analysis is used, they shall be validated by an additional test of the window unit having the highest allowable design pressure the glass shall comply with Section 2403.

Reason: The current exception limits the use of comparative analysis to window units smaller than the size originally tested for labeling purposes. If comparative analysis is used to provide a higher design pressure rating of the smaller unit, its resistance to air infiltration and water penetration at the correspondingly higher design pressure required by AAMA/WDMA/CSA 101/I.S.2/A440 must be verified by testing of the unit. These characteristics cannot be determined by calculation.

Comparative analysis is also appropriate to rate window units larger than the size originally tested for labeling purposes to lower design pressures. In this scenario, the corresponding design pressure used to verify resistance to air infiltration and water penetration would also be lower. Testing would not be required to verify this level of performance since a higher level has already been determined by testing of the same components in a smaller window unit.

This proposal revises this section as appropriate to permit the use of comparative analysis for larger as well as smaller window units than those tested for labeling. The last sentence of the section is also revised to specify that when engineering analysis is used, the glass in the fenestration product must also comply with Section 2403. Section 2403 establishes specific criteria for the deflection of the framing supporting the glass.

S172-11				
Public Hearing: Committee	ee: AS	AM	D	
Assembly	y: ASF	AMF	DF	
				1710.5 #1-S-RUTH

S173–12 1710.5

Proponent: Julie Ruth, P.E., JRuth Code Consulting, representing American Architectural Manufacturers (AAMA) and Joseph R. Hetzel, P.E., Thomas Associates, Inc., representing Door & Access Systems Manufacturers Association (DASMA) International

Revise as follows:

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 1710.5.1 or 1710.5.2. For the purposes of this section, the required design pressure shall be determined using the allowable stress design load combinations of Section 1605.3.

Exception: Structural wind load design pressures for window units smaller than the size tested in accordance with Section 1710.5.1 or 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.

Reason: The standards referenced in Section 1710.5 are based upon allowable stress design. This includes AAMA/WDMA/CSA 101/I.S.2/440, ASTM E330 and ANSI/DASMA 108. This proposal adds a sentence to the beginning of the section that clarifies that ASD loads are to be used in the application of this section.

S173-11					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	,				1710.5 #2-S-RUTH

S174–12 1710.5.1, 1710.5.2

Proponent: Julie Ruth, P.E., JRuth Code Consulting, representing American Architectural Manufacturers Association (AAMA) (julruth@aol.com)

Revise as follows:

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/DMA/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 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.

1710.5.2 Exterior windows and door assemblies not provided for in Section 1710.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330 <u>except that the</u> structural performance of garage doors and rolling 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.

Reason: At the present time exterior windows and sliding doors are required to be tested and labeled in accordance with AAMA/WDMA/CSA 101/I.S.2/A440. While this specification does require the fenestration product to be tested for resistance to structural load in accordance with ASTM E330, it also requires a number of other tests to be performed. These include resistance to air leakage and water penetration. Other tests such as forced entry resistance may be required depending upon the operator type of the product.

The integrity of the building envelope is dependent upon the performance of the fenestration in the envelope. This is as true for swinging doors as it is for sliding doors and windows. Previous attempts to extend the AAMA/WDMA/CSA 101/I.S.2/A440 labeling requirement to swinging doors were met with resistance. But to date, no acceptable alternative method of determining adequate performance of these products has been provided.

Products that are labeled in accordance with AAMA/WDMA/CSA 101/I.S.2/A440 are now available on the marketplace. Its time to tighten up this important component of the building envelope and require swinging doors to provide the same level of protection to the interior of the building that other components of the building envelope are required to provide.

Cost Impact: There will be no cost increase for products that are already being tested in compliance with Section 1710.5, as required by the IBC.

S174-12

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Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1710.5.1 #1-S-RUTH.doc

S175 – 12 1710.5.1

Proponent: Thomas S. Zaremba, Roetzel & Associates, representing Glazing Industry Code Committee (tzaremba@ralaw.com)

Revise as follows:

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 1710.5.2. Products <u>in Risk Category I and II</u> <u>buildings</u> tested and labeled as conforming to AAMA/WDMA/CSA101/I.S.2/A440 or comply with Section 1710.5.2. Products <u>in Risk Category I and II</u> <u>buildings</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 <u>provided one of the following is met:</u>

- 1. The required design pressure for the fenestration product does not exceed 60 psf or
- 2. All glass in the fenestration product is tempered or laminated.

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. The glass strength calculations in ASTM E1300 use this as a basis to establish a probability of glass breakage less than 8 in 1000. However, Section 1710.5.1 currently exempts certain residential and light commercial products from this requirement if they are labeled to the AAMA/WDMA/CSA 101/I.S.2/A440 standard. While this may be appropriate when these products are used in applications with lower design loads and/or lower risk building types, allowing this exception for *all* product types in *all* occupancies is far too broad. This proposal would correct this overbreadth by ensuring that products used in higher risk situations be firmly supported and meet the frame deflection limit to restore an appropriate safety margin consistent with ASTM E1300.

Specifically, this proposal would limit the exception to only risk category I and II buildings, and products used in higher risk category buildings must meet the Chapter 24 requirement for firmly supported glazing. This includes hospitals, public assembly areas with over 300 people, schools (often used as storm shelters), mission-critical facilities, and infrastructure. To provide flexibility, the proposal also maintains the exception for lower design pressures less than 60 psf, and where tempered or laminated glass is used as an alternative method to reduce the probability of glass breakage and/or potential risk of falling glass. This proposal is significantly different than other proposals discussed in previous cycles, which would have removed the exception for all buildings other than lowrise residential. This proposal takes a much more moderate approach to restore the appropriate safety margin consistent with Chapter 24 and ASTM E 1300 in higher risk situations, but leave the exception and flexibility for residential and light commercial products in lower risk applications.

S175-12					
Public Hearing: C	Committee:	AS	AM	D	
A	Assembly:	ASF	AMF	DF	
	-				1710.5.1-S-ZAREMBA.doc

S176–12 202, 1710.6, 2404.2, 2405.5, 2405.5.1, 2405.5.2

Proponent: Julie Ruth, P.E., JRuth Code Consulting, representing American Architectural Manufacturers Association (AAMA) (julruth@aol.com)

Revise as follows:

SECTION 202 DEFINITIONS

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

Revise as follows:

1710.6 Skylights and sloped glazing. Unit skylights and tubular daylighting devices (TDDs) shall comply with the requirements of Section 2405. All other Skylights and sloped glazing shall comply with the requirements of Chapter 24.

Revise as follows:

2404.2 Sloped glass. Glass sloped more than 15 degrees (0.26 rad) from vertical in skylights, sunrooms, sloped roofs and other exterior applications shall be designed to resist the most critical of the following combinations of loads.

Exception: <u>The design pressure rating of</u> unit skylights <u>and tubular daylighting devices</u> shall be <u>designed determined</u> in accordance with Section 2405.5.

(Portions of section not shown remain unchanged)

2405.5 Unit skylights <u>and tubular daylighting devices</u>. Unit skylights <u>and tubular devices</u> shall be tested and labeled as complying with AAMA/WDMA/CSA 101/I.S./A440. The *label* shall state the name of the manufacturer, the *approved* labeling agency, the product designation and the performance grade rating as specified in AAMA/WDMA/CSA 101/I.S.2/A440. If the product manufacturer has chosen to have the performance grade of the skylight rated separately for positive and negative design pressure, then the *label* shall state both performance grade ratings as specified in AAMA/WDMA/CSA 101/I.S.2/A440. If the skylight is not rated separately for positive and negative design pressure, then the *label* shall state both performance grade ratings as specified in AAMA/WDMA/CSA 101/I.S.2/A440 and the skylight shall comply with Section 2405.5.2. If the skylight is not rated separately for positive and negative pressure, then the performance grade rating shown on the *label* shall be the performance grade rating determined in accordance with AAMA/WDMA/CSA 101/I.S.2/A440 for both positive and negative design pressure and the skylight shall conform to Section 2405.5.1.

2405.5.1 Unit Skylights rated for the same performance grade for both positive and negative design pressure. The design of unit skylights shall be based on the following equation:

F_g ≤ PG **13)**

(Equation 24-

where:

 F_g = Maximum load on the skylight determined from Equations 24-2 through 24-4 in Section 2404.2. *PG* = Performance grade rating of the skylight. 2405.5.2 Unit Skylights rated for separate performance grades for positive and negative design pressure. The design of unit skylights rated for performance grade for both positive and negative design pressures shall be based on the following equations:

$$F_{gi} \le PG_{Po}$$
$$F_{go} \le PG_{Ne}$$

where:

 PG_{Pos} = Performance grade rating of the skylight under positive design pressure; PG_{Neg} = Performance grade rating of the skylight under negative design pressure; and F_{gi} and F_{go} are determined in accordance with the following:

For $W_o \ge D$,

where:

 W_o = Outward wind force, psf (kN/m²) as calculated in Section 1609.

D = The dead weight of the glazing, psf (kN/m²) as determined in Section 2404.2 for glass, or by the weight of the plastic, psf (kN/m²) for plastic glazing.

 F_{gi} = Maximum load on the skylight determined from Equations 24-3 and 24-4 in Section 2404.2. F_{go} = Maximum load on the skylight determined from Equation 24-2.

For $W_o < D$,

where:

 W_o = Is the outward wind force, psf (kN/m²) as calculated in Section 1609.

D = The dead weight of the glazing, psf (kN/m²) as determined in Section 2404.2 for glass, or by the weight of the plastic for plastic glazing.

 F_{gi} = Maximum load on the skylight determined from Equations 24-2 through 24-4 in Section 2404.2. F_{go} = 0.

Reason: The overall intent of this proposal is to clarify the requirements for tubular daylighting devices, within the context of skylights and sloped glazing in the IBC.

Tubular daylighting devices are a type of skylights, just as unit skylights are. The 2012 IBC contains a definition of TDDs that is consistent with the 2012 IRC. Part I of this proposal simply clarifies that, like unit skylights, TDDs are a type of skylight. This change would also bring consistency to the definition of skylights and sloped glazing between the IRC and IBC.

Section 1710.6 was intended to point the code user to the structural testing provisions of Chapter 24 for skylights and sloped glazing. As currently written, however, it may be misinterpreted as only requiring unit skylights and TDDs to comply with Section 2405.5, and not Chapter 24 in its entirety. This is not correct. The removal of a separate reference for unit skylights and TDDs will help to clarify this.

As currently written, the exception to Section 2404.2 may be interpreted as only requiring unit skylights and TDDs to meet Section 2405.5. This is not the intent of this exception. The proposed revision clarifies that only the design pressure rating of unit skylights and TDDs is to be determined in accordance with Section 2405.5. These products must still meet the other requirements of Chapter 24, and specifically of Section 2405.

Section 2405.5 is revised to clarify that both unit skylights and TDDs are to be tested and labeled in accordance with AAMA/WDMA/CSA 101/I.S.2/A440.

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

S176-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1710.6-S-RUTH.doc

(Equation 24-14) (Equation 24-15)

S177–12 1711.1, 2303.5, 2304.9.3, Chapter 35 (NEW)

Proponent: Brad Douglas, P.E., American Wood Council

Delete without substitution:

1711.1 Joist hangers. Testing of joist hangers shall be in accordance with Sections 1711.1.1 through 1711.1.3, as applicable.

1711.1.1 General. 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).

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 r espect to the header of ⁴/₈ inch (3.2 mm).
- 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.

1711.1.2.1 Design value modifications for joist hangers. Allowable design values for joist hangers that are determined by Item 4 or 5 in Section 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 1711.1.2. Allowable design values determined by Item 1, 2 or 3 in Section 1711.1.2.

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 $\frac{1}{8}$ inch (3.2 mm).

(Renumber subsequent sections)

Revise as follows:

2303.5 Test standard for joist hangers. For the required test standards for joist hangers see Section 1711.1 Joist hangers shall conform to requirements of ASTM D 7147.

Revise as follows:

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 1711.1 ASTM D 7147.

Add new standard to Chapter 35 as follows:

ASTM

D 7147-05, Specification for Testing and Establishing Allowable Loads of Joist Hangers

Reason: The 2009 IBC updated the reference to ASTM D 1761 from a prior edition to a 2006 edition for the testing of joist hangers. However, ASTM D1761-06 no longer contains provisions for testing of joist hangers. These provisions were moved to and revised in ASTM D7147. The revisions included sampling and evaluation criteria (currently included in IBC section 1711.1) as well as further refinements regarding quality of test materials, adjustments for variation in test materials, and limits on design values with materials other than those tested. In addition, since ASTM D7147 is specific to joist hangers used with wood and contains provisions that go beyond testing, it is more appropriate to reference it in Chapter 23 rather than Chapter 17.

S177-12					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
	•				1711.1-S-COATS.doc

S178–12 1711.1.1, 1711.1.2, 1711.1.2.1, 1711.1.3, Chapter 35 (NEW)

Proponent: Randall Shackelford, P.E., Simpson Strong-Tie Company, Inc., (rshackelford@strongtie.com)

Revise as follows:

1711.1 Joist hangers. Testing of joist hangers shall be in accordance with Sections 1711.1.1 through 1711.1.3 <u>1711.1.2</u>, as applicable.

1711.1.1 General <u>Allowable stress design</u>. The <u>allowable</u> vertical load-bearing capacity, torsional moment capacity and deflection characteristics of joist hangers shall be determined in accordance with ASTM D 1761 <u>7147</u> using lumber having a specific gravity of 0.49 or greater, but not greater than 0.55, as determined in accordance with AF&PA NDS for the joist and headers.

Exception: The joist length shall not be required to exceed 24 inches (610 mm).

1711.1.2 Vertical load capacity for joist hangers. The vertical load-bearing 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-bearing 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).
- 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.

1711.1.2.1 Design value modifications for joist hangers. Allowable design values for joist hangers that are determined by Item 4 or 5 in Section 1711.1.2 shall be permitted to be modified by the appropriate load duration factors as specified in AF&PA NDS but shall not exceed the direct loads as determined by Item 1, 2 or 3 in Section 1711.1.2. Allowable design values determined by Item 1, 2 or 3 in Section 1711.1.2. Allowable design values determined by Item 1, 2 or 3 in Section 1711.1.2.

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).

(Renumber subsequent sections)

1711.1.2 <u>LRFD or Strength Design.</u> The resistance of joist hangers for LRFD or strength design shall be determined using an approved method.

Add new standard to Chapter 35 as follows:

ASTM

D 7147—11 Standard Specification for Testing and Establishing Allowable Loads of Joist Hangers

Reason: In the 2012 IBC, this section requires that the capacity of joist hangers be determined using ASTM D1761. The 2006 edition of that standard is referenced in Chapter 35. The problem is that the 2006 edition no longer contains requirements for testing of joist hangers. All joist hanger testing requirements were removed, and a new standard, ASTM D7147 was written for testing of joist hangers. The newest edition of the ASTM D7147 standard is the 2011 edition. That is being suggested as the most applicable standard for determining allowable loads for joist hangers.

The specific methods for evaluating the test data can be deleted because all those requirements are now contained in ASTM D7147, whereas they were not present in D1761.

This standard is only applicable for determining allowable stress loads. It does not give guidance for determining resistances for LRFD. There are documents that give such guidance, such as the American Wood Council *Load and Resistance Factor Design* (*LRFD*) *Manual for Engineered Wood Construction*. There is precedence for having sections for both ASD and LRFD. Sections 1908 and 1909 contain requirements for design of anchorage to concrete using allowable stress and strength design, respectively.

Cost Impact: ASTM D7147 does contain some additional requirements for testing and recording the hanger steel strength, wood specific gravity, and bending yield strength of fasteners, which could increase the cost of testing over ASTM D1761-2000, but that standard is no longer referenced anyway so that is not really an option.

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S178-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1711 1-S-SHACKELEORD doc

S179–12 1803.5.12

Proponent: Mark Petersen, The Shaw Group, Inc., representing Deep Foundations Institute, Seismic and Lateral Load Committee

Revise as follows:

1803.5.12 Seismic Design Categories D through F. For structures assigned to *Seismic Design Category* D, E or F, 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 supporting more than 6 feet (1.83 m) of backfill height due to design earthquake ground motions.
- 2. The potential for liquefaction and , soil strength loss, post-liquefaction reconsolidation settlement, and lateral spreading deformation shall be evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the maximum considered earthquake ground motions and the design earthquake ground motions. The deformations associated with the maximum considered earthquake motions should be used to evaluate collapse of the structure. The deformations associated with the design earthquake ground acceleration shall be determined based on:
 - 2.1 A site-specific study in accordance with Section 21.5 of ASCE 7 for the maximum considered earthquake ground motions, use two-thirds of the site-specific maximum considered earthquake peak ground acceleration;; or
 - 2.2 In accordance with Section 11.8.3 of ASCE 7 for the maximum considered earthquake motions and in accordance with Section 11.4.5 of ASCE 7 for the design earthquake motions.
- 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:
 - 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: buildings designed to ASCE 7-05 seismic provisions are expected to resist collapse during an MCE event, and therefore the possible effects of liquefaction on the stability of the foundation and structure require evaluation at this level." (C.B. Crouse, M.S. Power, and D.G. Anderson, "New Liquefaction Requirements and Associated Peak Ground Acceleration Maps", 2010 Structures Congress, ASCE.

The proposed change is consistent with the code philosophy. That is, collapse is evaluated based on liquefaction-induced deformations resulting from the maximum considered earthquake motions and design is based on the liquefaction-induced deformations resulting from the design earthquake ground motions.

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

51/9-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1803.5.12-S-PETERSEN.doc

0470 40

S180–12 1803.5.6

Proponent: Theodore L Droessler, Clark County Nevada, representing Clark County Development Services, and R. David Charles, P.E., representing ASFE, the Geoprofessional Business Association.

Revise as follows:

1803.5.6 Rock strata. Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the presence of rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 10 feet (3048 mm) below the level of the foundations to provide assurance of the soundness of the foundation bed assess the competency of the rock and its load-bearing capacity in terms of the rock strength and the presence, orientation, and condition of discontinuities, weathering profiles and other similar profiles of the sampled rock as they apply at a particular site.

Reason: The proposed modification is intended to make the wording of the Code addressing the evaluation of rock materials for foundation support more consistent with current geotechnical engineering practice.

The current wording suggests that it is possible to provide "assurance of the soundness of rock" during the geotechnical evaluation phase. Unfortunately, experience has shown that even at sites where rigorous evaluation of rock conditions is undertaken, it is often determined during construction that rock conditions between the locations sampled can vary significantly. Many times the actual rock conditions at foundation locations are exposed or better defined (through excavation, proof-drilling, etc.) during construction, and interpretations of the conditions exposed during the construction process are necessary to complete the design of the foundation element.

The proposed modifications to Section 1803.5.6 express the characteristics necessary to assess the rock strata and estimate a load-bearing capacity based on observations and testing.

S180-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				1803 5 6-S-CHARLES-DROESSLER doc

S181–12 1803.5.7, 1804.1, 1804.2 (NEW), 1804.2.1 (NEW),

Proponent: Edwin Huston, National Council of Structural Engineers Associations- Code Advisory Committee - General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

1803.5.7 Excavation near foundations. Where excavation will remove lateral support from any foundation, an investigation shall be conducted to assess the potential consequences and address mitigation measures a Registered Design Professional shall prepare a report summarizing the condition of the structure as determined from examination of the structure, the review of available design documents and if necessary, the excavation of test pits. The Registered Design Professional shall determine the requirements for underpinning and protection and prepare site-specific plans, details, and sequence of work for submission. Such support may be provided by underpinning, sheeting, and bracing, or by other means acceptable to the building official.

1804.1 Excavation near foundations. Excavation for any purpose shall not remove lateral support from any foundation <u>or adjacent foundation</u> without first underpinning or protecting the foundation against settlement or lateral translation.

1804.2 Underpinning. Where the protection and/or support of adjacent structures is required, the underpinning system shall be designed and installed in accordance with provisions of this chapter and Chapter 33.

1804.2.1 Underpinning and bracing installation. Where underpinning is used for the support of adjacent structures, the piers, wall piles or footings shall be installed in such manner so as to prevent the lateral or vertical displacement of the adjacent structure, to prevent deterioration of the foundations or other effects that would disrupt the adjacent structure. The sequence of installation shall be identified in the design.

Reason: At present, excavation of foundations is not specifically addressed in relation to adjacent structures. Section 3307, Protection of Adjacent Property, states: "Adjoining public and private property shall be protected from damage during construction, remodeling and demolition work. Protection shall be provided for footings, foundations, party walls, chimneys, skylights and roofs."

The code currently has minimal and vague requirements of the due diligence required for investigation for excavation near a neighboring structure. Failures to perform proper pre-construction investigations and monitoring procedures have led to failures in construction during underpinning and excavation operations. Improper excavations result nationally in doors and windows that don't open, increasing through cracking of bearing walls and support members, failures of structural members and to collapse and fatalities.

Specific guidelines are provided to identify responsibilities and basic requirements for providing safe and successful underpinning and excavations.

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

S181-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1803.5.7-S-HUSTON.doc

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S182–12 1806.2

Proponent: Theodore L Droessler, Clark County Nevada, representing Clark County Development Services, R. David Charles, P.E., representing ASFE, the Geoprofessional Business Association.

Revise as follows:

1806.2 Presumptive load-bearing values. The load-bearing values used in design for supporting soils near the surface shall not exceed the values specified in Table 1806.2 unless data to substantiate the use of higher values are submitted and *approved*. Where the *building official* has reason to doubt the classification, strength or compressibility of the soil, <u>or in cases where settlement and/or differential settlement of the foundations is a concern for the serviceability or stability of the structure</u>, the requirements of Section 1803.5.2 shall be satisfied. Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions. Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted.

Exception: A presumptive load-bearing capacity shall be permitted to be used where the *building official* deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight or temporary structures.

Reason: The proposed modification is intended to clarify that the use of "presumptive load-bearing values" should be limited to certain situations, for which judgment needs to be exercised. The term compressibility is replaced with serviceability to reflect similar language used in Chapter 16 for deflection of structural members.

While there is history of using "presumptive load-bearing capacities" for lightly loaded and temporary structures in some jurisdictions, the use of such "presumptive" values to design foundations for structures, without the benefit of subsurface evaluation and geotechnical analysis, should be used with caution. In the event that total or differential settlement of a structure is of concern, a geotechnical evaluation (consistent with Section 1803.5.2) should be performed by a qualified geotechnical professional.

S182-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1806.2-S-CHARLES-DROESSLER.doc

S183–12 1808.2

Proponent: Theodore L Droessler, Clark County Nevada, representing Clark County Development Services, R. David Charles, P.E., representing ASFE, the Geoprofessional Business Association.

Revise as follows:

1808.2 Design for capacity and settlement. Foundations shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that <u>the estimated total and</u> differential settlement is <u>minimized</u> shall not cause harmful distortion or instability in the structure, nor cause any element to be loaded beyond its limit states or allowable capacity.</u> Foundations in areas with expansive soils shall be designed in accordance with the provisions of Section 1808.6.

Reason: The proposed modification is intended to make the wording of the Code addressing the design of shallow foundation systems consistent with practice of the design of shallow foundation systems. The proposed change also makes the wording of the Code for shallow foundations consistent with the existing wording of the Code for deep foundation systems.

The existing wording of the Code states that shallow foundation systems are to be designed such that "differential settlement is minimized." In many cases, several shallow foundation options are identified as feasible during the geotechnical evaluation for a given structure, with different degrees of potential differential settlement under different options. The final selection of the appropriate foundation option for a specific project should be made by the design team, (including the structural engineer, the geotechnical engineer and the Owner), based on consideration of cost and the implications of the estimated total and differential settlement. The option chosen for a specific project may not necessarily be the one for which "differential settlement is minimized." Therefore, rewording this section of the Code to recognize current practice is appropriate.

S183-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	•				1808.2-S-CHARLES-DROESSLER.doc

S184-12 1808.3.2 (NEW)

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Add new text as follows:

1808.3.2 Surcharge. No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or the surcharge. Existing footings or foundations which will be affected by any excavation shall be underpinned or otherwise protected against settlement and shall be protected against lateral movement.

Reason: The code does not comment on permanent loads surcharging a neighboring structure. It references surcharge loads only in reference to construction loading in Chapter 33.

S184-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	1808.3.2 (NEW)-S-HUSTON.doc

S185–12 1810.2.5

Proponent: Lori A. Simpson, P.E., GE, Treadwell & Roll, a Langan Company, representing Deep Foundations Institute

Revise as follows:

1810.2.5 Group effects. The analysis shall include group effects on lateral behavior where the center-tocenter spacing of deep foundation elements in the direction of lateral force is less than eight times the least horizontal dimension of an element. The analysis shall include group effects on axial behavior where the center-to-center spacing of deep foundation elements is less than three times the least horizontal dimension of an element. <u>Group effects shall be evaluated using an approved method of analysis; the</u> analysis for uplift of grouped elements with center-to-center spacing less than three times the least horizontal dimension shall be evaluated in accordance with Section 1810.3.3.1.6.

Reason: To make the evaluation of group effects on uplift more clear that it needs to be performed where spacing is less than three times the least horizontal dimension. While this section may seem clear without the change, Section 1810.3.3.1.6 makes it unclear what spacing necessitates evaluation of group effects for uplift. Cross referencing the other section, plus changes made to Section 1810.3.3.1.6 (see another Code Change Proposal), will clarify this issue.

S185-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				1810.2.5-S-SIMPSON.doc

S186–12 1810.3.1, 1810.3.1.1, 1810.3.1.7 (NEW), 1810.3.1.8 (NEW), 1810.3.2.4, 1810.3.2.6, Table 1810.3.2.6, 1810.3.2.7, 1810.3.2.8, 1905.1.11

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1810.3.1 Design conditions. Design of deep foundations shall include the design conditions specified in Sections 1810.3.1.1 through <u>1810.3.1.6</u> <u>1810.3.1.8</u>, as applicable. <u>As an alternate, the allowable loads</u> can be evaluated using the allowable stresses in Section 1810.3.2.6.

1810.3.1.1 Design methods for concrete elements. Where concrete deep foundations are laterally supported in accordance with Section 1810.2.1 for the entire height and applied forces cause bending moments no greater than those resulting from accidental eccentricities, structural design of the element using the load combinations of Section 1605.3 and the allowable stresses specified in this chapter shall be permitted. Otherwise, the structural design of concrete deep foundation elements shall use the load combinations of Section 1605.2 and *approved* strength design methods. the member design shall be in accordance with ACI 318 subject to the other requirements in this chapter.

- 1. Prestressed precast concrete piles may use other *approved* strength design methods.
- Permanent casing can be used in place of confinement reinforcement for cast-in-place concrete elements not greater than 16 inches in diameter if strength and stiffness of the casing is equal or greater than that of the specified confinement reinforcing.

1810.3.1.7 Timber piles. Timber deep foundation elements shall be designed as piles or poles in accordance with AF&PA NDS.

1810.3.1.8 Steel H Piles. Steel H-Piles in Seismic Design Categories D, E, or F shall comply with the Provisions of AISC 360 and AISC 341. In other cases the use of AISC 360 is allowed.

1810.3.2.4 Timber. Timber deep foundation elements shall be designed as piles or poles in accordance with AF&PA NDS. Round timber elements shall conform to ASTM D 25. Sawn timber elements shall conform to DOC PS-20.

1810.3.2.6 Allowable stresses. The allowable stresses for materials used in deep foundation elements shall not exceed those specified in Table 1810.3.2.6. Allowable stresses in Table 1810.2.6 can be used only where deep foundations are laterally supported in accordance with Section 1810.2.1 for the entire height and applied forces cause bending moments no greater than those resulting from accidental eccentricities, structural design of the element using the load combinations of Section 1605.3. Use of allowable stresses is subject to the constraints of Section 1810.3.1.

ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS				
	MAXIMUM ALLOWABLE			
MATERIAL TYPE AND CONDITION	STRESS ^ª			
1.Concrete or grout in compression ^b				
Cast in place with a permanent casing in accordance with Section				
1810.3.2.7	0 4 f '			
Cast-in-place in a pipe, tube, other permanent casing or rock	0.33 <i>f</i> ′			
Cast-in-place without a permanent casing	0.35			
Precast nonprestressed	$0.33t'_{-}$			
Precast prestressed	$0.33f'_{\rm c}$ - $0.27f_{\rm pc}$			

TABLE 1810.3.2.6

MATERIAL TYPE AND CONDITION	MAXIMUM ALLOWABLE STRESS ^a
2.Nonprestressed reinforcement in compression	0.4 <i>f</i> _v ≤ 30,000 psi
 Structural steel in compression Cores within concrete-filled pipes or tubes Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8 Pipes or tubes for micropiles Other pipes Pipes, tubes or H-piles 	0.5 $F_y \le 32,000$ psi 0.5 $F_y \le 32,000$ psi 0.4 $F_y \le 32,000$ psi 0.35 $F_y \le 16,000$ psi 0.6 $F_y \le 0.5$ F
4.Nonprestressed reinforcement in tension Within micropiles Other conditions	$0.6f_{y}$ $0.5f_{y} \le 24,000 \text{ psi}$
 Structural steel in tension Pipes, tubes or H-piles, where justified in accordance with Section <u>1810.3.2.8</u> <u>Other pipes</u>, tubes or H-piles Helical piles 	0.5 F _y ≤ 32,000 psi 0.35 F _y ≤ 16,000 psi 0.6 F _y ≤ 0.5 F _u
6.Timber	In accordance with the AF&PA NDS

For SI: 1 pound per square inch = 6.895 kPa.

a. f'_c is the specified compressive strength of the concrete or grout; f_{pc} is the compressive stress on the gross concrete section due to effective prestress forces only; f_y is the specified yield strength of reinforcement; F_y is the specified minimum yield stress of structural steel; F_u is the specified minimum tensile stress of structural steel.

b. The stresses specified apply to the gross cross-sectional area within the concrete surface. Where a temporary or permanent casing is used, the inside face of the casing shall be considered the concrete surface.

1810.3.2.7 Increased allowable compressive stress for cased cast-in-place elements. The allowable compressive stress in the concrete shall be permitted to be increased as specified in Table 1810.3.2.6 for those portions of permanently cased cast-in-place elements that satisfy all of the following conditions:

- 1. The design shall not use the casing to resist any portion of the axial load imposed.
- 2. The casing shall have a sealed tip and be mandrel driven.
- 3. The thickness of the casing shall not be less than manufacturer's standard gage No.14 (0.068 inch) (1.75 mm).
- 4. The casing shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.
- 5. The ratio of steel yield strength (Fy) to specified compressive strength (f'c) shall not be less than six.
- 6. The nominal diameter of the element shall not be greater than 16 inches (406 mm).

1810.3.2.7 Cased cast-in-place elements. Permanently cased ast-in-place concrete elements shall comply with the following.

- 1. The design shall not use the casing to resist any portion of the imposed axial load.
- 2. For mandrel driven piles
 - a. The casing shall have a sealed tip and be mandrel driven.
 - b. The thickness of the casing shall not be less than No. 14 (0.068 inch) (1.75 mm) gage.
 - c. The nominal diameter of the element shall not be greater than 16 inches (406 mm).
 - d. The ratio of steel yield strength (F_y) to specified compressive strength (f'_c) shall not be less than six.
- 3. The casing shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.

1810.3.2.8 Justification of higher allowable stresses. Use of allowable stresses greater than those specified in Section 1810.3.2.6 shall be permitted where supporting data justifying such higher stresses is filed with the *building official*. Such substantiating data shall include:

- 1. A geotechnical investigation in accordance with Section 1803; and
- Load tests in accordance with Section 1810.3.3.1.2, regardless of the load supported by the element.

The design and installation of the deep foundation elements shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and deep foundations who shall submit a report to the *building official* stating that the elements as installed satisfy the design criteria.

Add new text as follows:

1905.1.11 ACI 318, Section 1.1.6. Modify ACI 318 Section 1.1.6 to read as follows:

<u>1.1.6 – This Code does not govern design and installation of portions of prestressed concrete piles, in</u> ground except for structures assigned to Seismic Design Categories D, E. and F. See 21.12.4 for requirements for concrete piles, drilled piers, and caissons in structures assigned to Seismic Design Categories D, E, and F.

Reason: The current code references to increased allowable stresses do not provide guidance on what the increased allowable stresses should be, with a few exceptions. The proposal resolves this ambiguity by making it clear that when higher capacities are desired or the constraints on the use of allowable stresses are not satisfied that strength design methods be used. Thus we no longer need to define values of the higher allowable stresses. This is supported by Section 1810.3.1.1, and other sections which endorse the use of strength design methods.

The existing provisions defining when increased allowable stresses could be used create additional problems since the criteria were not related to additional structural capacity. The load test provisions in Sections 1810.2.8 and 1810.3.3.1.2 address geotechnical failure modes and are not appropriate to evaluate structure failure modes and thus are inappropriate to justify increased member structural capacity. The load tests listed in this chapter are still useful in evaluating geotechnical failure modes but should not be used to define structural capacities.

A structural failure mode below the calculated capacity of the structural element would suggest either significant calculation error, damage during installation, or material not meeting the specified requirements. Load testing isolated piles is not the optimum way to identify these potential problems. Project controls are needed to identify and control these problems since if they occur the default allowable stresses may not be adequate.

The changes proposed here impact only the structural capacity of the deep foundation elements and not the geotechnical failure modes.

In several locations in Section 1810.3.1 instead of referring to "approved strength design methods" specific standards are listed. In the case of Timber piles and steel H piles the standards referenced are existing requirements. In the case of concrete the consensus of the foundation literature endorsed the use of ACI 318. The reference to ACI 318 makes it clear that the standard supplements but does not replace the existing provisions already in this Chapter. Listing specific design standards reduces any ambiguity as to what is acceptable.

1810.3.1

Range of sections listed is revised to reflect the sub sections added by this change.

The last sentence is added to make it clear that Section 1810.3.2.6 is subservient to Section 1810.3.1 1810.3.1.1

Specific reference to ACI is needed both to make specific the strength design provisions mentioned in the current version and because the special inspection provisions in Chapter 17 make reference to provisions of ACI 318 that would not be mandated for these deep foundation elements.

ACI 543 and ACI 336 endorse this approach. The appropriateness of adopting ACI 318 is further supported by that fact that ACI 318 already has design provisions for deep foundation concrete elements.

Item 1 is added to reflect the fact that prestressed piles are not addressed in ACI 318. Refer to the new Section 1905.1.11. Item 2 addresses the special case of small diameter piles cast-in-place concrete piles with permanent bracing.

1810.3.1.7

The language for timber piles was moved, unchanged, from Section 1810.3.2.4 since Section 1810.3.2 deals with materials and not design methodologies.

1810.3.1.8

Because this chapter defines the design standards for deep foundation elements it is necessary to specifically invoke the requirements in AISC 341 that are applicable in high seismic conditions.

1810.3.2.4

The design provisions are moved to Section 1810.3.1 which more appropriately deals with design conditions. **1810.3.2.6**

The limitations on the use of concrete deep foundation elements in Section 18810.3.1.2 are made applicable to all deep foundation elements. When buckling is a concern or we are dealing with combined bending and axial forces a more robust methodology is appropriate.

The added language also makes it clear that the use of allowable stresses is subservient to the constraints of Section 1810.3.1 that defines constraints on design of deep foundation elements.

TABLE 1810.3.2.6

The line in item 1 referencing Section 1810.3.2.7 is deleted. Rather than defining increased allowable stress the engineer has the option of using the strength design provisions in Section 1810.3.1.1. Recognizing that applying strength design methods for the piles addressed in 1810.3.2.7 may be difficult because of difficulties with installing confinement reinforcement, Item 1 has been added to Section 1810.3.1.1 allowing the casing to be used as confinement reinforcement thus facilitating the application of strength design methods for these piles.

The lines in items 3 and 5 of the table referencing Section 1810.3.2.8 are deleted because the criteria is not relevant for evaluating structural capacity as opposed to geotechnical capacity. In the absence of the deleted item the designer would be directed to the new Section 1810.3.1.8 which references AISC 360 and AISC 341.

The provision for timber in item 6 of the table are deleted because they are already addressed in Section 1810.3.2.4 **1810.3.2.7**

This section has been reformatted since they no longer serve as prerequisites for higher allowable stresses. The existing sub-section 2 would require the use of a mandrel when installing all permanent casing. The changes limit this requirement to the class of piles normally installed with mandrels.

1810.3.2.8

Load tests described in paragraph 1810.3.3.1.2 address the geotechnical capacity and not the structural capacity of deep foundation elements. Thus the provisions of this section should not be used to access the structural capacity of deep foundation elements.

The inspection requirements in the last paragraph are not necessary since Sections 1705.7 and 1705.8 already define the necessary special inspections.

1905.1.11:

Without this change to the scope of ACI 318 the IBC does not provide any technical criteria for the design of the structural aspects of concrete piles, drilled piers, and caissons. ACI 336 "Design and Construction of Drilled Piers", and ACI 543 "Recommendations for Design, Manufacture of Concrete Piles", endorse the use of ACI 318 to design these deep foundation elements. The change reflects current practice.

The change in scope of ACI 318 does not include adding prestressed concrete piles because of limited time to resolve technical concerns.

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

S186-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1810.3.1-S-KERR.doc
S187–12 1810.3.2.3, Table 1810.3.2.6, 1810.3.5.3.1, 1810.3.5.3.2 (NEW), 1810.3.5.3.3 (NEW), Chapter 35 (NEW)

Proponent: Bonnie Manley, American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

1810.3.2.3 Structural steel <u>Steel</u>. Structural steel <u>H</u>-piles <u>and structural steel sheet piling shall conform</u> <u>to the material requirements in ASTM A6.</u> Steel pipe <u>piles shall conform to the material requirements in ASTM A 252.</u> and Fully welded steel piles <u>shall be</u> fabricated from plates <u>shall that</u> conform to <u>the</u> <u>material requirements in</u> ASTM A 36, ASTM A 252, ASTM A 283, ASTM A 572, ASTM A 588 <u>or</u> ASTM A 690, <u>ASTM A 913 or ASTM A 992</u>.

TABLE 1810.3.2.6 ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS MATERIAL TYPE AND CONDITION

	STRESS ^a
3. Structural steel Steel in compression	
Cores within concrete-filled pipes or tubes	0.5 <i>F_y</i> ≤ 32,000 psi
Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8	0.5 <i>F_y</i> ≤ 32,000 psi
Pipes or tubes for micropiles	0.4 <i>F_y</i> ≤ 32,000 psi
Other pipes, tubes or H-piles	0.35 <i>F_y</i> ≤ 16,000 psi
Helical piles	$0.6 F_y \le 0.5 F_u$
5. Structural steel Steel in tension	
Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8	0.5 <i>F_y</i> ≤ 32,000 psi
Other pipes, tubes or H-piles	0.35 <i>F_y</i> ≤ 16,000 psi
Helical piles	$0.6 F_y \le 0.5 F_u$

a. f'_{c} is the specified compressive strength of the concrete or grout; $f_{\rho c}$ is the compressive stress on the gross concrete section due to effective prestress forces only; f_{y} is the specified yield strength of reinforcement; F_{y} is the specified minimum yield stress of structural steel; F_{u} is the specified minimum tensile stress of structural steel.

(Portions of Table not shown remain unchanged)

1810.3.5.3.1 <u>Structural steel</u> H-piles. Sections of <u>structural steel</u> H-piles shall comply with the <u>requirements for HP shapes in ASTM A6, or the</u> following:

- 1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
- 2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).
- 3. Flanges and web shall have a minimum nominal thickness of 3/8 inch (9.5 mm).

1810.3.5.3.2 Fully welded steel piles fabricated from plates. Sections of fully welded steel piles fabricated from plates shall comply with the following:

- 1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
- 2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).
- 3. Flanges and web shall have a minimum nominal thickness of 3/8 inch (9.5 mm).

1810.3.5.3.3 Structural steel sheet piling. Individual sections of structural steel sheet piling shall conform to the profile indicated by the manufacturer, and shall conform to the general requirements specified by ASTM A 6.

Add new standard to Chapter 35 as follows:

ASTM

<u>A6/A6M-11 Standard Specifications for General Requirements for Rolled Structural Steel Bars, Plates,</u> Shapes, and Sheet Piling

Reason: Section 1810.3.2.3 of this proposal improves the clarity of Section 1810.3.2.3 as it applies to steel foundation elements. First, it coordinates the title with the language that follows. Structural steel is defined in Section 202 and steel pipe piles and fully welded steel piles do not necessarily fall into that classification. Second, the section assigns the appropriate ASTM references to the applicable foundation elements. ASTM A 252 applies only to steel pipe piles. ASTM A 913 and ASTM A 992 both apply to structural shapes and not plates, thus they are not appropriate for fully welded steel piles fabricated from plates. Finally, ASTM A 6 has been added as the appropriate reference for the material requirements for H-piles and another common steel foundation system -- sheet piling. Since ASTM A 913, or ASTM A 992 – duplicate reference of those standards is not necessary for H-piles and sheet piling.

Table 1810.3.2.6 of this proposal coordinates the title change in Section 1810.3.2.3 with requirements in Table 1810.3.2.6. Structural steel is defined in Section 202 and steel pipe and fully welded steel piles do not necessarily fall into that classification, but the intent is to apply the allowable stress limits to those sections as well. Consequently, the term "structural" has been deleted.

1810.3.5.3.1, 1810.3.5.3.2 of this proposal clarifies the Section 1810.3.5.3 by separating the requirements for structural steel H-piles from fully welded steel piles fabricated from plates and adding a new section on structural steel sheet piling. Within the section on structural steel H-piles, Section 1810.3.5.3.1, reference is made to ASTM A 6 for HP shapes, which automatically satisfy the three specified dimensional limitations. Additionally, allowance is made for other structural steel H-pile shapes, if they meet the three dimensional limitations. Clarifying language is added as a new Section 1810.3.5.3.2 permitting the three dimensional limitations. Clarifying language is added as a new Section 1810.3.5.3.2 permitting the three dimensional limitations to be applied to fully welded steel piles fabricated from plates. Finally, Section 1810.3.5.3.3 is introduced for structural steel sheet piling requiring that the profiles conform to manufacturer's specifications and the general requirements in ASTM A 6. **Chapter 35** of this proposal adopts the latest edition of ASTM A 6 into Chapter 35.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S187-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1810.3.2.3-S-MANLEY.doc

S188–12 1810.3.2.5

Proponent: Mark Gilligan, P.E., S.E., representing self (mark@gilligan.name)

Revise as follows:

C100 12

1810.3.2.5 Protection of materials. Where boring records or site conditions indicate possible deleterious action on the materials used in deep foundation elements because of soil constituents, changing water levels or other factors, the elements shall be adequately protected by materials, methods or processes *approved* by the *building official*. Protective materials shall be applied to the elements so as not to be rendered ineffective by installation. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

The following protections will be considered to comply with this requirement:

- 1. Concrete in compliance with Chapters 4 and 7 of ACI 318.
- 2. Wood piles treated in accordance with Section 1810.3.2.4.1.

Reason: The added language makes it clear that conformance with the listed criteria will satisfy the existing language. The current provision does not provide any objective basis for the building official or engineer to evaluate the protection needed. The proposed criteria are already requirements in the code.

3100-12					
Public Hearing:	Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF		
	•				1810.3.2.5-S-GILLIGAN.doc

S189–12 1810.3.3.1.2

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1810.3.3.1.2 Load tests. Where design compressive loads are greater than those determined using the allowable stresses specified in Section 1810.3.2.6, Where the design load for any deep foundation element is in doubt, or where cast-in-place deep foundation elements have an enlarged base formed either by compacting concrete or by driving a precast base, control test elements shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one element shall be load tested in each area of uniform subsoil conditions. Where required by the *building official*, additional elements shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test element as assessed by one of the published methods listed in Section 1810.3.3.1.3 with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a *registered design professional* with consideration given to tolerable total and differential settlements at design load in accordance with Section 1810.2.3. In subsequent installation of the balance of deep foundation elements, all elements shall be deemed to have a supporting capacity equal to that of the control element where <u>all of the following are satisfied:</u>

- 1. such Elements are of the same type, size and relative length as the test element;
- 2. <u>Elements</u> are installed using the same or comparable methods and equipment as the test element;
- 3. Elements are installed in similar subsoil conditions as the test element; and,
- 4. For driven elements, where the rate of penetration (e.g., net displacement per blow) of such elements is equal to or less than that of the test element driven with the same hammer through a comparable driving distance.

Reason: This section addresses capacity of the soil or of the soil to foundation element transfer and thus the tests are not appropriate to evaluate the structural capacity of the deep foundation element. Thus the reference to Section 1810.3.2.6 is not appropriate.

Reference to registered design professional is deleted because it is redundant. The last sentence of the last paragraph has been reorganized to make it more readable.

S189-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1810.3.3.1.2-S-KERR.doc

S190–12 1810.3.3.1.6

Proponent: Lori A. Simpson, P.E., GE, Treadwell & Rollo, a Langan Company, representing Deep Foundations Institute

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 less than three 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 <u>allowable working</u> 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, plus two-thirds of the ultimate shear resistance long the soil block.

Reason: A period is added because there was a run on sentence which rendered the section unclear. Also, the spacing is clarified to be consistent with Section 1810.2.5. Section 1810.3.3.1.6 had defined the need to evaluate group effects where spacing is at least 2.5 times the least horizontal dimension, but did not define a maximum spacing at which group effects did not need to be evaluated. The minimum spacing for evaluation of group effects on uplift capacity is not appropriate. Section 1810.2.5 says that group effects only need to be evaluated where the spacing is less than 3 times the least horizontal dimension, so that is repeated herein for consistency.

S190-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
2				1810.3.3.1.6-S-SIMPSON.doc

S191–12 1810.4.12

Proponent: Mark Gilligan, S.E., representing self (mark@gilligan.name)

Delete without substitution:

1810.4.12 Special inspection. Special inspections in accordance with Sections 1705.7 and 1705.8 shall be provided for driven and cast-in-place deep foundation elements, respectively. Special inspections in accordance with Section 1705.9 shall be provided for helical piles.

Reason: This paragraph is redundant. This would imply the need for cross references from each of the material sections

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

S191-12 Public Hearing: Committee: AS AM D Assembly: ASF AMF DF 1810.4.12-S-GILLIGAN.doc

S192–12 1810.4.3

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1810.4.3 Location plan. A plan showing the location and designation of deep foundation elements by an identification system shall be filed with the *building official* prepared prior to installation of such elements. The location plan shall be submitted with the special inspection report prepared for deep foundation elements. Detailed records for elements shall bear an identification corresponding to that shown on the plan.

Reason: Since the purpose of the location plan is to allow interpretation of the inspection reports, the location plan should be provided with the special inspection report.

S192-12					
Public Hearing: Comm	ittee: A	AS	AM	D	
Assem	ibly: A	SF	AMF	DF	
					1810.4.3-S-KERR.doc

S193–12 1901.3

Proponent: Matthew Senecal, P.E., representing American Concrete Institute (ACI)

Revise as follows:

1901.3 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, embeds, 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 post-tensioning 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.
- 12. Protective coatings and systems of reinforcement.
- 13. Post-installed anchor installation requirement.
- 14. Concrete cover to reinforcement, embedments, and anchors.
- 15. Tolerances on cover, reinforcement placement, and dimensions of structural elements.
- 16. Concrete exposure category and class as defined in 1904.

Reason: This requirement is similar to 1.2.1 in ACI 318-11. It is provided here as a checklist for the building code official. ACI 318 does, however, have a few more mandatory items specified in other parts of the code. This amendment addresses those additional items.

Item 11 was adjusted for all structures. It is important that all parties, present and future, know if the slab serves an essential structural purpose. This is independent of whether seismic loads are predominating or not.

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

S193-11

Public Hearing: Co	ommittee: sembly:	AS ASF	AM AMF	D DF	
	,				1901.3-S-SENECAL

S194-12 1901.3

Proponent: Stephen V. Skalko, Portland Cement Association

Delete without substitution:

1901.3 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.
- Provision for dimensional changes resulting from creep, shrinkage and temperature.
- 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 post-tensioning 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.

Reason: There are three reasons this code change is needed to modify Chapter 19 "Concrete" of the International Building Code (IBC):

- 1 This proposal eliminates duplications between the IBC and ACI 318. ACI 318 already addresses the requirements for construction documents in Chapter 1.
- 2. Changes to requirements for construction documents in ACI 318 will not conflict with a separate list in the IBC
- 3. The list in the IBC is only a partial listing of the requirements in ACI 318 suggesting that the other requirements for reporting information in construction documents in accordance with ACI 318 are not necessary.

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

S194-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1901.3-S-SKALKO.doc

S195–12 1903.1, 1903.2, 1903.3

Proponent: Stephen V. Skalko, Portland Cement Association

Revise as follows:

1903.1 General. Materials used to produce concrete, concrete itself and testing thereof shall comply with the applicable standards listed in ACI 318. *Where required, special inspections and tests shall be in accordance with Chapter 17.*

1903.2 1903.1 Glass fiber reinforced concrete. Glass fiber reinforced concrete (GFRC) and the materials used in such concrete shall be in accordance with the PCI MNL 128 standard.

1903.3 1903.2 Flat wall insulating concrete form (ICF) systems. *Insulating concrete form material used for forming flat concrete walls shall conform to ASTM E 2634.*

Reason: "Section 1901.2 Plain and reinforced concrete." already requires that materials and tests be in compliance with ACI 318. This redundancy is not required. Also, provisions of chapter 17 of the code are applicable based on Section 1901.4. The language "Where required, special inspections and tests shall be in accordance with Chapter 17." is unnecessary and is being deleted.

S195-12					
Public Hearing:	Committee:	AS	AM	D	
•	Assembly:	ASF	AMF	DF	
					1903.1 #1-S-SKALKO.doc

S196–12 1903.1, 1903.2 (NEW), Chapter 35 (NEW)

Proponent: Stephen V. Skalko, Portland Cement Association

Revise as follows:

1903.1 General. Materials used to produce concrete, concrete itself and testing thereof shall comply with the applicable standards listed in ACI 318.

Exception. The following standards as referenced in Chapter 35 shall be permitted to be used.

1. ASTM C 150 2. ASTM C 595 3. ASTM C 1157

<u>1903.2 Special Inspections.</u> *Where required, special inspections and tests shall be in accordance with Chapter 17.*

Add new standards to Chapter 35:

ASTM

C150-12 Specification for Portland Cement

C595-12 Specification for Blended Hydraulic Cement

C1157-11 Standard Performance Specification for Hydraulic Cement

Reason: To update the specifications standards for Portland Cement, Blended Hydraulic Cement, and Hydraulic Cement referenced for use in concrete. Due to the change in the IBC code development cycle, ACI 318-11 may be the edition finally referenced for concrete in IBC 2015. ACI 318-11 references the 2009 editions of C150, C595 and C1157, which would be more than five years out of date by 2015.

ASTM Committee C01 approved modifications included in the most recent editions of these cement standards that are compatible with ACI 318-11 or ACI 318-14 and provide improvements to the standards as follows:

ASTM C150-12

Compared to ASTM C150-09 referenced in ACI 318-11, ASTM C150-12 includes revisions that:

- Make the air permeability test the default method for determining compliance with specific surface fineness requirements and moves determination by the turbidimetric method to the optional table. This reflects industry practice.
- Clarification on Type II (MH) moderate heat and moderate sulfate resistant cement heat index requirements, clarification on procedure for determining potential phase (Bogue) composition, and some additional minor improvements. No changes are made to the physical or chemical requirements of C150.

Additionally, compared to ASTM C150-07a referenced in IBC 2012 Chapter 35, ASTM C150-12 includes revisions to:

- 1. Distinguish between organic and inorganic processing additions and include a limit of 5% on inorganic processing additions and 1% on organic processing additions.
- 2. Modify procedures for determining potential phase composition to account for effect of inorganic processing additions in cement on potential phase composition calculations.
- 3. Include provisions for a Type II (MH) designation for moderate heat and moderate sulfate resistant cement.
- 4. Various other minor improvements. Again no changes were made to the physical or chemical requirements of C150 for portland cements.

The variations in product that will result from the use of C150-12 versus C150-07 will not adversely impact the performance of concrete with regard to compliance with ACI 318 or the provisions of the IBC.

C595-12

Compared to C595-09 referenced in ACI 318-11, ASTM C595-12 includes revisions to:

- Include provisions for a new Type IL portland-limestone blended cement designation for cement containing from 5% to 15% limestone. C595 Type IL has same physical requirements as Type IP and IS (<70), which are also comparable to ASTM C150 physical requirements. Portland-limestone cement provides an alternative for improving the sustainability of concrete.
- 2. Several clarifications and improvements to the C595 provisions for Type IT ternary blended cements.
- 3. Clarifications and improvements to C595 naming practice used to identify amount slag, pozzolan or limestone contained in blended cements.

Additionally, compared to C595-08a referenced in IBC 2012 Chapter 35, ASTM C595-12 also includes provisions for Type IT ternary blended cement (cements containing portland cement with either a combination of two different pozzolans, or slag cement and a pozzolan, a pozzolan and a limestone, or a slag cement and a limestone). Ternary blended cements have the same physical requirements as Type IT and Type IS (<70) cements. Ternary blended cements were first introduced in the 2009 edition of ASTM C595.

The variations in product that will result from the use of C595-12 versus C595-08a will not adversely impact the performance of concrete with regard to compliance with ACI 318 or the provisions of the IBC.

ASTM C1157-12

Compared to C1157-09 referenced in ACI 318-11, ASTM C595-12 includes revisions to:

- 1. Include provisions for distinguishing between air entraining and non air-entraining C1157 cements with appropriate
- designations and limits consistent with those of ASTM C150 and C595 for air entraining and non air entraining cements.

2. A minor modification to correct the significant figures for minimum strength limits for SI unit values listed in Table 1. The variations in product that will result from the use of C1157-12 versus C1157-09 will not adversely impact the performance of concrete with regard to compliance with ACI 318 or the provisions of the IBC.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S196-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1903.1 #2-S-SKALKO.doc

S197–12 1903.2

Proponent: Matthew Senecal, P.E., representing American Concrete Institute (ACI)

Revise as follows:

1903.2 Glass fiber reinforced concrete. Glass fiber reinforced concrete (GFRC) and the materials used in <u>such nonstructural precast</u> concrete <u>elements</u> shall be in accordance with the PCI MNL 128 standard.

Reason: This is a clarification on what concrete elements are covered by PCI MNL 128.

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

S197-11

• • • • • • •				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1903.2-S-SENECAL

S198–12 1903.4 (NEW), Chapter 35 (NEW)

Proponent: James K. Hicks, P.E., Cera Tech, P.E., representing self (jim.hicks@ceratechinc.com)

Add new text as follows:

1903.4 Rapid hardening concrete. Rapid hardening concrete shall be permitted to be produced using hydraulic cement conforming to ASTM C1600.

Add new standard to Chapter 35 as follows:

ASTM

C 1600-11 Standard Specification for Rapid Hardening Hydraulic Cement

Reason: In those instances wherein rapid hardening is desired, cements conforming to ASTM C 1600 Standard Specification for Rapid hardening Hydraulic Cements are generally desirable and useable. ASTM C 1600 can be one of four cement types, General Rapid Hardening (GRH), Moderate Rapid Hardening (MRH), Very Rapid Hardening (VRH) and Ultra Rapid Hardening (URH). C 1600 is a Specification giving numerous performance requirements. Primary characteristics (with inherent increased design flexibility) are:

•Can produce rapid-hardening concrete, precast concrete, block, mortar and grout.

•Depending on the type cement used and the specific mixture, cements meeting ASTM C 1600 can provide either normal, medium or fast time to service (1.5 to 48 h)

•ASTM C 1600 has rigid durability requirements.

ASTM C 1600 cements are used in products such as:

Materials for Concrete Repairs

High Strength Grouts

Precast

Paving

Some Cements - Mass Concrete

Some Cements – Heat Resistant

· Some Cements - Chemical Resistant

In addition to following pertinent ACI and ASTM requirements, users of C 1600 cements must heed manufacturers instructions for use. Specific durability aspects of any given mortar or concrete to be evaluated by the appropriate test method(s).

Cost Impact: Economic cost of the concrete utilizing C 1600 cements while it may be approximately equal or higher when comparing cementitious to cementitious, is typically negligible for the concrete when considering the costs of other ingredients, transport, placement, finishing and curing.

Environmental costs are generally lower with C 1600 cements as fuel use is generally less, or with the case of activated fly ash based cements, no fuel is used and grinding is not required.

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S198-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					1903.4 (NEW)-HICKS.doc

S199–12 1904.1, 1904.2, Figure 1904.2, Table 1904.2

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Delete and substitute as follows:

1904.1 Exposure categories and classes. Concrete shall be assigned to exposure classes in accordance with the durability requirements of ACI 318 based on:

- 1. Exposure to freezing and thawing in a moist condition or deicer chemicals;
- 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.

1904.2 Concrete properties. Concrete mixtures shall conform to the most restrictive maximum watercementitious materials ratios, maximum cementitious admixtures, minimum air-entrainment and minimum specified concrete compressive strength requirements of ACI 318 based on the exposure classes assigned in Section 1904.1.

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 1904.2 based on the weathering classification (freezing and thawing) determined from Figure 1904.2 in lieu of the durability requirements of ACI 318.

()					
	MINIMUM SPECIFIED COMPRESSIVE STRENGTH				
TYPE OR LOCATION OF CONCRETE	(f ′ ₀ at 28 days, psi)				
CONSTRUCTION	Negligible exposure	Moderate exposure	Severe exposure		
Basement walls [€] and foundations not exposed to	2 500	3 500	2 500 ^a		
the weather	2,500	2,500	2,300		
Basement slabs and interior slabs on grade,	2 500	2 500	2 500 °		
except garage floor slabs	2,000	2,000	2,000		
Basement walls ^c , foundation walls, exterior walls					
and other vertical concrete surfaces exposed to	2,500	3,000^b	3,000 ^b		
the weather					
Driveways, curbs, walks, patios, porches, carport					
slabs, steps and other flatwork exposed to the	2,500	3,000^{b,d}	3,500 ^{b,d}		
weather, and garage floor slabs					

TABLE 1904.2 MINIMUM SPECIFIED COMPRESSIVE STRENGTH (f 'c)

For SI: 1 pound per square inch = 0.00689 MPa.

a. Concrete in these locations that can be subjected to freezing and thawing during construction shall be of air-entrained concrete in accordance with Section 1904.2.

b. Concrete shall be air entrained in accordance with ACI 318.

c. Structural plain concrete basement walls are exempt from the requirements for exposure conditions of Section 1904.2.

d. For garage floor slabs where a steel trowel finish is used, the total air content required by ACI 318 is permitted to be reduced to not less than 3 percent, provided the minimum specified compressive strength of the concrete is increased to 4,000 psi.



FIGURE 1904.2 WEATHERING PROBABILITY MAP FOR CONCRETE^{a,b,c}

- Lines defining areas are approximate only. Local areas can be more or less severe than indicated by the region classification.
 A "severe" classification is where weather conditions encourage or require the use of deicing chemicals or where there is potential for a continuous presence of moisture during frequent cycles of freezing and thawing. A "moderate" classification is where weather conditions encourage or require to freezing and thawing, but where deicing chemicals are not generally used. A "negligible" classification is where weather conditions rarely expose concrete in the presence of moisture to freezing and thawing.
- c. Alaska and Hawaii are classified as severe and negligible, respectively.

1904.1 Structural concrete. Structural concrete shall conform to the durability requirements of ACI 318.

1904.2 Nonstructural concrete. The registered design professional shall assign nonstructural concrete a freeze-thaw exposure class, as defined in ACI 318, based on the anticipated exposure of nonstructural concrete. Nonstructural concrete shall have a minimum specified compressive strength, f'_{c} of 2500 psi for Class F0; 3000 psi for Class F1; and 3500 psi for Classes F2 and F3. Nonstructural concrete shall be air entrained in accordance with ACI 318.

Reason: This proposal replaces the weathering probability map with ACI 318's performance requirements; removes the exception for structural concrete; and clarifies the durability requirements for nonstructural concrete.

Probability map: The weathering probability map for concrete can be inaccurate since it is possible to have "severe," "moderate," or "negligible" environments in any of the predefined zones shown on the map. ACI 318 requires the designer to classify concrete into one of the freezing and thawing classes as follows:

- F0 Concrete not exposed to freezing-and-thawing cycles
- F1 Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture

F2 - Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture

F3 – Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals

The concrete classes must be applied by the designer, regardless of geographic location. The commentary to ACI 318 provides further discussion and examples to help the designer determine the appropriate class. It is therefore recommended to remove the map and adopt the ACI 318 approach.

Table: The first and second rows of the table provide limits for interior concrete. Interior concrete is equivalent to Class F0 in ACI 318, which requires a minimum concrete compressive strength of 2500 psi. Therefore, the minimum concrete compressive strength requirements listed in the first two rows are the same as the minimum requirements of ACI 318 and may be removed.

The third row of the table provides an exception for exterior structural concrete walls above or below ground. The exception allows for 3000 psi concrete for any environment other than "negligible" or Class F0. Research¹⁻² shows that concrete with a minimum amount of hydrated cement resists the negative effects of freezing and thawing. ACI 318 has determined that 4500 psi concrete provides adequate cement hydration for the range of available concrete mixtures used in construction. It is therefore recommended to remove this exception for structural concrete.

The fourth row of the table states strength limits for exterior nonstructural concrete. ACI 318 does not have durability requirements for nonstructural concrete. Therefore, these limits are not an exception to 318 but a requirement. These limits are simply restated in terms of exposure classes as shown in the revision. The limitation on building category and concrete type have been removed, since this appears to be a misunderstanding of what is required in ACI 318.

References:

1. Klieger, P., 1956, "Curing Requirements for Scale Resistance of Concrete," Highway Research Board Bulletin 150, pp.18-31. (PCA Bulletin 82)

2. Mather, B., 1990, "How to Make Concrete that will be Immune to the Effects of Freezing and Thawing," Paul Klieger Symposium on Performance of Concrete, SP-122, D. Whiting, ed., American Concrete Institute, Farmington Hills, MI, pp. 1-18.

Cost Impact: The code change proposal may increase the cost of construction for structural concrete but decrease the cost for nonstructural concrete. By changing the requirement from geometric location to performance criteria, the cost will increase or decrease depending on location and exposure.

S199-12

Public Hearing:	Committee:	AS	AM	D	
0	Assembly:	ASF	AMF	DF	
	2				1904.1 (NEW)-S-SENECAL

S200–12 1904, 1904.1, 1904.2, Table 1904.2, Figure 1904.2

Proponent: Stephen V. Skalko, Portland Cement Association

Delete without substitution:

SECTION 1904 DURABILITY REQUIREMENTS

1904.1 Exposure categories and classes. Concrete shall be assigned to exposure classes in accordance with the durability requirements of ACI 318 based on:

- 1. Exposure to freezing and thawing in a moist condition or deicer chemicals;
- 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.

1904.2 Concrete properties. Concrete mixtures shall conform to the most restrictive maximum watercementitious materials ratios, maximum cementitious admixtures, minimum air-entrainment and minimum specified concrete compressive strength requirements of ACI 318 based on the exposure classes assigned in Section 1904.1.

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 1904.2 based on the weathering classification (freezing and thawing) determined from Figure 1904.2 in lieu of the durability requirements of ACI 318.

TABLE 1904.2 MINIMUM SPECIFIED COMPRESSIVE STRENGTH (f 'c)

FIGURE 1904.2 WEATHERING PROBABILITY MAP FOR CONCRETE^{a, b, c}

Reason: There are three reasons that support this code change intended to modify Chapter 19 "Concrete" of the *International Building Code* (IBC):

- 1. This proposal removes redundancies in language and reference to specific sections of ACI 318. Such redundancies are not necessary and may be detrimental in that they may cause confusion and result in errors. The introductory section of Chapter 19, "1901.2 Plain and reinforced concrete" adequately and appropriately requires compliance with ACI 318.
- 2. Current language and approaches referenced in the IBC are inconsistent with both the methods for classifying exposure and the requirements for concrete based on the various exposures. Exposures are no longer limited to freeze-thaw durability and weathering. The new criteria in ACI 318 address freezing and thawing, sulfate, low permeability and corrosion protection. The weathering probability map in no way reflects the problems associated with sulfate exposure. The map puts areas that have the potential for high sulfate exposure into the negligible category.
- 3. It is clearly not the intent of ACI 318 to have provisions that are only applicable to certain structures. ACI 318 has been developed and is intended for use in such a manner that all provisions are applicable for all structural concrete regardless of occupancy. All provisions of ACI 318 are applicable to all structural concrete regardless of occupancy.

S200-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
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S201–12 202, 1901.2, 1902, 1905.1 thru 1905.1.10, 1906.1

Proponent: Stephen V. Skalko, P.E., Portland Cement Association

Add new text as follows:

SECTION 202 DEFINITIONS

WALL PIER. A wall segment with a horizontal length-to-thickness ratio of at least 2.5, but not to exceed 6, whose clear height is at least two times its horizontal length.

Revise as follows:

1901.2 Plain and reinforced concrete. Structural concrete shall be designed and constructed in accordance with the requirements of this chapter and ACI 318 as amended in Section 1905 of this code. Except for the provisions of Sections 1904 and 1907, the design and construction of slabs on grade shall not be governed by this chapter unless they transmit vertical loads or lateral forces from other parts of the structure to the soil.

1902.1 General. The words and terms defined in ACI 318 shall, for the purposes of this chapter and as used elsewhere in this code for concrete construction, have the meanings shown in ACI 318 as modified by Section 1905.1.1.

1905.1 General. The text of ACI 318 shall be modified as indicated in Sections 1905.1.1 through 1905.1.10.

1905.1.1 ACI 318, Section 2.2. Modify existing definitions and add the following definitions to ACI 318, Section 2.2.

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.

ORDINARY PRECAST STRUCTURAL WALL. A precast wall complying with the requirements of Chapters 1 through 18.

ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. A cast-in-place wall complying with the requirements of Chapters 1 through 18.

ORDINARY STRUCTURAL PLAIN CONCRETE WALL. A wall complying with the requirements of Chapter 22, excluding 22.6.7.

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 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.

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1905.1.2 ACI 318, Section 21.1.1.
1905.1.3 ACI 318, Section 21.4.
1905.1.4 ACI 318, Section 21.9.
1905.1.5 ACI 318, Section 21.10.
1905.1.6 ACI 318, Section 21.12.1.1.
1905.1.7 ACI 318, Section 22.6.
1905.1.8 ACI 318, Section 22.10.
1905.1.9 ACI 318, Section D.3.3.
1905.1.10 ACI 318, Section D.4.2.2.
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1906.1 Scope. The design and construction of structural plain concrete, both cast-in-place and precast, shall comply with the minimum requirements of ACI 318, as modified in Section 1905.

Exception: For Group R-3 occupancies and buildings of other occupancies less than two stories above grade plane of light-frame construction, the required 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.

Reason: There are four main reasons in support of this code change intended as modifications of Chapter 19 "Concrete" of the *International Building Code* (IBC):

- The requirements and modifications that currently appear in Section 1905 of the 2012 edition of the IBC have been appropriately considered via an ANSI accredited standards development process. Modifications to ACI 318 in the IBC are unnecessary.
- 2) The proposal removes redundancies in definitions of ACI 318. Such redundancies are not necessary and may be detrimental in that they may cause confusion and result in errors. The introductory section of Chapter 19, "1901.2 Plain and reinforced concrete" adequately and appropriately requires compliance with ACI 318.
- 3) This proposal improves the consistency of this chapter with other chapters of the IBC such as Chapter 20 which simply states: "Aluminum used for structural purposes in buildings and structures shall comply with AA ASM 35 and AA ADM 1. The nominal loads shall be the minimum design loads required by Chapter 16."
- If the chapters and sections in whatever edition of ACI 318 that becomes referenced in the 2015 edition of the IBC are not properly coordinated, there will be confusion and will increase the potential for errors in design and construction.
 In addition to these general reasons specific additional reasons are provided for each part:

Section 1902.1in the IBC simply directs the user to the appropriate sections of ACI 318 which are already mandated in Section 1901.2. This deletes redundant language.

Section 1905

- "Design displacement" is adequately and appropriately defined in ACI 318.
- Current definition of "Detailed plain concrete structural wall" is not a definition and inappropriately sets design and construction criteria in a definition. Further if the wall is in compliance with Chapter 22 then it is also in compliance with Section 22.6.7 and the redundancy is not necessary.
- Current definition of "Ordinary precast structural wall" is not a definition and inappropriately sets design and construction criteria in a definition.
- Current definition of "Ordinary reinforced concrete structural wall" is not a definition and inappropriately sets design and construction criteria in a definition.
- Current definition of "Ordinary structural plain concrete wall" is not a definition and inappropriately sets design and construction criteria in a definition.
- Current definition of "Special structural wall" is not a definition and inappropriately sets design and construction criteria in a definition. This definition also further modifies the definitions in ASCE 7 increasing confusing.

Wall pier

6204 42

• The definition of "Wall pier" is not specific to concrete and should be included in Chapter 2.

Section 1905.1.2 thru 1905.1.10

Most of the current sections in the IBC simply direct the user to the appropriate sections of ACI 318 which are already mandated in Section 1901.2. In addition, these provisions have been addressed by the ACI 318 Committee.

3201-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				1901.2-S-SKALKO.doc

S202–12 1905.1.1, 1905.1.3, 1905.1.4, 1905.1.9, 1905.1.10

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1905.1.1 ACI 318, Section 2.2. Modify existing definitions and add the following definitions to ACI 318, Section 2.2.

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.

ORDINARY PRECAST STRUCTURAL WALL. A precast wall complying with the requirements of Chapters 1 through 18.

ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. A *cast-in-place* wall complying with the requirements of Chapters 1 through 18.

ORDINARY STRUCTURAL PLAIN CONCRETE WALL. A wall complying with the requirements of Chapter 22, *excluding 22.6.7*.

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 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.

1905.1.3 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, 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 Sy. 21.4.5 - Wall piers in Seismic Design Category D, E or F shall comply with Section 1905.1.4 of the International Building Code.

21.4.6 - Wall piers not designed as part of a moment frame in buildings assigned to Seismic Design Category 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 (305 mm).

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.

21.4.7 - Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

1905.1.4 ACI 318, Section 21.9. Modify ACI 318, Section 21.9, by deleting Section 21.9.8 and replacing with the following:

21.9.8 - Wall piers and wall segments.

21.9.8.1 - Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in 21.9.8.2.

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 segment have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.

21.9.8.2 - Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces 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).

21.9.8.3 - Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

1905.1.9 ACI 318, Section D.3.3. Delete ACI 318 Sections D.3.3.4 through D.3.3.7 and replace with the following:

D.3.3.4 - The anchor design strength associated with concrete failure modes shall be taken as $0.75 \phi N_n$ and $0.75 \phi V_n$, where ϕ is given in D4.3 or D4.4 and N_n and V_n are determined in accordance with D5.2, D5.3, D5.4, D6.2 and D6.3, assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.

D.3.3.5 - 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.3.3.6 or D.3.3.7 is satisfied.

Exceptions:

- 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.
- 2. D.3.3.5 need not apply and the design shear strength in accordance with D.6.2.1(c) need not be computed for anchor bolts attaching wood sill plates of bearing or non-bearing walls of light-frame wood structures to foundations or foundation stem walls provided all of the following are satisfied:
 - 2.1. The allowable in-plane shear strength of the anchor is determined in accordance with AF&PA NDS Table 11E for lateral design values parallel to grain.
 - 2.2 The maximum anchor nominal diameter is 5/8 inches (16 mm).
 - 2.3. Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).
 - 2.4. Anchor bolts are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate.
 - 2.5. Anchor bolts are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.
 - 2.6. The sill plate is 2-inch or 3-inch nominal thickness.
- 3. Section D.3.3.5 need not apply and the design shear strength in accordance with Section

D.6.2.1(c) need not be computed for anchor bolts attaching cold-formed steel track of bearing or non-bearing walls of light-frame construction to foundations or foundation stem walls provided all of the following are satisfied:

- 3.1. The maximum anchor nominal diameter is 5/8 inches (16 mm).
- 3.2. Anchors are embedded into concrete a minimum of 7 inches (178 mm).
- 3.3. Anchors are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the track.
- 3.4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.
- 3.5. The track is 33 to 68 mil designation thickness.

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

4. In light-frame construction, design of anchors in concrete shall be permitted to satisfy D.3.3.8.

D.3.3.6 - Instead of D.3.3.5, 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.3.3.4.

Exceptions:

- 1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.6.
- 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.3.3.6.

D.3.3.7 - As an alternative to D.3.3.5 and D.3.3.6, it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.4.

D.3.3.8 – In light-frame construction, bearing or nonbearing walls, shear strength of concrete anchors less than or equal to 1 inch [25 mm] in diameter of sill plate or track to foundation or foundation stem wall need not satisfy D.3.3.7 when the design strength of the anchors is determined in accordance with D.6.2.1(c).

1905.1.10 ACI 318, Section D.4.2.2. Delete ACI 318, Section D.4.2.2, 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-7 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.

Reason: The purpose for this proposal is to update the 2012 IBC for consistency with ACI 318-11 and as explained below.

- 1. In IBC Section 1905.1.1, the definition of "wall pier" is deleted because of the definition of "wall pier" in Section 2.2 of ACI 318-11.
- 2. In IBC Section 1905.1.3, Sections 21.4.5 through 21.4.7 are deleted because of Section 21.4.4 of ACI 318-11, which reads: "In structures assigned to SDC D, E or F, wall piers shall be designed in accordance with 21.9 or 21.13."
- 3. IBC Section 1905.1.4 is deleted because of Section 21.9.8 of ACI 318-11, which specifies requirements for wall piers.
- 4. IBC Section 1905.1.9 is deleted because of Sections D.3.3.4 through D.3.5 of ACI 318-11, which specify seismic design requirements for anchors in structures that are substantially revised from the corresponding provisions in Sections D.3.3.3 through D.3.3.6 of ACI 318-08.
- IBC Section 1905.1.10 is deleted because of Sections D.4.2.2 and D.4.3 of ACI 318-11, which specify requirements for concrete breakout strength and bond strength that are substantially revised from the corresponding provisions in Section D.4.2.2 of ACI 318-08.

S202-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	,				1905.1.1-S-BRAZIL.doc

S203–12 1905.1, 1905.1.1, 1905.1.3, 1905.1.4

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Revise as follows:

1905.1 General. The text of ACI 318 shall be modified as indicated in Sections 1905.1.1 through 1905.1.10 1905.1.9.

WALL PIER. A wall segment with a horizontal length-tothickness ratio of at least 2.5, but not exceeding 6, whose clear height is at least two times its horizontal length.

1905.1.3 ACI 318, Section 21.4. Modify ACI 318, Section 21.4, by <u>adding new Section 21.4.3 and</u> renumbering <u>existing</u> Section 21.4.3 to become 21.4.4. and adding new Sections 21.4.3, 21.4.5, 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 Sy.

21.4.5 - Wall piers in Seismic Design Category D, E or F shall comply with Section 1905.1.4 of the International Building Code.

21.4.6 - Wall piers not designed as part of a moment frame in buildings assigned to Seismic Design Category 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 (305 mm).

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.

21.4.7 - Wall segments with a horizontal length-to thickness ratio less than 2.5 shall be designed as columns.

1905.1.4 ACI 318, Section 21.9. Modify ACI 318, Section 21.9, by deleting Section 21.9.8 and replacing with the following:

21.9.8 - Wall piers and wall segments.

21.9.8.1 - Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in 21.9.8.2.

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.

21.9.8.2 - Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces 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).

21.9.8.3 - Wall segments with a horizontal length-to thickness ratio less than 2.5 shall be designed as columns.

Reason: This proposal removes the requirements for wall piers. Wall pier requirements are in 1905 because ACI 318-08 did not address the design of this component. ACI 318 incorporated wall pier design in the 2011 edition. Therefore, these amendments should now be removed.

S203-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1905.1-S-SENECAL

S204–12

1905.1, 1905.1.1, 1905.1.2 (NEW), 1905.1.2.1 (NEW), 1905.1.2.2 (NEW), 1905.1.2.3 (NEW), 1905.1.2.4 (NEW), 1905.1.2.5 (NEW), 1905.1.3.1 (NEW), 1905.1.3.2 (NEW), 1905.1.3.3 (NEW), 1905.1.3.4, 1905.1.3.5, 1905.1.4.1 (NEW), 1905.1.4.2 (NEW), 1905.1.4.3 (NEW), 1905.1.4.4, 1905.1.5.1

Proponent: Stephen V. Skalko, Portland Cement Association

Revise as follows:

SECTION 1905 MODIFICATIONS TO ACI 318 SEISMIC DESIGN OF STRUCTURAL CONCRETE

1905.1 General. The text of <u>Concrete shall be designed and constructed in accordance with</u> ACI 318 shall be modified as indicated in <u>and</u> Sections 1905.1.1 through 1905.1.10.

1905.1.1 ACI 318, Section 2.2 Definitions. Modify existing definitions and add the following definitions to ACI 318, Section 2.2. The following definitions shall apply:

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.

ORDINARY PRECAST STRUCTURAL WALL. A precast wall complying with the requirements of Chapters 1 through 18.

ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. A cast-in-place wall complying with the requirements of Chapters 1 through 18.

ORDINARY STRUCTURAL PLAIN CONCRETE WALL. A wall complying with the requirements of Chapter 22, excluding 22.6.7.

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 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 all."

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.

1905.1.2 Structural concrete assemblies. Structural concrete assemblies described shall comply with the requirements of this Section.

1905.1.2.1 Detailed plain concrete structural wall. Detailed plain concrete structural walls shall comply with the requirements of ACI 318 Chapter 22 including Section 22.6.7, and the applicable requirements of Sections 1905.3 through 1905.11.

1905.1.2.2 Ordinary precast structural wall. -Ordinary precast structural walls shall comply with the requirements of ACI 318 Chapters 1 through 18 and the applicable requirements of Sections 1905.3 through 1905.11.

<u>1905.1.2.3 Ordinary reinforced concrete structural wall</u>. A cast-in-place ordinary reinforced concrete structural wall comply with the requirements of ACI 318 Chapters 1 through 18 and the applicable requirements of Sections 1905.3 through 1905.11.

1905.1.2.4 Ordinary structural plain concrete wall. Ordinary structural plain concrete walls shall comply with the requirements of ACI 318 Chapter 22, excluding 22.6.7 and the applicable requirement of Sections 1905.3 through 1905.11.

1905.1.2.5 Special structural wall. Special structural walls made of cast-in-place or precast concrete shall comply with the applicable requirements of ACI 318 Sections 21.1.3 through 21.1.7, 21.1.9, and 21.1.10 and Sections 1905.3 through 1905.11 and 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."

1905.1.2 ACI 318, Section 21.1.1 . Modify ACI 318 Sections 21.1.1.3 and 21.1.1.7 to read as follows: **1905.1.3 Requirements for structures based on assigned Seismic Design Categories.** The requirements of this section shall apply for the assigned Seismic Design Category.

1905.1.3.1 Provisions of ACI 318. The provisions of ACI 318 Sections 21.1.1.3 and Section 21.1.1.7 shall not apply.

<u>21.1.1.3</u> – <u>1905.1.3.2</u> Structures assigned to Seismic Design Category A. <u>Structures assigned to</u> <u>Seismic Design Category A</u> shall satisfy requirements of <u>ACI 318</u> Chapters 1 to 19 and 22 <u>and the</u> <u>requirements of ACI</u> Chapter 21 does <u>shall</u> not apply.

1905.1.3.3 Structures assigned to Seismic Design Category B, C, D, E or F. 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.

<u>1905.1.3.4 Structural elements of plain concrete.</u> Except for structural elements of plain concrete complying with Section 1905.1.8 of the International Building Code, Structural elements of plain concrete are prohibited in structures assigned to Seismic Design Category C, D, E or F.

Exception: Structural elements of plain concrete complying with Section 1905.1.9

21.1.1.7 – **1905.1.3.5 Seismic force resisting systems.** 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 <u>for structures assigned to, regardless of the</u> Seismic Design Category B, C, D, E or F:

- (a) Ordinary moment frames shall satisfy ACI 318 Section 21.2.
- (b) Ordinary reinforced concrete structural walls *and ordinary precast structural walls* need not satisfy any provisions in <u>ACI 318</u> Chapter 21.
- (c) Intermediate moment frames shall satisfy ACI 318 Section 21.3.
- (d) Intermediate precast structural walls shall satisfy ACI Section 21.4.
- (e) Special moment frames shall satisfy ACI 318 Section 21.5 through 21.8.
- (f) Special structural walls shall satisfy ACI 318 Section 21.9.
- (g) Special structural walls constructed using precast concrete shall satisfy ACI 318 Section 21.10.

All special moment frames and special structural walls shall also satisfy <u>ACI 318 Section</u> 21.1.3 through 21.1.7.

1905.1.3 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, 21.4.6 and 21.4.7 to read as follows:

1905.1.4 Connections. Connections shall comply with the requirements of ACI 318 Section 21.4 and the following requirements:

<u>21.4.3</u> <u>1905.1.4.1 Connections designed to yield.</u> 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 Sy- 21.4.5 -

<u>1905.1.4.2 Wall piers in Seismic Design Category D, E, or F.</u> *Wall piers in Seismic Design Category D, E or F shall comply with Section* 1905.1.4 <u>1905.1.5</u> *of the International Building Code*.

<u>21.4.6</u> <u>1905.1.4.3 Wall piers not designed as part of a moment frame in buildings assigned to</u> <u>Seismic Design Category C.</u> Wall piers not designed as part of a moment frame in buildings assigned to Seismic Design Category C shall have transverse reinforcement designed to resist the shear forces determined from <u>ACI 318 Section</u> 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 <u>ACI 318 Section</u> 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.

<u>21.4.7 –</u> <u>1905.1.4.4 Wall segments.</u> Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

1905.1.4 ACI 318, Section 21.9. Modify ACI 318, Section 21.9, by deleting Section 21.9.8 and replacing with the following: <u>1905.1.5 Special structural walls and coupling beams.</u> Wall piers and wall segments in special structural walls shall comply with Section 1905.1.5.1.

21.9.8 - 1905.1.5.1 Wall piers and wall segments.

 $\frac{21.9.8.1}{21.9.8.1}$ — <u>a.</u> Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in <u>ACI 318 Section</u> 21.9.8.2.

Exceptions:

- 1. Wall piers that satisfy <u>ACI 318 Section</u> 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.

 $\frac{21.9.8.2}{10}$ b. Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces determined from <u>ACI 318 Section</u> 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).

21.9.8.3 <u>c.</u> Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

1905.1.5 ACI 318, Section 21.10. Modify ACI 318, Section 21.10.2, to read as follows: **1905.1.6 Special structural walls constructed using precast.** In addition to Section 21.10.2 of ACI 318 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 <u>Section 1905.1.4.1</u>. **1905.1.6 ACI 318, Section 21.12.1.1.** Modify ACI 318, Section 21.12.1.1, to read as follows: 21.12.1.1 - Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between a structure and ground shall comply with the

1905.1.7 Foundations. The requirements of this section shall apply for foundations resisting earthquakeinduced forces or transferring earthquake-inducted forces between a structure and ground.

- a. The requirements of ACI 318 Section 21.12.1.1 shall not apply.
- b. <u>The</u> requirements of <u>ACI 318</u> Section 21.12 and other applicable provisions of ACI 318 unless modified by Chapter 18 of the International Building Code.

1905.1.7 ACI 318, Section 22.6. Modify ACI 318, Section 22.6, by adding new Section 22.6.7 to read as follows:

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.

<u>22.6.7.2 -</u> <u>1905.1.8 Detailed plain structural concrete walls</u>. For detailed plain structural concrete wall reinforcement shall be provided as follows:

(a) <u>1905.1.8.2 Vertical reinforcement.</u> Vertical reinforcement of at least 0.20 square inch (129 mm2) 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>ACI 318 Section</u> 22.6.6.5.

(b) <u>1905.1.8.2 Horizontal reinforcement.</u> Horizontal reinforcement at least 0.20 square inch (129 mm2) 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.

1905.1.8 ACI 318, Section 22.10. Delete ACI 318, Section 22.10, and replace with the following:

<u>22.10 – 1905.1.9 Plain concrete in earthquake-resisting structures.</u> The requirements of this Section shall apply to plain concrete in structures assigned to Seismic Design Category C, D, E or F. (a) The requirements of ACI 318 Section 22.10 shall not apply.

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

(a) <u>1</u>. 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 studbearing 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) <u>2.</u> 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) <u>3.</u> Plain concrete footings supporting walls are permitted, provided the footings have at least two continuous longitudinal reinforcing bars.

<u>c.</u> 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.

<u>d.</u> For footings that exceed 8 inches (203 mm) in thickness, a minimum of one bar shall be provided at the top and bottom of the footing. Continuity of reinforcement shall be provided at corners and intersections.

Exceptions:

- 1. In Seismic Design Categories A, B and C, detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls, are permitted to have plain concrete footings without longitudinal reinforcement.
- 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.

1905.1.9 ACI 318, Section D.3.3. Delete ACI 318 Sections D.3.3.4 through D.3.3.7 and replace with the following:

D.3.3.4 – 1905.10 Seismic design for anchoring to concrete. Requirements for seismic design of anchorage to concrete shall comply with this Section.

- (a). The requirements of ACI 318 Sections D3.3.4 through D3.3.7 shall not apply.
- (b). The anchor design strength associated with concrete failure modes shall be taken as 0.75ØNn and 0.75ØVn, where Ø is given in D4.3 or D4.4 and Nn and Vn are deteremined in accordance with <u>ACI 318 Sections</u> D5.2, D5.3, D5.4, D6.2 and D6.3, assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.
- <u>(c).</u> D.3.3.5 Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with <u>ACI 318 Sections</u> D.5.1 and D.6.1, unless either D.3.3.6 or D.3.3.7 is satisfied.

Exceptions:

- 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 <u>ACI 318</u> Section D.3.3.5.
- <u>ACI 318 Section</u> D.3.3.5 need not apply and the design shear strength in accordance with <u>ACI Section</u> D.6.2.1(c) need not be computed for anchor bolts attaching wood sill plates of bearing or non-bearing walls of light-frame wood structures to foundations or foundation stem walls provided all of the following are satisfied:
 - 2.1. The allowable in-plane shear strength of the anchor is determined in accordance with AF&PA NDS Table 11E for lateral design values parallel to grain.
 - 2.2 The maximum anchor nominal diameter is 5/8 inches (16 mm).
 - 2.3. Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).
 - 2.4. Anchor bolts are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate.
 - 2.5. Anchor bolts are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.
 - 2.6. The sill plate is 2-inch or 3-inch nominal thickness.

- <u>ACI 318</u> Section D.3.3.5 need not apply and the design shear strength in accordance with <u>ACI 318</u> Section D.6.2.1(c) need not be computed for anchor bolts attaching coldformed steel track of bearing or non-bearing walls of light-frame construction to foundations or foundation stem walls provided all of the following are satisfied:
 2.1 The provided provided all of the following are satisfied:
 - 3.1. The maximum anchor nominal diameter is 5/8 inches (16 mm).
 - 3.2. Anchors are embedded into concrete a minimum of 7 inches (178 mm).
 - 3.3. Anchors are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the track.
 - 3.4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.
 - 3.5. The track is 33 to 68 mil designation thickness.

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

4. In light-frame construction, design of anchors in concrete shall be permitted to satisfy <u>Section 1905.1.1(f)</u> D.3.3.8.

D.3.3.6 (d). Instead of D.3.3.5 the requirements in Section 1905.1.10(c), 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 Section 1905.1.10 (b).D.3.3.4.

Exceptions:

- 1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.6<u>1905.1.10(d)</u>.
- 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.3.3.6 1905.1.10(d).

D.3.3.7 - (e). As an alternative to D.3.3.5 and D.3.3.6 Sections 1905.1.10(c) and (d), it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.4 Section 1905.1.10(b).

D.3.3.8 - (f.) In light-frame construction, bearing or nonbearing walls, shear strength of concrete anchors less than or equal to 1 inch [25 mm] in diameter of sill plate or track to foundation or foundation stem wall need not satisfy D.3.3.7 when the design strength of the anchors is determined in accordance with D.6.2.1(c).

1905.1.10 ACI 318, Section D.4.2.2. Delete ACI 318, Section D.4.2.2, and replace with the following:

D.4.2.2 - 1905.1.11 Anchors with diameters less than 4 in.

- (a). The requirements of ACI 318 Section D4.2.2. shall not apply.
- (b).. The concrete breakout strength requirements for anchors in tension shall be considered satisfied by the design procedure of <u>ACI 318 Section</u> D.5.2 provided <u>ACI 318</u> Equation D-7 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 <u>ACI 318 Section</u> D.6.2. For anchors in shear with diameters exceeding 2 inches, shear anchor reinforcement shall be provided in accordance with the procedures of <u>ACI 318 Section</u> D.6.2.9.

Reason: There are three main reasons in support of this code change intended as modifications of Chapter 19 "Concrete" of the *International Building Code* (IBC):

1. The proposal removes redundancies in definitions of ACI 318. Such redundancies are not necessary and may be detrimental in that they may cause confusion and result in errors. The introductory section of Chapter 19, "1901.2 Plain and reinforced concrete" adequately and appropriately requires compliance with ACI 318.

- 2. It is more appropriate to display several definitions in Section 1905.1.1 as code requirements instead of definitions. The language within these definitions contain specific criteria which should be stated as code requirements. These have been revised accordingly.
- 3. It is inappropriate to modify the provisions of ACI 318 within the body of the IBC. If one wishes to revise provisions of ACI 318 the changes should be submitted to the ACI process. If the provisions contained within Section 1905 are in addition to ACI 318 then they should be worded as such. This proposal makes these revisions to the IBC to reflect these additional requirements.

S204-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
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S205–12 1905.1.3, 1905.1.4, 1905.1.5

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (skerr@jwa-se.com)

Revise as follow:

1905.1.3 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, 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 Sy.

21.4.5 - Wall piers in Seismic Design Category D, E or F shall comply with Section 1905.1.4 of the International Building Code <u>ACI 318 Section 21.9.9</u>.

21.4.6 - Wall piers not designed as part of a moment frame in buildings assigned to Seismic Design Category 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 (305 mm).

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.

21.4.7 - Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

1905.1.4 ACI 318, Section 21.9. Modify ACI 318, Section 21.9, by deleting Section 21.9.8 and replacing with the following:

21.9.8 - Wall piers and wall segments.

21.9.8.1 - Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in 21.9.8.2.

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.

21.9.8.2 - Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces 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).

21.9.8.3 - Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

1905.1.5 ACI 318, Section 21.10. Modify ACI 318, Section 21.10.2, to read as follows:

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.

Reason: The purpose of this proposal is to align the IBC Chapter 19 modifications of ACI 318 with the new version of ACI 318. 1905.1.3: ACI 318 Section 21.9.9 is a new section written for wall piers in buildings assigned to SDC D, E or F. This proposal will mandate the requirement of wall pier detailing requirement in the lower SDCs which has been in the Code since 2000.

1905.1.4: Reason: ACI 318 Section 21.9.9 is a new section written for wall piers in buildings assigned to SDC D, E or F. Requirement in this section is no longer needed.

1905.1.5: This requirement is now included under ACI 318-11 section 21.9.1., the requirement in this section is no longer needed.

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

S205-12 Public Hearing: Committee: AS AM D Assembly: ASF AMF DF 1905.1.3-S-KERR.doc

S206–12 1905.1.8

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Delete and substitute as follows:

1905.1.8 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 CategoyC, 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 studbearing 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 Seismic Design Categories A, B and C, detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls, are permitted to have plain concrete footings without longitudinal reinforcement.
- 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.

1905.1.8 ACI 318, Section 22.10. The requirements of Section 22.10 shall apply for plain concrete in earthquake-resisting structures assigned to Seismic Design Category C, D, E, or F.

Reason: The only substantive difference provided in this revision to ACI 318 is the inclusion of SDC C. The remaining text provides a few detailing changes that are based on engineering practice, not design principles. It is therefore recommended to remove such changes, and retain the inclusion for plain concrete in structures assigned to SDC C. Discussion:

22.10 is an editorial revision to the heading in ACI 318 which reads, "Plain concrete in earthquake-resisting structures." This revision is unnecessary.
22.10.1.(a) above is similar to 22.10.1.(c) in ACI 318. The IBC provision limits the height of the wall to 8 ft, which is a practical limit. The design requirements of 22.6 in ACI 318 take wall height into account. Therefore, the height limit is unnecessary.

22.10.1.(b) above is similar to 22.10.1.(a) in ACI 318. The IBC provision limits the projection of the footing where ACI 318 limits the use of plain concrete. Removing this exception results in a slightly more conservative design.

22.10.1.(c) above is similar to 22.10.1.(b) in ACI 318. The IBC provision provides direction on where to place reinforcement, which is a detailing practice. Removing this exception results in no design change.

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

S206-11

Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1905.1.8-S-SENECAL

S207–12 1905.1.9

Proponent: Kevin Moore, Certus Consulting, Inc. (NCSEA, NIBS, BSSC, CRSC), representing NCSEA Seismic Subcommittee

Delete and substitute as follows:

1905.1.9 ACI 318, Section D.3.3.

1905.1.9 ACI 318, Section D.3.3.5.3 Modify ACI 318, Section D.3.3.5.3, by adding the following: exceptions:

D.3.3.5.3 — Anchors and their attachments shall be designed using one of options (a) through (c):

- (a) The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects in the attachment.
- (b) The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchors by a non-yielding attachment.
- (c) The anchor or group of anchors shall be designed for the maximum shear obtained from design load combinations that include E, with E increased by Ωo. The anchor design shear strength shall satisfy the shear strength requirements of D.4.1.1.

Exceptions:

- <u>1. Per option D.3.3.5.3 (b), anchor or group of anchor shear strength design need not</u> <u>be computed per D.6.2 or D.6.3 for anchor bolts attaching wood sill plates of bearing</u> <u>or nonbearing walls of light-frame wood structures to foundations or foundation stem</u> <u>walls provided all of the following are satisfied:</u>
 - 1.1. The allowable in-plane shear strength of the anchor is determined in accordance with AF&PA
 - <u>NDS Table 11E for lateral design values parallel to grain.</u>
 - 1.2. The maximum anchor nominal diameter is 5/8 inches (16 mm).
 - <u>1.3.</u> Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).
 - <u>1.4.</u> Anchor bolts are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate.
 - 1.5. Anchor bolts are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.
 - 1.6. The sill plate is of 2-inch or 3-inch nominal thickness.
- 2. Per option D.3.3.5.3 (b), anchor or group of anchor shear strength design need not be computed per D.6.2 or D.6.3 for anchor bolts attaching cold-formed steel track of bearing or nonbearing walls of light-frame construction to foundations or foundation stem walls provided all of the following are satisfied: 2.2. Anchors are embedded into concrete a minimum of 7 inches (178 mm).
 - 2.3. Anchors are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the track.
 - 2.4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.
 - 2.5. The track is 33 to 68 mil designation thickness.

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

Reason: As currently written in the 2012 IBC, Section 1905.1.9, Section D.3.3 does not align with the section numbering of ACI 318-11, Appendix D. This editorial revision realigns the ACI 318-11 language with primary exceptions for light frame construction sill plates, since they remain relevant.

Please note that the following exceptions were not brought forward: Exception 1 of D.3.3.5 and Exception 2 of D3.3.6 on anchors designed to resist wall out-of-plane forces; Exception 1 of D3.3.6 on anchors designed to support nonstructural components; and, Exception 4 of D.3.3.5 on light-frame construction. The applicability of Exception 1 of D.3.3.5, Exception 4 of D.3.3.5, Exception 1 of D3.3.6 and, Exception 2 of D3.3.6 applied to ACI 318-11, Appendix D could not be verified. Proponents of these exceptions are encouraged to re-evaluate them based upon provisions of ACI 318-11, Appendix D and bring forward public comments as appropriate.

S207-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					1905.1.9-S-MOORE.doc

S208–12 1905.1.9

Proponent: Stephen Kerr, S.E. Josephson Werdowatz and Associates, representing Structural Engineers Association of California (skerr@jwa-se.com)

Revise as follows:

1905.1.9 ACI 318, Section D.3.3. Delete ACI 318 Sections D.3.3.4 through D.3.3.7 and replace with the following Modify ACI Section D.3.3.5.3 as follows:

D.3.3.4 - The anchor design strength associated with concrete failure modes shall be taken as $0.75\phi N_n$ and $0.75\phi V_n$, where ϕ is given in D4.3 or D4.4 and N_n and V_n are determined in accordance with D5.2, D5.3, D5.4, D6.2 and D6.3, assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.

D.3.3.5 D.3.3.5.3 - Anchors <u>and their attachments</u> 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.3.3.6 or D.3.3.7 is satisfied. using one of options (a) through (f):

- (a) The anchor or group of anchors shall have ϕV_n not less than the maximum force that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects in the attachment.
- (b) The anchor or group of anchors shall have ϕV_n not less than the maximum shear that can be transmitted to the anchors by a non-yielding attachment.
- (c) The anchor or group of anchors shall have ϕV_n not less than the maximum shear obtained from design load combinations that include *E*, with *E* increased by Ω_0 . The anchor design shear strength shall satisfy the shear strength requirements of D.4.1.1.

Exceptions:

- 1. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.
- (d)2.D.3.3.5 D3.3.5.3 need not apply and the design shear strength in accordance with D.6.2.1(c) need not be computed for anchor bolts attaching wood sill plates of bearing or non-bearing walls of light-frame wood structures to foundations or foundation stem walls provided all of the following are satisfied:
 - 2.1. <u>1.</u> The allowable in-plane shear strength of the anchor is determined in accordance with AF&PA NDS Table 11E for lateral design values parallel to grain.
 - 2.2 2. The maximum anchor nominal diameter is 5/8 inches (16 mm).
 - 2.3. 3. Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).
 - 2.4. <u>4.</u> Anchor bolts are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate.
 - <u>2.5.5.</u> Anchor bolts are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.
 - <u>2.6.</u> The sill plate is 2-inch or 3-inch nominal thickness.
- (e)3. Section D.3.3.5 3.3.5.3 need not apply and the design shear strength in accordance with Section D.6.2.1(c) need not be computed for anchor bolts attaching cold-formed steel track

of bearing or non-bearing walls of light-frame construction to foundations or foundation stem walls provided all of the following are satisfied:

- 3.1. <u>1.</u> The maximum anchor nominal diameter is 5/8 inches (16 mm).
- 3.2. 2. Anchors are embedded into concrete a minimum of 7 inches (178 mm).
- 3.3. <u>3.</u> Anchors are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the track.
- <u>3.4.</u> <u>4.</u> Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.
- 3.5. 5. The track is 33 to 68 mil designation thickness.
 6. Allowable in-plane shear strength of exempt anchors, parallel to the edge of concrete shall be permitted to be determined in accordance with AISI S100 Section E3.3.1.
 - 4. In light-frame construction, design of anchors in concrete shall be permitted to satisfy D.3.3.8.

D.3.3.6 - Instead of D.3.3.5, 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.3.3.4.

Exceptions:

- 1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.6.
- 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.3.3.6.

D.3.3.7 - As an alternative to D.3.3.5 and D.3.3.6, it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.4.

D.3.3.8 (f) In light-frame construction, bearing or nonbearing walls, shear strength of concrete anchors less than or equal to 1 inch [25 mm] in diameter of sill plate or track to foundation or foundation stem wall need not satisfy D.3.3.7 D.3.3.5.3 (a) through (c) when the design strength of the anchors is determined in accordance with D.6.2.1(c).

Reason: ACI 318-11 has made major modification to Appendix D. This proposed modification to section 1905.1.9 intends to maintain the well thought-out design provisions for sill bolts with minimum edge distance introduced into IBC 2012.

S208-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	5				1905.1.9-S-KERR.doc

S209–12 1905.1.9, 1905.1.10

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Revise as follows:

1905.1.9 ACI 318, Section D.3.3. Delete Modify ACI 318 Sections D.3.3.4 through D3.3.7 and replace D.3.3.5 by adding D.3.3.5.5 with the following as follows:

D.3.3.4 - The anchor design strength associated with concrete failure modes shall be taken as 0.75^a Nn and 0.75^a Vn, where ^a is given in D4.3 or D4.4 and Nn and Vn are deteremined in accordance with D5.2, D5.3, D5.4, D6.2 and D6.3, assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.

D.3.3.5 - 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.3.3.6 or D.3.3.7 is satisfied.

Exceptions:

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

D.3.3.5.5 – For shear parallel to an edge, the following exceptions are permitted:

- 2. (a). D.3.3.5 need not apply and the design shear strength in accordance with D.6.2.1(c) need not be computed for anchor bolts attaching wood sill plates of bearing or nonbearing walls of light-frame wood structures to foundations or foundation stem walls provided all of the following are satisfied:
 - 2.1. <u>1.</u> The allowable in-plane shear strength of the anchor is determined in accordance with AF&PA NDS Table 11E for lateral design values parallel to grain.
 - 2.2 2. The maximum anchor nominal diameter is 5/8 inches (16 mm).
 - 2.3.3. Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).
 - 2.4. <u>4.</u> Anchor bolts are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate.
 - 2.5. <u>5.</u> Anchor bolts are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.
 - 2.6. <u>6.</u> The sill plate is 2-inch or 3-inch nominal thickness.
- 3.(b). Section D.3.3.5 need not apply and the design shear strength in accordance with Section D.6.2.1(c) need not be computed for anchor bolts attaching cold-formed steel track of bearing or non-bearing walls of light-frame construction to foundations or foundation stem walls provided all of the following are satisfied:
 - 3.1. <u>1.</u> The maximum anchor nominal diameter is 5/8 inches (16 mm).
 - <u>3.2</u>. <u>2.</u> Anchors are embedded into concrete a minimum of 7 inches (178 mm).
 - 3.3. <u>3.</u> Anchors are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the track.
 - 3.4. <u>4.</u> Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.
 - 3.5. 5. The track is 33 to 68 mil designation thickness.

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

4 In light-frame construction, design of anchors in concrete shall be permitted to satisfy D.3.3.8.

D.3.3.6 - Instead of D.3.3.5, 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.3.3.4.

Exceptions:

- 1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.6.
- 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.3.3.6.

D.3.3.7 - As an alternative to D.3.3.5 and D.3.3.6, it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.4. D.3.3.8 - In light-frame construction, bearing or nonbearing walls, shear strength of concrete anchors less than or equal to 1 inch [25 mm] in diameter of sill plate or track to foundation or foundation stem wall need not satisfy D.3.3.7 when the design strength of the anchors is determined in accordance with D.6.2.1(c).

1905.1.10 ACI 318, Section D.4.2.2. Delete ACI 318, Section D.4.2.2, 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-7 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.

Reason: Appendix D had numerous changes in ACI 318-11, but the code change proposal process is ahead of the administrative update process in which ACI 318-11 was approved. Therefore, the exceptions to the 2012 IBC were based on ACI 318-08.

In the 2012 IBC, all of the changes to the ACI 318-11 seismic anchor provisions were deleted and the ACI 318-08 provisions were inserted with exceptions. ACI does not understand how this change occurred because this was not agreed to at the hearings. This code change proposal synchronizes IBC with ACI 318-11.

1905.1.9:

- Remove D.3.3.4 and D.3.3.5: These provisions were copied from ACI 318-08.
- Remove Exception 1 to D.3.3.5: Anchors with loads increased by Ω_o do not need to meet the requirements of D.3.3 as allowed in D.3.3.4.3(d) for tension and D3.3.5.3.(c) for shear in ACI 318-11.
- Revise Exceptions 2, 3, and 4 to D.3.3.5: There is no longer a 0.75 reduction for shear in seismic nor does the anchor need to, "be designed to be governed by the steel strength of a ductile steel element," for shear. These exceptions may be removed, but the exceptions as written do not require a concrete failure check according to D.6.2.1(c). Therefore, keep the items as exceptions for shear.
- Remove Exception 4 to D.3.3.5: There is no longer a 0.75 reduction for shear in seismic nor does the anchor need to "be designed to be governed by the steel strength of a ductile steel element" for shear.
- Remove D.3.3.6: This provision was copied from ACI 318-08.
- Remove Exception 1 and 2 to D.3.3.6: Anchors with loads increased by Ω_o do not need to meet the requirements of D.3.3 as allowed in D.3.3.4.3.(d) for tension and D3.3.5.3.(c) for shear in ACI 318-11.
- Remove D.3.3.7: This provision was copied from ACI 318-08.
- Revise D.3.3.8: See note about Exception 4 to D.3.3.5 above.

1905.1.10: This modification was made in the 2009 IBC in anticipation that ACI 318 was going to increase the limits. In ACI 318-11, the limitation on length was removed and the maximum diameter of the anchor was increased to 4 in. Therefore, the modification may now be removed.

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

S209-11

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1905.1.9-S-SENECAL

S210-12 1905.1.10 (NEW)

Proponent: Bonnie Manley, American Iron and Steel Institute (bmanley@steel.org)

Add new text as follows:

1905.1.10 ACI 318, Section D.3.3.4.4. Modify ACI 318, Section D.3.3.4.4 by adding the following exception:

D.3.3.4.4 — The anchor design tensile strength for resisting earthquake forces shall be determined from consideration of (a) through (e) for the failure modes given in Table D.4.1.1 assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked:

- (a) φ*Nsa* for a single anchor, or for the most highly stressed individual anchor in a group of anchors;
- (b) 0.75φNcb or 0.75φNcbg, except that Ncb or Ncbg need not be calculated where anchor reinforcement satisfying D.5.2.9 is provided;
- (c) 0.75φNpn for a single anchor, or for the most highly stressed individual anchor in a group of anchors;
- (d) 0.75φNsb or 0.75φNsbg; and
- (e) 0.75 \u03c6 Na or 0.75 \u03c6 Nag

where ϕ is in accordance with D.4.3 or D.4.4.

Exception:

- 1. The anchor design strength need not be reduced by the 0.75 factor for anchors in structural steel seismic force resisting systems designed in accordance with Section 2205, with the following restrictions:
 - a. Anchor rod has a minimum diameter of 3/4".
 - b. Anchor rod has a minimum embedment of 12".

c. <u>Concrete foundation elements receiving the anchor rods have the minimum</u> <u>reinforcement required in accordance with ACI 318, Chapters 7 and 10 located within</u> the upper half of the embedment depth of the anchor rods.

Reason: Section D2.6 of AISC 341-10 prescribes column anchorage required strengths based upon the maximum required strength of the structural steel members delivering the load to the anchorage. These forces are elevated to ensure that the column base has adequate strength to permit the expected ductile behavior for which the system was designed in order for the expected performance to be achieved. In recognition of these elevated design forces, the 0.75 strength reduction factor is redundant and not deemed necessary for cast-in-place anchors, since these anchors have performed well in past earthquakes. Conservatively, this exception is limited to cast-in-place anchors with a minimum diameter of 3/4", a minimum concrete embedment of 12", and minimum reinforcement for temperature and shrinkage crack control.

S210-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1905.1.10 (NEW)-S-MANLEY.doc

S211-12 1906.1

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Delete without substitution:

1906.1 Scope. The design and construction of structural plain concrete, both cast-in-place and precast, shall comply with the minimum requirements of ACI 318, as modified in Section 1905.

Exception: For Group R-3 occupancies and buildings of other occupancies less than two stories above grade plane of light-frame construction, the required 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.

Reason: This proposal removes the exception to the minimum footing thickness required in 22.7.4 in ACI 318-11. The exception reduces the minimum footing thickness from 8 in. to 6 in. The requirement for 8 in. has been in the ACI 318 code since 1941. The requirement for a 6 in. thickness has been in various model codes for a variety of exceptions over the past 40 to 50 years.

Recommend that the IBC accept the ACI 318 limit.

S211-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				1906.1-S-SENECAL

S212–12 1901.2, 1901.3 (NEW), 1907

Proponent: Matthew Senecal, P.E., representing American Concrete Institute (ACI)

Revise as follows:

1901.2 Plain and reinforced <u>Structural</u> concrete. <u>Plain and reinforced</u> structural concrete shall be designed and constructed in accordance with the requirements of this chapter and ACI 318 as amended in Section 1905 of this code. Except for the provisions of Sections 1904 and 1907, the design and construction of slabs on grade shall not be governed by this chapter unless they transmit vertical loads or lateral forces from other parts of the structure to the soil.

1901.3 Nonstructural concrete. Plain and reinforced nonstructural concrete shall be designed and constructed in accordance with the durability requirements of 1904. The thickness of slabs on ground supported directly on the ground shall not be less than 3.5 inches (89 mm).

Delete without substitution:

SECTION 1907 MINIMUM SLAB PROVISIONS

1907.1 General. The thickness of concrete floor slabs supported directly on the ground shall not be less than 31/2 inches (89 mm). A 6-mil (0.006 inch; 0.15 mm) polyethylene vapor retarder with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other *approved* equivalent methods or materials shall be used to retard vapor transmission through the floor slab.

Exception: A vapor retarder is not required:

- For detached structures accessory to occupancies in Group R-3, such as garages, utilitybuildings or other unheated facilities.
- 2. For unheated storage rooms having an area of less than 70 square feet (6.5 m2) and carports attached to occupancies in Group R-3.
- 3. For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.
- 4. For driveways, walks, patios and other flatwork which will not be enclosed at a later date.
- 5. Where *approved* based on local site conditions.

Reason: This code change proposal (1) removes a repetitive requirement given in ACI 318, (2) removes vapor barrier as a default requirement, and (3) condenses the remaining code requirements into a single provision.

(1) Repetitive requirement: Section 1.1.7 in ACI 318-11 states the following:

"This Code does not govern design and construction of slabs-on-ground, unless the slab transmits vertical loads or lateral forces from other portions of the structure to the soil."

In 1901.2, the phrase "the design and construction of slabs on grade shall not be governed by this chapter unless they transmit vertical loads or lateral forces from other parts of the structure to the soil" simply repeats the intent of ACI 318, 1.1.7.

(2) Vapor barrier: Requiring the vapor retarder to be directly under the slab is not always the best design. ACI 302.1R-04, Concrete Floor and Slab Construction, states the following related to concerns for placing concrete directly on a vapor retarder:

"Placing concrete in direct contact with the vapor retarder or barrier, however, requires additional consideration if potential slabrelated problems are to be avoided. When compared with identical concrete cast on a draining base, concrete placed in direct contact with a vapor retarder or barrier has been shown to exhibit significantly larger length change in the first hour after casting, during drying shrinkage, and when subject to environmental change; there is also more settlement (Suprenant 1997). Care should be taken in design detailing to minimize restraint to such movement (Anderson and Roper 1977). Where reinforcing steel is present, settlement cracking over the steel is more likely because of the increased settlement resulting from a longer bleeding period. The potential for a greater measure of slab curl is also increased."

Figure 3.1 from ACI 302.1R-04 (below) is a flow chart that describes when and where to place a vapor retarder.

Therefore, it is proposed to remove the requirement that, in all cases, a vapor barrier be placed between the base course or subgrade and the concrete floor slab. The Registered Design Professional should be given the responsibility to determine the need and location of the vapor retarder.



Fig. 3.1 – DECISION FLOW CHART TO DETERMINE IF A VAPOR RETARDER / BARRIER IS REQUIRED AND WHERE IT IS TO BE PLACED

NOTES:

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- (1) IF GRANULAR MATERIAL IS SUBJECT TO FUTURE MOISTURE INFILTRATION, USE FIG. 2.
- (2) IF FIGURE 2 IS USED, A REDUCED JOINT SPACING, A LOW SHRINKAGE MIX DESIGN, OR OTHER MEASURES TO MINIMIZE SLAB CURL WILL LIKELY BE REQUIRED.

(3) One code provision: If the two deletions are accepted above, the slab requirements may be reduced to an exception in 1901.2.

S212-11				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
5				1901 2-S-SENECAL

S213–12 Table 1705.3, 1908, 1908.1, 1908.2, Table 1908.2, 1908.3, 1908.4, 1908.5

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Revise as follows:

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE		
3. Inspection of anchors cast in concrete where allowable loads have been increased or where strength design is used.	_	х	ACI 318: 8.1.3, 21.2.8	1908.5, 1909.1		

TABLE 1705.3REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

(Portions of Table not shown remain unchanged)

Delete without substitution:

SECTION 1908 ANCHORAGE TO CONCRETE—ALLOWABLE STRESS DESIGN

1908.1 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. 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, the design strength of anchors shall be determined in accordance with Section 1909. Bolts shall conform to ASTM A 307 or an *approved* equivalent.

1908.2 Allowable service load. The allowable service load for headed anchors in shear or tension shall be as indicated in Table 1908.2. Where anchors are subject to combined shear and tension, the following relationship shall be satisfied:

(*Ps / Pt*)5/3 + (*Vs / Vt*) 5/3 ≤ 1 (Equation 19-1)

where:

Ps = Applied tension service load, pounds (N). Pt = Allowable tension service load from Table 1908.2, pounds (N). Vs = Applied shear service load, pounds (N). Vt = Allowable shear service load from Table 1908.2, pounds (N).

	ALLOWABLE SERVICE LOAD ON EMBEDDED BOLTS (pounds)									
BOLT	MINIMUM	EDGE	SDACING	MINIMUM CONCRETE STRENGTH (psi)						
DIAMETER	EMBEDMENT	DISTANCE	JPACING	_f _e ' = 2	,500	f _e ' = 3	,000	f _e ' = 4	,000	
(inches)	(inches)	(inches)	(menes)	Tension	Shear	Tension	Shear	Tension	Shear	
1/4	2-1/2	1-1/2	3	200	500	200	500	200	500	
3/8	3	2-1/4	4 -1/2	500	1,100	500	1,100	500	1,100	
1/2	4 4	3 5	6 6	950 1,450	1,250 1,600	950 1,500	1,250 1,650	950 1,550	1,250 1,750	

TABLE 1908.2 OWABLE SERVICE LOAD ON EMBEDDED BOLTS (pounds

BOLT	MINIMUM	EDGE	SDACING	M	IINIMUM	CONCRET	E STREN	IGTH (psi)	
DIAMETER	EMBEDMENT	DISTANCE	(inches)	-f _c ' = 2	2,500	f _e ' = 3	,000	$f_{\rm G}'=4$,000
(inches)	(inches)	(inches)	(monoo)	Tension	Shear	Tension	Shear	Tension	Shear
5/8	4- 1/2 4- 1/2	3-3/4 6-1/4	7-1/2 7-1/2	1,500 2,125	2,750 2,950	1,500 2,200	2,750 3,000	1,500 2,400	2,750 3,050
3/4	5 5	4 -1/2 7-1/2	9 8	2,250 2,825	3,250 4,275	2,250 2,950	3,560 4,300	2,250 3,200	3,560 4,400
7/8	6	5-1/4	10-1/2	2,550	3,700	2,550	4 ,050	2,550	4 ,050
4	7	6	12	3,050	4, 125	3,250	4 ,500	3,650	5,300
1-1/8	8	6-3/4	13-1/2	3,400	4,750	3,400	4 ,750	3,400	4,750
1-1/4	9	7-1/2	15	4,000	5,800	4,000	5,800	4,000	5,800

1908.3 Required edge distance and spacing. The allowable service loads in tension and shear specified in Table 1908.2 are for the edge distance and spacing specified. The edge distance and spacing are permitted to be reduced to 50 percent of the values specified with an equal reduction in allowable service load. Where edge distance and spacing are reduced less than 50 percent, the allowable service load shall be determined by linear interpolation.

1908.4 Increase in allowable load. Increase of the values in Table 1908.2 by one-third is permitted where the provisions of Section 1605.3.2 permit an increase in allowable stress for wind loading.

1908.5 Increase for special inspection. Where *special inspection* is provided for the installation of anchors, a 100 percent increase in the allowable tension values of Table 1908.2 is permitted. No increase in shear value is permitted.

Reason: This proposal removes allowable stress design for anchoring to concrete. This approach to anchor design is not consistent with the standards published by ACI, AISC, or ASCE

S213-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1908-S-SENECAL

S214-12 1908, 1909

Proponent: Stephen V. Skalko, Portland Cement Association

Delete without substitution:

SECTION 1908 ANCHORAGE TO CONCRETE—ALLOWABLE STRESS DESIGN

1908.1 Scope.

1908.2 Allowable service load.

1908.3 Required edge distance and spacing.

1908.4 Increase in allowable load.

1908.5 Increase for special inspection.

SECTION 1909 ANCHORAGE TO CONCRETE—STRENGTH DESIGN

1909.1 Scope.

Reason: There are three main reasons in support of this code change to modify Chapter 19 "Concrete" of the *International Building Code* (IBC):

- 1. The proposal removes redundancies for anchorage between ACI 318 and the IBC. Such redundancies are not necessary and may be detrimental in that they may cause confusion and result in errors. The introductory section of Chapter 19, "1901.2 Plain and reinforced concrete" adequately and appropriately requires compliance with ACI 318.
- ACI 318 addresses the criteria for anchorage in a more complete approach. Sections 1908 and 1909 instruct the user that when allowable stress design or strength design are used for anchorage to concrete the other criteria and requirements in ACI 318 are no longer required, including but not limited to break out and types and amount of anchor reinforcement.
- 3. If the chapters and sections in whatever edition of ACI 318 that becomes referenced in the 2015 edition of the IBC are not properly coordinated, there will be confusion and will increase the potential for errors in design and construction.

S214-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				1908-S-SKALKO.doc

S215–12 Table 1705.3, 1901.3 (NEW), 1909

Proponent: Matthew Senecal, P.E, American Concrete Institute (ACI)

Revise as follows:

REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION							
VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE			
3. Inspection of anchors cast in concrete where allowable loads have been increased or where strength design is used.	_	Х	ACI 318: <u>D.9.2</u> 8.1.3, 21.1.8	1908.5, 1909.1			
 Inspection of anchors post- installed in hardened concrete members^b-: 	_	×	ACI 318: - <u>3.8.6, 8.1.3,</u> 	1909.1			
<u>a. Adhesive anchors</u> installed in horizontally or upwardly inclined orientations to resist sustained tension loads	X		ACI 318: D.9.2.4	—			
<u>b. Mechanical anchors and</u> adhesive anchors not defined in 4.a.		<u>×</u>	ACI 318: D.9.2	_			

TABLE 1705.3 REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

b. Specific requirements for special inspection shall be included in the research report for the anchor issued by an approved source in accordance with ACI 355.2 D.9.2 in ACI 318, or other qualification procedures. Where specific requirements are not provided, special inspection requirements shall be specified by the registered design professional and shall be approved by the building official prior to the commencement of the work.

(Portions of table not shown remain unchanged)

1901.3 Anchoring to concrete. Anchoring to concrete shall be in accordance with ACI 318 as amended in Section 1905, and applies to cast-in (headed bolts, headed studs, and hooked J- or L-bolts) anchors and post-installed expansion (torque-controlled and displacement-controlled), undercut, and adhesive anchors.

Delete without substitution:

SECTION 1909 ANCHORAGE TO CONCRETE—STRENGTH DESIGN

1909.1 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 Appendix D of ACI 318 as modified by Sections 1905.1.9 and 1905.1.10, 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 1905.1.9 and 1905.1.10, shall be in accordance with an *approved* procedure.

Reason: Requirements for the design and installation of adhesive anchors was included in ACI 318-11. Requirements for continuous inspection were added for adhesive anchors installed horizontally or in upwardly inclined orientations with sustained loads.

The difficulty of installing adhesive anchors greatly increases when gravity works to drain the placed epoxy out of the predrilled hole. For consistent installation, trained personnel are essential.

Under sustained tension loads, epoxy will creep and debond as evidenced by the epoxy anchors that supported the ceiling panels in the I-90 connector tunnel in Boston. A proper installation is critical in this case and requires continuous inspection.

In the interest of writing concise code language, recommend deleting this section 1909 and providing a general requirement just after 1901.2, "Plain and reinforced concrete."

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

S215-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				1909-S-SENECAL

S216–12 Table 1705.3, 1901.2.1 (NEW), 1910, 1910.1, 1910.2, 1910.3, 1910.4, 1910.5, 1910.6, 1910.7, 1910.8, 1910.9, 1910.9.1, 1910.9.2, 1910.9.3, 1910.10, 1910.11, 1910.12, 1910.13

Proponent: Matthew Senecal, P.E., American Concrete Association (ACI) and Christopher Darnell, American Shotcrete Association

Revise as follows:

TABLE 1705.3 REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
1. Inspection of reinforcing steel, including prestressing tendons, and placement.	_	х	ACI 318: 3.5, 7.1-7.7	1910.4

 Verifying use of required design mix. 	—	х	ACI 318: Ch. 4, 5.2-5.4	1904.2 , 1910.2, 1910.3
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.	Х	l	ASTM C 172 ASTM C 31 ACI 318: 5.6, 5.8	1910.10
7. Inspection of concrete and shotcrete placement for proper application techniques.	Х		ACI 318: 5.9, 5.10	1910.6, 1910.7, 1910.8
8. Inspection for maintenance of specified curing temperature and techniques.	_	Х	ACI 318: 5.11- 5.13	1910.9

(Portions of table not shown remain unchanged)

1901.2.1 Shotcrete. Shotcrete is mortar or concrete that is pneumatically projected at high velocity onto a surface. Shotcrete shall conform to the requirements of this chapter for plain or reinforced concrete, as applicable.

Delete without substitution:

SECTION 1910 SHOTCRETE

1910.1 General. Shotcrete is mortar or concrete that is pneumatically projected at high velocity onto a surface. Except as specified in this section, shotcrete shall conform to the requirements of this chapter for plain or reinforced concrete.

1910.2 Proportions and materials. Shotcrete proportions shall be selected that allow suitable placement procedures using the delivery equipment selected and shall result in finished in-place hardened shotcrete meeting the strength requirements of this code.

1910.3 Aggregate. Coarse aggregate, if used, shall not exceed 3/4 inch (19.1 mm).

1910.4 Reinforcement. Reinforcement used in shotcrete construction shall comply with the provisions of Sections 1910.4.1 through 1910.4.4.

1910.4.1 Size. The maximum size of reinforcement shall be No. 5 bars unless it is demonstrated by preconstruction tests that adequate encasement of larger bars will be achieved.

1910.4.2 Clearance. When No. 5 or smaller bars are used, there shall be a minimum clearance between parallel reinforcement bars of 21/2 inches (64 mm). When bars larger than No. 5 are permitted, there shall be a minimum clearance between parallel bars equal to six diameters of the bars used. When two curtains of steel are provided, the curtain nearer the nozzle shall have a minimum spacing equal to 12 bar diameters and the remaining curtain shall have a minimum spacing of six bar diameters.

Exception: Subject to the approval of the *building official*, required clearances shall be reduced where it is demonstrated by preconstruction tests that adequate encasement of the bars used in the design will be achieved.

1910.4.3 Splices. Lap splices of reinforcing bars shall utilize the noncontact lap splice method with a minimum clearance of 2 inches (51 mm) between bars. The use of contact lap splices necessary for support of the reinforcing is permitted when *approved* by the *building official*, based on satisfactory preconstruction tests that show that adequate encasement of the bars will be achieved, and provided that the splice is oriented so that a plane through the center of the spliced bars is perpendicular to the surface of the shotcrete.

1910.4.4 Spirally tied columns. Shotcrete shall not be applied to spirally tied columns.

1910.5 Preconstruction tests. When required by the *building official*, a test panel shall be shot, cured, cored or sawn, examined and tested prior to commencement of the project. The sample panel shall be representative of the project and simulate job conditions as closely as possible. The panel thickness and reinforcing shall reproduce the thickest and most congested area specified in the structural design. It shall be shot at the same angle, using the same nozzleman and with the same concrete mix design that will be used on the project. The equipment used in preconstruction testing shall be the same equipment used in the work requiring such testing, unless substitute equipment is *approved* by the *building official*.

1910.6 Rebound. Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be used as aggregate.

1910.7 Joints. Except where permitted herein, unfinished work shall not be allowed to stand for more than 30 minutes unless edges are sloped to a thin edge. For structural elements that will be under compression and for construction joints shown on the *approved construction documents*, square joints are permitted. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

1910.8 Damage. In-place shotcrete that exhibits sags, sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced. Shotcrete above sags and sloughs shall be removed and replaced while still plastic.

1910.9 Curing. During the curing periods specified herein, shotcrete shall be maintained above 40°F (4°C) and in moist condition.

1910.9.1 Initial curing. Shotcrete shall be kept continuously moist for 24 hours after shotcreting is complete or shall be sealed with an *approved* curing compound.

1910.9.2 Final curing. Final curing shall continue for seven days after shotcreting, or for three days if highearly-strength cement is used, or until the specified strength is obtained. Final curing shall consist of the initial curing process or the shotcrete shall be covered with an *approved* moisture-retaining cover.

1910.9.3 Natural curing. Natural curing shall not be used in lieu of that specified in this section unless the relative humidity remains at or above 85 percent, and is authorized by the *registered design* professional and approved by the building official.

1910.10 Strength tests. Strength tests for shotcrete shall be made by an *approved agency* on specimens that are representative of the work and which have been water soaked for at least 24 hours prior to testing. When the maximum-size aggregate is larger than 3/8 inch (9.5 mm), specimens shall consist of not less than three 3-inch-diameter (76 mm) cores or 3-inch (76 mm) cubes. When the maximum-size aggregate is 3/8 inch (9.5 mm) or smaller, specimens shall consist of not less than 2-inch-diameter (51 mm) cores or 2-inch (51 mm) cubes.

1910.10.1 Sampling. Specimens shall be taken from the in-place work or from test panels, and shall be taken at least once each shift, but not less than one for each 50 cubic yards (38.2 m3) of shotcrete.

1910.10.2 Panel criteria. When the maximum-size aggregate is larger than 3/8 inch (9.5 mm), the test panels shall have minimum dimensions of 18 inches by 18 inches (457 mm by 457 mm). When the maximum size aggregate is 3/8 inch (9.5 mm) or smaller, the test panels shall have minimum dimensions of 12 inches by 12 inches (305 mm by 305 mm). Panels shall be shot in the same position as the work, during the course of the work and by the nozzlemen doing the work. The conditions under which the panels are cured shall be the same as the work.

1910.10.3 Acceptance criteria. The average compressive strength of three cores from the in-place work or a single test panel shall equal or exceed 0.85 *f* '*c* with no single core less than 0.75 f_c '. The average compressive strength of three cubes taken from the in-place work or a single test panel shall equal or exceed f_c ' with no individual cube less than 0.88 f_c '. To check accuracy, locations represented by erratic core or cube strengths shall be retested.

Reason: Shotcrete is a concrete placement method that is specified by a registered design professional in a set of contract documents. ACI 318 covers the essential information necessary for the design and construction of concrete structures using shotcrete. The requirements currently in the IBC are outdated construction specifications.

ACI 506.2, Specification for Shotcrete, provides a detailed set of construction requirements that represents the current standard of practice. The specification contains quality assurance requirements, 35 referenced standard test methods and material specifications, placement requirements, and acceptance criteria. ACI 506.2 is not being submitted as a reference standard since construction specifications should not be in the building code.

S216-11				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1910-S-SENECAL

S217–12 1911, 2501.1.1, 2514 (NEW)

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI) and Michael Gardner, Gypsum Association

Revise as follows:

0047 44

SECTION 1911 2514 REINFORCED GYPSUM CONCRETE

1911.1 2514.1 General. Reinforced gypsum concrete shall comply with the requirements of ASTM C 317 and ASTM C 956.

1911.2 <u>2514.2</u> Minimum thickness. The minimum thickness of reinforced gypsum concrete shall be 2 inches (51 mm) except the minimum required thickness shall be reduced to 11/2 inches (38 mm), provided the following conditions are satisfied:

- 1. The overall thickness, including the formboard, is not less than 2 inches (51 mm).
- 2. The clear span of the gypsum concrete between supports does not exceed 33 inches (838 mm).
- 3. Diaphragm action is not required.
- 4. The design live load does not exceed 40 pounds per square foot (psf) (1915 Pa).

2501.1.1 General. Provisions of this chapter shall govern the materials, design, construction and quality of gypsum board, lath, gypsum plaster and cement plaster, and reinforced gypsum concrete.

Reason: The design and construction of gypsum concrete roof decks and slabs are governed by ASTM C317 and ASTM C956. The product is gypsum-based and maintained by the ASTM C 11 Gypsum Products group; thus, making it more appropriate for inclusion in Chapter 25.

5217-11				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1911-S-SENECAL

S218–12 1901.3 (NEW), 1912

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Revise as follows:

1901.3 Composite structural steel and concrete structures. Systems of structural steel acting compositely with reinforced concrete shall be designed in accordance with Section 2206 of this code.

SECTION 1912 CONCRETE-FILLED PIPE COLUMNS

1912.1 General. Concrete filled pipe columns shall be manufactured from standard, extra-strong or double-extra-strong steel pipe or tubing that is filled with concrete so placed and manipulated as to secure maximum density and to ensure complete filling of the pipe without voids.

1912.2 Design. The safe supporting capacity of concrete filled pipe columns shall be computed in accordance with the *approved* rules or as determined by a test.

1912.3 Connections. Caps, base plates and connections shall be of *approved* types and shall be positively attached to the shell and anchored to the concrete core. Welding of brackets without mechanical anchorage shall be prohibited. Where the pipe is slotted to accommodate webs of brackets or other connections, the integrity of the shell shall be restored by welding to ensure hooping action of the composite section.

1912.4 Reinforcement. To increase the safe load-supporting capacity of concrete-filled pipe columns, the steel reinforcement shall be in the form of rods, structural shapes or pipe embedded in the concrete core with sufficient clearance to ensure the composite action of the section, but not nearer than 1 inch (25 mm) to the exterior steel shell. Structural shapes used as reinforcement shall be milled to ensure bearing on cap and base plates.

1912.5 Fire-resistance-rating protection. Pipe columns shall be of such size or so protected as to develop the required fire-resistance ratings specified in Table 601. Where an outer steel shell is used to enclose the fire protective covering, the shell shall not be included in the calculations for strength of the column section. The minimum diameter of pipe columns shall be 4 inches (102 mm) except that in structures of Type V construction not exceeding three *stories above grade plane* or 40 feet (12 192 mm) in *building height*, pipe columns used in basements and as secondary steel members shall have a minimum diameter of 3 inches (76 mm).

1912.6 Approvals. Details of column connections and splices shall be shop fabricated by *approved* methods and shall be *approved* only after tests in accordance with the *approved* rules. Shop-fabricated concrete-filled pipe columns shall be inspected by the *building official* or by an *approved* representative of the manufacturer at the plant.

Reason: The design and construction of concrete-filled pipe columns is covered in the reference standards stated in 2206 of this code. The requirements above are not complete nor have they been maintained. Recommend adding a general statement that directs the user to the appropriate section.

S218-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				1912-S-SENECAL

S219–12 202, 2101.2, 2101.2.1, 2101.2.2, 2101.2.3, 2101.2.4, 2101.2.5, 2101.2.7, 2101.3, 2101.3.1, 2102.1, 2111.2

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards (jthompson@ncma.org), Phil Samblanet, Masonry Alliance for Codes and Standards, representing The Masonry Society (psamblanet@masonrysociety.org)

Delete without substitution:

SECTION 202 DEFINITIONS

ANCHOR. Metal rod, wire or strap that secures masonry to its structural support.

Revise as follows:

2101.2 Design methods. Masonry shall comply with the provisions of one of the following design methods in this chapter <u>TMS 402/ACI 530/ASCE 5 or TMS 403</u> 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, the prestressed masonry provisions of Section 2101.2.3, or the direct design requirements of Section 2101.2.7 shall comply with Section 2105 <u>applicable requirements of this chapter</u>.

2101.2.1 Allowable stress design. Masonry designed by the *allowable stress design* method shall comply with the provisions of Sections 2106 and 2107.

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

2101.2.3 Prestressed masonry. Prestressed masonry shall be designed in accordance with Chapters 1 and 4 of TMS 402/ACI 530/ASCE 5 and Section 2106. *Special inspection* during construction shall be provided as set forth in Section 1705.4.

2101.2.4 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of Sections 2106 and 2109 or Chapter 5 of TMS 402/ACI 530/ASCE 5.

2101.2.5 Glass unit masonry. Glass unit masonry shall comply with the provisions of Section 2110 or Chapter 7 of TMS 402/ACI 530/ASCE 5.

2101.2.6 2101.2.1 Masonry veneer. Masonry veneer shall comply with the provisions of Chapter 14 or Chapter 6 of TMS 402/ACI 530/ASCE 5.

2101.2.7 Direct design. Masonry designed by the direct design method shall comply with the provisions of TMS 403.

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 permitted 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.

2101.3 Special Inspection. The special inspection of masonry shall be as defined in Chapter 17, or an itemized testing and inspection program shall be provided that meets or exceeds the requirements of Chapter 17.

2101.3.1 <u>2111.2</u> Fireplace drawings. The *construction documents* shall describe in sufficient detail the location, size and construction of masonry fireplaces. The thickness and characteristics of materials and the clearances from walls, partitions and ceilings shall be indicated.

2102.1 General. For the purposes of this chapter and as used elsewhere in this code, the following terms are defined in Chapter 2:

ANCHOR.

Reason: Section 2101 provides a series of pointers to specific sections of the IBC as well as the referenced masonry standards that, due largely to the evolution of Chapter 21 over time, has become a source of confusion. In addition, the 2013 edition of TMS 402 standard has been substantially reorganized to be more user friendly; requiring in turn that a number of the Chapters and Sections referenced in TMS 402 be updated. Instead of updating these pointers, this change proposal simply consolidates the charging language of Section 2101. No technical change is intended or implied. Specific discussion related to this change: 1) The reference to Chapter 14 for masonry veneers is maintained as Chapter 14 addresses some types of masonry veneer not covered by the reference standard (for example, anchored stone veneer). Chapter 14 already contains a reference to Chapter 6 of the reference standard.

2) The construction document requirements of Section 2101.3 are virtually identical to the requirements of Section 1.2.2 of TMS 402 and are therefore proposed to be deleted.

3) Although somewhat redundant, a reference to Chapter 17 for special inspection is maintained as a new Section 2101.3 to reinforce compliance with these requirements.

4) Section 2101.3.1 for fireplace drawings is relocated to Section 2111.2, which covers requirements specific to fireplaces.
5) While the term anchor (or anchorage) is used generically throughout the IBC for all types of building materials, this term (as applied specifically to masonry construction) is used only in Section 2101.3, which is proposed for deletion. As such, the IBC definition is proposed for deletion as well. The definition of 'anchor' in TMS 402 is identical to the IBC definition.

S219-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2101-S-SAMBLANET-THOMPSON.doc

S220-12

Proponent: Phillip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

CLEANOUT (for Chapter 21). An opening to the bottom of a grout space of sufficient size and spacing to allow the removal of debris.

DIMENSIONS (for Chapter 21).

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. Dimensions specified for the manufacture or construction of a unit, joint or element.

EXISTING CONSTRUCTION. Any buildings and structures for which the *start of construction* commenced before the effective date of the community's first flood plain management code, ordinance or standard. "Existing construction" is also referred to as "existing structures."

EXISTING STRUCTURE (For Section 1612.2). See "Existing construction".

FOUNDATION PIER (for Chapter 21). An isolated vertical foundation member whose horizontal dimension measured at right angles to its thickness does not exceed three times its thickness and whose height is equal to or less than four times its thickness.

OTHER STRUCTURES (for Chapters 16-23). Structures, other than buildings, for which *loads* are specified in Chapter 16.

WALL (for Chapter 21). A vertical element with a horizontal length-to-thickness ratio greater than three, used to enclose space.

Cavity wall. A wall 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 wall. 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 multi-*wythe* wall built of *masonry units* arranged to provide an air space between the *wythes* and with the *wythes* bonded together with *masonry units*. **Parapet wall.** The part of any wall entirely above the roof line.

Reason: The purpose for this proposal is to adjust the definitions in Section 202 to (1) clarify their purpose and (2) to correct errors from approved changes in previous ICC code development cycles that were not made in the building code.

Adding "for Chapter 21" to the definitions of "cleanout," "dimensions," "foundation pier" and "wall" is done to reduce their applicability to what is their intended purpose, namely the structural provisions for masonry in Chapter 21. The terms are sufficiently common in use to justify this action and will make them consistent with the definitions of "area," "cell," "shear wall" and "strength," which are identified in a similar manner.

Adding "for Chapter 21" to the definition of "wall" is also done because of ICC Proposal FS85-07/08-AS, Part II, which added to Section 2102.1 after the definition of "wall" the following: "The definition of 'wall' is limited in application to the provisions of Chapter 21." I was the proponent of this proposal and I requested that this be posted as errata but a posting did not occur nor was the language incorporated into later printings of the 2009 IBC or into the 2012 IBC.

The definitions of "existing construction" and "existing structure" are being deleted because they serve no purpose in the building code. There are no instances of "existing construction" in the 2012 IBC other than as shown in this proposal. There are numerous

instances of "existing structure" in the 2012 IBC but there are none in Section 1612 and the definition of "existing structure" is limited to that section as specified in Section 202.

The addition of "for Chapters 16-23" to the definition of "other structures" is because of the use of the term in other sections of the building code (e.g., Sections 402.6.2, 424.3 and 3102.1). The source document for the term is ASCE 7 and it was in 2009 IBC Section 1602.1.

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

S220-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				1612.2-S-BRAZIL.doc

S221–12 2103.15 (NEW), 2103.15.1 (NEW), 2103.15.2 (NEW), 2103.15.3 (NEW), 2104.5 (NEW), Chapter 35 (NEW)

Proponent: John Mulder, Intertek Testing Services NA, Inc., representing International Standards Organization Technical Committee 77, *Products in Fibre-reinforced Cement*

Add new text as follows:

2103.15 Fiber-cement backer board and underlayment. Fiber-cement backer board and underlayment shall conform to the requirements of Section 2103.15.1, 2103.15.2, or 2103.15.3, and shall be so identified on labeling listing an approved quality control agency.

2103.15.1 Fiber-cement backer board. Fiber-cement backer board complying with either ASTM C1288 or ISO 8336, Category B or C, is a suitable *backing* for decoration with paint, wallpaper, resilient flooring, tile, natural stone, or dimensioned stone *veneer* on floors, walls, and ceilings in interior dry areas; and for interior use in wet areas of walls and ceilings as permitted in Section 2509.2.

2103.15.2 Fiber-cement underlayment. Fiber-cement underlayment complying with either ASTM C1288 or ISO 8336, Category B or C, is a suitable *backing* for decoration with resilient flooring, tile, natural stone, or dimensioned stone *veneer* in interior wet or dry areas.

2103.15.3 Fiber-cement backer board. Fiber-cement backer board complying with ISO 8336, Category A, is a suitable *backing* for decoration with tile, natural stone, or dimensioned stone *veneer* on exterior walls.

2104.5 Fiber-cement backer board and underlayment construction. Fiber-cement backer board and underlayment complying with Section 2103.15.1, 2103.15.2, or 2103.15.3, shall be installed in accordance with approved manufacturer's instructions.

Add new standard to Chapter 35 as follows:

ISO

ISO 8336 Fibre-cement flat sheets -- Product Specification and Test Methods

Reason: Fiber-cement backer board and underlayment products are cement-based masonry-type products currently recognized for use through ICC-ES evaluation reports (see attached ESR-1381[reference Sections 2, 3 and 4.3], ESR-2280[reference Sections 2, 3 and 4.3], and ESR-2292[reference Sections 2, 3 and 4.2]). The inclusion in this Code Section confirms their currently recognized use as a base for tile setting materials also included in Chapter 21 of the Code. Fiber-cement backer board and underlayment products are masonry-type products currently recognized for use through ICC-ES evaluation reports (see attached ESR-1381[reference Sections 2.0, 4.3], ESR-2280[reference Section 4.2], and ESR-2292[reference Section 4.2]). The new reference here provides construction guidance. "See the ICC-ES website (http://www.icc-es.org/) to gain access to the referenced ESR reports. "

Cost Impact: The code change proposal will not increase the cost of construction because the proposed addition of fiber-cement backer board or underlayment products only provides for the choice and use of a type of backer board or underlayment product currently recognized through evaluation reports for use in accordance with the Code.

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S221-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2103 (NEW)-S-MULDER.doc

S222–12 202, 2102.1, 2103.1, 2103.2, 2103.3, 2103.4, 2103.5, 2103.6, 2103.7, 2103.8, 2103.9, 2103.12, 2103.13, 2103.14

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards, (jthompson@nema.org), Phil Samblanet, Masonry Alliance for Codes and Standards, representing The Masonry Society (psamblanet@masonrysociety.org)

Delete without substitution:

SECTION 202 DEFINITIONS

THIN-BED MORTAR. *Mortar* for use in construction of AAC unit masonry with *joints* 0.06 inch (1.5 mm) or less.

Revise as follows:

2102.1 General. For the purposes of this chapter and as used elsewhere in this code, the following terms are defined in Chapter 2:

THIN-BED MORTAR.

2103.1 Masonry units. Concrete masonry units, clay or shale masonry units, stone masonry units, glass unit masonry, and AAC masonry units shall comply with Article 2.3 of TMS 602/ACI 530.1/ASCE 6. Architectural cast stone shall conform to ASTM C1364.

2103.1 Concrete masonry units. Concrete masonry units shall conform to the following standards: ASTM C 55 for concrete brick; ASTM C 73 for calcium silicate face brick; ASTM C 90 for load-bearing concrete masonry units or ASTM C 744 for prefaced concrete and calcium silicate masonry units.

2103.2 Clay or shale masonry units. Clay or shale masonry units shall conform to the following standards: ASTM C 34 for structural clay *load-bearing wall* tile; ASTM C 56 for structural clay nonload-bearing wall tile; ASTM C 62 for building brick (solid masonry units made from clay or shale); ASTM C 1088 for solid units of thin veneer brick; ASTM C 126 for ceramic-glazed structural clay facing tile, facing brick and solid masonry units; ASTM C 212 for structural clay facing tile; ASTM C 216 for facing brick (solid masonry units made from clay or shale); ASTM C for structural clay facing tile; ASTM C 216 for facing brick (solid masonry units made from clay or shale); ASTM C 126 for structural clay facing tile; ASTM C 216 for facing brick (solid masonry units made from clay or shale); ASTM C 652 for hollow brick (hollow masonry units made from clay or shale) or ASTM C 1405 for glazed brick (single-fired solid brick units).

Exception: Structural clay tile for nonstructural use in fireproofing of structural members and in wall furring shall not be required to meet the compressive strength specifications. The fire-resistance rating shall be determined in accordance with ASTM E 119 or UL 263 and shall comply with the requirements of Table 602.

2103.3 AAC masonry. AAC masonry units shall conform to ASTM C 1386 for the strength class specified.

2103.4 Stone masonry units. Stone masonry units shall conform to the following standards: ASTM C 503 for marble building stone (exterior); ASTM C 568 for limestone building stone; ASTM C 615 for granite building stone; ASTM C 616 for sandstone building stone; or ASTM C 629 for slate building stone.

2103.5 Architectural cast stone. Architectural cast stone shall conform to ASTM C 1364.

2103.6 Ceramic tile. Ceramic tile shall be as defined in, and shall conform to the requirements of, ANSI A137.1.

2103.7 Glass unit masonry. Hollow glass units shall be partially evacuated and have a minimum average glass face thickness of 3/16 inch (4.8 mm). Solid glass-block units shall be provided when required. The surfaces of units intended to be in contact with mortar shall be treated with a polyvinyl butyral coating or latex-based paint. Reclaimed units shall not be used.

2103.8 <u>**2103.1.1**</u> **Second-hand units.** Second-hand masonry units shall not be reused unless they conform to the requirements of new units. The units shall be of whole, sound materials and free from cracks and other defects that will interfere with proper laying or use. Old mortar shall be cleaned from the unit before reuse.

2103.9 Mortar. Mortar for use in masonry construction shall conform to ASTM C 270 and Articles 2.1 and 2.6 A of TMS 602/ACI 530.1/ASCE 6, except for mortars listed in Sections 2103.10, 2103.11 and 2103.12. Type S or N mortar conforming to ASTM C 270 shall be used for glass unit masonry.

2103.2 Mortar. Mortar for masonry construction shall comply with Section 2103.2.1, 2103.2.2, or 2103.2.3.

2103.2.1 Masonry mortar. Mortar for use in masonry construction shall conform to Articles 2.1 and 2.6 A of TMS 602/ACI 530.1/ASCE 6.

2103.10 2103.2.2 Surface-bonding mortar. Surface-bonding mortar shall comply with ASTM C 887. Surface bonding of concrete masonry units shall comply with ASTM C 946.

2103.11 2103.2.3 Mortars for ceramic wall and floor tile. Portland cement mortars for installing ceramic wall and floor tile shall comply with ANSI A108.1A and ANSI A108.1B and be of the compositions indicated in Table 2103.11.

2103.12 Mortar for AAC masonry. Thin-bed mortar for AAC masonry shall comply with Article 2.1 C.1 of TMS 602/ ACI 530.1/ASCE 6. Mortar used for the leveling courses of AAC masonry shall comply with Article 2.1 C.2 of TMS 602/ ACI 530.1/ASCE 6.

2103.13 2103.3 Grout. Grout shall comply with Article 2.2 of TMS 602/ACI 530.1/ASCE 6.

2103.14 <u>2103.4</u> Metal reinforcement and accessories. Metal reinforcement and accessories shall conform to Article 2.4 of TMS 602/ACI 530.1/ASCE 6. Where unidentified reinforcement is *approved* for use, not less than three tension and three bending tests shall be made on representative specimens of the reinforcement from each shipment and grade of reinforcing steel proposed for use in the work.

Reason: The modifications proposed here simply consolidate the material requirements of Section 2103 by referencing the appropriate articles in TMS 602 instead of transcribing these provisions into the IBC. No substantive change is intended or implied. Some provisions are maintained in Section 2103 as they are not addressed by TMS 602. These include: architectural cast stone meeting ASTM C1364, compressive strength exemptions for structural clay tile used as fireproofing, second-hand units, surface-bonding mortar, mortars for tile, and testing of unidentified reinforcement and accessories.

S222-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					2103-S-SAMBLANET-THOMPSON doc

S223–12 2103.1, Chapter 35 (NEW)

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards (jthompson@nema.org)

Revise as follows:

2103.1 Concrete masonry units. Concrete masonry units shall conform to the following standards: ASTM C 55 for concrete brick; ASTM C 73 for calcium silicate face brick; ASTM C 90 for load-bearing concrete masonry units or ASTM C 744 for prefaced concrete and calcium silicate masonry units <u>or ASTM C1634 for concrete facing brick</u>.

Add new standard to Chapter 35 as follows:

ASTM

C1634-11, Specification for Concrete Facing Brick

Reason: The change proposed to introduce a reference to ASTM C1634 that addresses the minimum requirements for concrete facing brick. Concrete facing brick manufactured to comply with ASTM C1634 are similar in nature to concrete brick manufactured to comply with ASTM C55, with a few notable exceptions as explained in a non-mandatory note contained in ASTM C1634:

NOTE 1—Specification C55 addresses concrete building brick used in non-facing, utilitarian applications (previously referred to in earlier editions of Specification C55 as Grade S—for general use where moderate strength and resistance to frost action and moisture penetration are required). This specification differs from Specification C55 in that it includes expanded consideration for properties of concrete brick used in facing applications and other exposures (previously referred to in earlier editions of Specification C55 as Grade N—for use as architectural veneer and facing units in exterior walls and for use where high strength and resistance to moisture penetration and severe frost action are desired).

Due to the intended applications of C1634 unit, the physical requirements contained in ASTM C1634 are more stringent relative to those of ASTM C55. For example, the average compressive strength of ASTM C55 brick is 2,500 psi; whereas ASTM C1634 required a minimum compressive strength of 3,500 psi. Similarly the maximum absorption requirements of ASTM C1634 are less than the corresponding requirements of ASTM C55.

Historically, the physical requirements for 'utility brick' and 'facing brick' were covered together within ASTM C55 (as Grade S and Grade N brick, respectively); albeit with unique requirements for each. This often resulted in confusion in the field when a specification simply cited "ASTM C55", as it was not clear if Grade S or Grade N brick were intended. As a result, the minimum physical requirements for 'utility brick' and 'facing brick' were separated into their own unique standards.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S223-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2103.1-S-THOMPSON.doc

S224–12 2103.15 (NEW), Chapter 35 (NEW)

Proponent: James K. Hicks, P.E., CeraTech, Inc., representing self (jim.hicks@ceratechinc.com)

Add new text as follows:

2103.15 Rapid Hardening Cement. Rapid hardening hydraulic cement shall conform to ASTM C1600,

Add new standard to Chapter 35 as follows:

ASTM

C1600-11 Standard Specification for Rapid Hardening Hydraulic Cement

Reason: For those instances wherein rapid hardening is desired, cements conforming to ASTM C 1600 Standard Specification for Rapid hardening Hydraulic Cements are useable. ASTM C 1600 can be one of four cement types, General Rapid Hardening (GRH), Moderate Rapid Hardening (MRH), Very Rapid Hardening (VRH) and Ultra Rapid Hardening (URH).

C 1600 is a Specification giving numerous performance requirements. Primary characteristics (with inherent increased design flexibility) are:

•Can produce rapid-hardening concrete, precast concrete, block, mortar and grout.

•Depending on the type cement used and the specific mixture, cements meeting ASTM C 1600 can provide normal, medium or fast time to service (1.5 to 48 h)

•ASTM C 1600 has rigid durability requirements.

ASTM C 1600 cements are used in products such as:

- Materials for Concrete Repairs
- •High Strength Grouts

•Precast •Paving

•Some Cements - Mass Concrete

•Some Cements – Heat Resistant

•Some Cements – Chemical Resistant

In addition to following pertinent ACI and ASTM requirements, users of C 1600 cements must heed manufacturers instructions for use. Specific durability aspects of any given mortar or concrete should be evaluated by the appropriate test method(s).

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S224-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2103.15 (NEW)-S-HICKS.doc

S225–12 2104.1, 2104.1.1, 2104.1.2, 3104.1.3, 2104.1.4, 2104.1.5, 2104.1.6, 2104.2, 2104.2.1, 2104.3, 2104.4

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards (jthompson@nema.org), Phil Samblanet, The Masonry Society, representing The Masonry Society (psamblanet@masonrysociety.org)

Revise as follows:

2104.1 Masonry construction. Masonry construction shall comply with the requirements of Sections 2104.1.1 through 2104.4.4 2104.1.1, 2104.1.2 and with TMS 602/ACI 530.1/ASCE 6.

2104.1.1 Tolerances. Masonry, except masonry veneer, shall be constructed within the tolerances specified in TMS 602/ACI 530.1/ASCE 6.

2104.1.2 Placing mortar and units. Placement of mortar, grout, and clay, concrete, glass, and AAC masonry units shall comply with TMS 602/ACI 530.1/ASCE 6.

2104.1.3 Installation of wall ties. Wall ties shall be installed in accordance with TMS 602/ACI 530.1/ASCE 6.

2104.1.4 Chases and recesses. Chases and recesses shall be constructed as masonry units are laid. Masonry directly above chases or recesses wider than 12 inches (305 mm) shall be supported on lintels.

2104.1.5 Lintels. The design for lintels shall be in accordance with the masonry design provisions of either Section 2107 or 2108.

2104.1.6 2104.1.1 Support on wood. Masonry shall not be supported on wood girders or other forms of wood construction except as permitted in Section 2304.12.

2104.2 Corbeled masonry. Corbeled masonry shall comply with the requirements of Section 1.12 of TMS 402/ACI 530/ ASCE 5.

2104.2.1 <u>2104.1.2</u> **Molded cornices.** Unless structural support and anchorage are provided to resist the overturning moment, the center of gravity of projecting masonry or molded cornices shall lie within the middle one-third of the supporting wall. Terra cotta and metal cornices shall be provided with a structural frame of *approved* noncombustible material anchored in an *approved* manner.

2104.3 Cold weather construction. The cold weather construction provisions of TMS 602/ACI 530.1/ASCE 6, Article 1.8 C, shall be implemented when the ambient temperature falls below 40°F (4°C).

2104.4 Hot weather construction. The hot weather construction provisions of TMS 602/ACI 530.1/ASCE 6, Article 1.8 D, shall be implemented when the ambient air temperature exceeds 100°F (37.8°C), or 90°F (32.2°C) with a wind velocity greater than 8 mph (12.9 km/hr).

Reason: The modifications proposed here simply consolidate the masonry construction requirements of Section 2104 by referencing the requirements of TMS 602 instead of transcribing these provisions into the IBC. No substantive change is intended or implied. Some provisions are maintained in Section 2104 as they are not addressed by TMS 602. These include: support of masonry on wood construction and support/anchorage of molded cornices and terra cotta.

S225-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	2104-S-SAMBI ANET-THOMPSON doc

S226–12 202, 2102.1, 2105.1 thru 2105.3.3

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards (jthompson@ncma.org), Phil Samblanet, The Masonry Society, representing The Masonry Society (psamblanet@masonrysociety.org)

Delete without substitution:

SECTION 202 DEFINITIONS

COMPRESSIVE STRENGTH OF MASONRY. Maximum compressive force resisted per unit of net crosssectional area of masonry, determined by the testing of masonry prisms.

PRISM. An assemblage of *masonry units* and *mortar* with or without grout used as a test specimen for determining properties of the *masonry*.

Revise as follows:

2102.1 General. For the purposes of this chapter and as used elsewhere in this code, the following terms are defined in Chapter 2:

COMPRESSIVE STRENGTH OF MASONRY.

PRISM

SECTION 2105 QUALITY ASSURANCE

2105.1 General.

2105.2 Acceptance relative to strength requirements.

2105.2.1 Compliance with f'm and f'AAC.

2105.2.2 Determination of compressive strength.

2105.2.2.1 Unit strength method.

2105.2.2.1.1 Clay masonry.

TABLE 2105.2.2.1.1 COMPRESSIVE STRENGTH OF CLAY MASONRY

2105.2.2.1.2 Concrete masonry.

TABLE 2105.2.2.1.2 COMPRESSIVE STRENGTH OF CONCRETE MASONRY

2105.2.2.1.3 AAC masonry.

2105.2.2.2 Prism test method.

2105.2.2.2.1 General.

2105.2.2.2.2 Number of prisms per test.

2105.3 Testing prisms from constructed masonry.

2105.3.1 Prism sampling and removal.

2105.3.2 Compressive strength calculations.

2105.3.3 Compliance.

SECTION 2105 QUALITY ASSURANCE

2105.1 General. A quality assurance program shall be used to ensure that the constructed masonry is in compliance with the *construction documents*.

The quality assurance program shall comply with the inspection and testing requirements of Chapter 17 and TMS 602/ACI 530.1/ASCE 6.

Reason: The modifications proposed here simply consolidate the masonry quality assurance requirements of Section 2105 by referencing the requirements of TMS 602 instead of transcribing these provisions into the IBC. No substantive change is intended or implied. The provisions of Section 2105 are virtually identical to the corresponding requirements in TMS 602.

S226-12					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
	,				2105 (NEW)-S-SAMBLANET.doc

S227–12 2107.1, 2107.2, 2107.2.1, 2107.3, 2107.4, 2108.1, 2108.2, 2108.3

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Code and Standards (Thompson@nema.org)

Revise as follows:

2107.1 General. The design of masonry structures using *allowable stress design* shall comply with Section 2106 and the requirements of Chapters 1 and 2 of TMS 402/ACI 530/ ASCE 5 except as modified by Sections 2107.2 through 2107.4 and 2107.3.

2107.2 TMS 402/ACI 530/ASCE 5, Section 2.1.7.7.1.1, lap splices. In lieu of Section 2.1.7.7.1.1, it shall be permitted to design lap splices in accordance with Section 2107.2.1.

2107.2.1 Lap splices. The minimum length of lap splices for reinforcing bars in tension or compression, I_d , shall be

 $I_{d} = 0.002 db f_{s}$

(Equation 21-1)

For SI: $I_d = 0.29db f_s$

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.

2107.3 2107.2 TMS 402/ACI 530/ASCE 5, Section 2.1.7.1, splices of reinforcement. Modify Section 2.1.8.7 as follows:

2.1.7.1 Splices of reinforcement. Lap splices, welded splices or mechanical splices are permitted in accordance with the provisions of this section. All welding shall conform to AWS D1.4. Welded splices shall be of ASTM A 706 steel reinforcement. Reinforcement larger than No. 9 (M #29) shall be spliced using mechanical connections in accordance with Section 2.1.7.7.3.

2107.4 2107.3 TMS 402/ACI 530/ASCE 5, Section 2.3.6, maximum bar size. Add the following to Chapter 2:

2.3.7 Maximum bar size. The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-quarter of the least dimension of the cell, course or collar joint in which it is placed.

2108.1 General. The design of masonry structures using strength design shall comply with Section 2106 and the requirements of Chapters 1 and 3 of TMS 402/ACI 530/ ASCE 5, except as modified by Section 2108.2 through 2108.3.

Exception: AAC masonry shall comply with the requirements of Chapters 1 and 8 of TMS 402/ACI 530/ASCE 5.

2108.2 TMS 402/ACI 530/ASCE 5, Section 3.3.3.3 development. Modify the second paragraph of Section 3.3.3.3 as follows:

The required development length of reinforcement shall be determined by Equation (3-16), but shall not be less than 12 inches (305 mm) and need not be greater than 72 d_{b} .

2108.3 2108.2 TMS 402/ACI 530/ASCE 5, Section 3.3.3.4, splices. Modify items (c) and (d) of Section 3.3.3.4 as follows:

3.3.3.4 (c). A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_{y_i} 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 (d). Mechanical splices shall be classified as Type 1 or 2 according to Section 21.2.6.1 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: Several cycles back the allowable stress and strength design lap splicing requirements for masonry construction were modified as research in this area was still underway. The modifications introduced into the IBC were a stop-gap measure of implementing lap splice detailing requirements based upon similar provisions from the UBC (for ASD) or an upper lap length cap (for SD) while this research was being completed. The research is now complete and the results are reflected in the latest edition of the TMS 402 standard; and as such the lap splicing modifications in Chapter 21 are proposed to be deleted.

Early in the research investigation, concern was expressed that lap splice lengths would become unfeasibly long for certain combinations of bar size, clearance, or cover distances in order to maintain the ductility inherently assumed or explicitly required by contemporary seismic design models and loading requirements as summarized in the following research investigation: http://www.ncma.org/resources/design/Research%20Reports/MR12.pdf

As this research concludes, some reinforcement detailing alternatives (such as a very large diameter reinforcing bar located with minimal masonry cover distance) do not provide targeted strength or ductility with lap splices unless the lap length is exceptionally long or reinforcement is placed transverse to the lap-spliced reinforcement (such as reinforcement in bond beams) as highlighted in the following report: http://www.ncma.org/resources/design/Research%20Reports/MR33.pdf. The IBC lap splice modifications proposed for deletion are based on an upper limit bond strength between reinforcement and grout (in the case of ASD) and an arbitrary cap (in the case of SD) that may not capture all possible failure modes, and therefore may not provide the same level of performance as the lap splice detailing requirements in the TMS 402 reference standard.

S227-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					2107-S-THOMPSON.doc
S228–12 2108.3

Proponent: Charles S. Bajnai, Chesterfield County, VA, representing ICC Building Code Action Committee

Revise as follows:

2108.3 TMS 402/ACI 530/ASCE 5, Section 3.3.3.4, splices. Modify items (c) and (d) of Section 3.3.3.4 as follows:

3.3.3.4 (c). 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 (d). Mechanical splices shall be classified as Type 1 or 2 according to Section 21.2.6.1 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: The International Code Council's Building Code Action Committee was asked to look at addressing the "special moment frames" reference in the code. This term actually refers to masonry wall frames [a.k.a. special moment frames] which were located in Section 2108.9.6 of the 2000 IBC. The requirements for masonry wall frames were removed from the IBC by code change S145-02 which, along with S122-02, substituted a reference to the strength requirements of the 2002 MSJC for the masonry strength design provisions of the IBC. No other current code or standard contains requirements for masonry wall frames so the reference serves no purpose. The committee also conferred with the Masonry Society and it was affirmed that the deletion of this term is appropriate.

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

S228-12

Public Hearing: Co	mmittee:	AS	AM	D	
Ass	sembly:	ASF	AMF	DF	
	-				2108.3-S-BAJNAI-BCAC.doc

S229–12 2111.1, 2111.3, 2111.4, 2113.1, 2113.3, 2113.4

Proponent: Charles S. Bajnai, Chesterfield County, VA, representing ICC Building Code Action Committee (bajnaic@chesterfield.gov)

Revise as follows:

SECTION 2111 MASONRY FIREPLACES

2111.1 Definition. A masonry fireplace is a fireplace constructed of concrete or masonry. Masonry fireplaces shall be constructed in accordance with this section.

2111.1 General. The construction of masonry fireplaces consisting of concrete or masonry shall be in accordance with this section.

2111.3 Seismic reinforcing. In structures assigned to Seismic Design Category A or B, reinforcement and seismic anchorage are not required. Masonry or concrete fireplaces shall be constructed, anchored, supported and reinforced as required in this chapter. In structures assigned to Seismic Design Category C or D, masonry and concrete fireplaces shall be reinforced and anchored as detailed in Sections 2111.3.1, 2111.3.2, 2111.4 and 2111.4.1 for chimneys serving fireplaces. In structures assigned to Seismic Design Category E or F, masonry and concrete chimneys shall be reinforced in accordance with the requirements of Sections 2101 through 2108.

2111.3 Seismic reinforcing. In structures assigned to *Seismic Design Category* A or B, seismic reinforcement is not required. In structures assigned to *Seismic Design Category* C or D, masonry fireplaces shall be reinforced and anchored as detailed in Sections 2111.3.1, 2111.3.2 and 2111.4. In structures assigned to *Seismic Design Category* E or F, masonry fireplaces shall be reinforced in accordance with the requirements of Sections 2101 through 2108.

2111.4 Seismic anchorage. Masonry and concrete chimneys in structures assigned to Seismic Design Category C or D shall be anchored at each floor, ceiling or roof line more than 6 feet (1829 mm) above grade, except where constructed completely within the *exterior walls*. Anchorage shall conform to the following requirements.

2111.4 Seismic anchorage. Masonry fireplaces and foundations shall be anchored at each floor, ceiling or roof line more than 6 feet (1829 mm) above grade with two 3/16-inch by 1-inch (4.8 mm by 25 mm) straps embedded a minimum of 12 inches (305 mm) into the chimney. Straps shall be hooked around the outer bars and extend 6 inches (152 mm) beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 1/2-inch (12.7 mm) bolts.

Exception: Seismic anchorage is not required for the following;

- 1. In structures assigned to Seismic Design Category A or B.
- 2. Where the masonry fireplace is constructed completely within the exterior walls.

2111.4.1 Anchorage. Two 3/16-inch by 1-inch (4.8 mm by 25.4 mm) straps shall be embedded a minimum of 12 inches (305 mm) into the chimney. Straps shall be hooked around the outer bars and extend 6 inches (152 mm) beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 1/2-inch (12.7 mm) bolts.

2113.1 Definition. A masonry chimney is a chimney constructed of 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.

2113.1 General. The construction of masonry chimneys consisting of solid masonry units, hollow masonry units grouted solid, stone or concrete shall be in accordance with this section.

2113.3 Seismic reinforcing. Masonry or concrete chimneys shall be constructed, anchored, supported and reinforced as required in this chapter. In structures assigned to *Seismic Design Category* C or D, masonry and concrete chimneys shall be reinforced and anchored as detailed in Sections 2113.3.1, 2113.3.2 and 2113.4. In structures assigned to *Seismic Design Category* A or B, reinforcement and Seismic anchorage is not required. In structures assigned to *Seismic Design Category* E or F, masonry and concrete chimneys shall be reinforced in accordance with the requirements of Sections 2101 through 2108.

2113.3 Seismic reinforcing. In structures assigned to *Seismic Design Category* A or B, seismic reinforcement is not required. In structures assigned to *Seismic Design Category* C or D, masonry chimneys shall be reinforced and anchored as detailed in Sections 2113.3.1, 2113.3.2 and 2113.4. In structures assigned to *Seismic Design Category* E or F, masonry chimneys shall be reinforced in accordance with the requirements of Sections 2101 through 2108 and anchored as detailed in Section 2113.4.

2113.4 Seismic anchorage. Masonry and concrete chimneys and foundations in structures assigned to *Seismic Design Category* C or D shall be anchored at each floor, ceiling or roof line more than 6 feet (1829 mm) above grade, except where constructed completely within the *exterior walls*. Anchorage shall conform to the following requirements.

2113.4 Seismic anchorage. Masonry chimneys and foundations shall be anchored at each floor, ceiling or roof line more than 6 feet (1829 mm) above grade with two 3/16-inch by 1-inch (4.8 mm by 25 mm) straps embedded a minimum of 12 inches (305 mm) into the chimney. Straps shall be hooked around the outer bars and extend 6 inches (152 mm) beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 1/2-inch (12.7 mm) bolts.

Exception: Seismic anchorage is not required for the following:

- 1. In structures assigned to Seismic Design Category A or B.
- 2. Where the masonry fireplace is constructed completely within the exterior walls.

2113.4.1 Anchorage. Two 3/16-inch by 1-inch (4.8 mm by 25 mm) straps shall be embedded a minimum of 12 inches (305 mm) into the chimney. Straps shall be hooked around the outer bars and extend 6 inches (152 mm) beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 1/2-inch (12.7 mm) bolts.

Reason: The ICC Building Code Action Committee was asked to look at several concerns with sections 2111 and 2113.

First, it was suggested that a definition for masonry fireplaces and chimneys be added to section 202 instead of the current code language that provides a definition of masonry fireplaces and masonry chimneys within the text of the code, Sections 2111.1 and 2113.1 respectively. However, the was the opinion of the committee that definitions are not necessary for this section. The word "Definitions" is proposed to be removed from the titles of Section 2111.1 and Section 2113.1 as shown and the language was modified from the current "defining" language to be "directive" language. No technical changes were made.

Secondly, there have been errors in the code masonry fireplaces and masonry chimneys were split into two separate sections (S261-99). Sections 2111.3 and 2111.4 refer to seismic reinforcement and anchorage for **fireplaces** while 2113.3 and 2113.4 refer to the seismic reinforcement and anchorage requirements for **chimneys**.

In section 2111.3 it states that "In structures assigned to Seismic Design Category C or D, masonry and concrete **fireplaces** shall be reinforced and anchored....." Then the following sentence says, "In structures assigned to Seismic Design Category E or F, masonry and concrete **chimneys** shall be reinforced in accordance with....". Section 2111 is describing **fireplaces** while 2113 describes **chimneys**. So, the reference to "chimneys" should be to "fireplaces". In addition to the wrong word being used, as written, it implies that fireplaces in SDC C and D are required to be "anchored" while (by omission due to the wrong word) they are not required to be "anchored" in SDC E and F.

In this proposal, sections 2111.3 and 2113.3 have been re-written to address those items as well as to re-organize them to be more clear. In addition, the "Seismic anchorage" and "Anchorage" sections 2111.4/2111.4.1 and 2113.4/2113.4.1, respectively, have been combined for clarity and to remove unnecessary language.

No technical changes have been made and no additional requirements have been added to either section.

S229-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2111.1-S-BAJNAI-BCAC.doc

S230-12 2111.1

Proponent: Jim Buckley, Buckley Rumford Co., representing Masonry Alliance for Codes and Standards and Clay Flue Lining Institute (buckley@rumford.com)

Revise as follows:

2111.1 Definition. A masonry fireplace is a fireplace constructed of concrete or masonry solid masonry units, hollow masonry units grouted solid, stone or concrete, hereinafter referred to as "masonry". Masonry fireplaces shall be constructed in accordance with this section.

Reason: To match the language in Section 2113.1 and in the IRC

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

S230-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2111.1-S-BUCKLEY.doc

S231–12 2111.2, 2111.3, 2111.4, 2111.12, 2113.3, 2113.3.1, 2113.3.2

Proponent: Jim Buckley, Buckley Rumford Co., representing Masonry Alliance for Codes and Standards and Clay Flue Lining Institute (buckley@rumford.com)

Revise as follows:

2111.2 Footings and foundations. Footings for masonry fireplaces and their chimneys shall be constructed of concrete or solid masonry at least 12 inches (305 mm) thick and shall extend at least 6 inches (153 mm) beyond the face of the fireplace or foundation wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings shall be at least 12 inches (305 mm) below finished grade.

2111.3 Seismic reinforcing reinforcement. In structures assigned to Seismic Design Category A or B, reinforcement and seismic anchorage are not required. Masonry or concrete fireplaces shall be constructed, anchored, supported and reinforced as required in this chapter. In structures assigned to *Seismic Design Category* C or D, masonry and concrete fireplaces shall be reinforced and anchored as detailed in Sections 2111.3.1, 2111.3.2, 2111.4 and 2111.4.1 for chimneys serving fireplaces. In structures assigned to *Seismic Design Category* E or F, masonry and concrete chimneys fireplaces shall be reinforced in accordance with the requirements of Sections 2101 through 2108.

2111.4 Seismic anchorage. Masonry and concrete chimneys <u>fireplaces</u> in structures assigned to *Seismic Design Category* C or D shall be anchored at each floor, ceiling or roof line more than 6 feet (1829 mm) above grade, except where constructed completely within the *exterior walls*. Anchorage shall conform to the following requirements.

2111.12 Fireplace fireblocking. All spaces between fireplaces and floors and ceilings through which fireplaces 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 be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney fireplaces.

2113.3 Seismic <u>reinforcing</u> <u>reinforcement</u>. Masonry or concrete chimneys shall be constructed, anchored, supported and reinforced as required in this chapter. In structures assigned to *Seismic Design Category* C or D, masonry and concrete chimneys shall be reinforced and anchored as detailed in Sections 2113.3.1, 2113.3.2 and 2113.4. In structures assigned to *Seismic Design Category* A or B, reinforcement and seismic anchorage is not required. In structures assigned to *Seismic Design Category* E or F, masonry and concrete chimneys shall be reinforced in accordance with the requirements of Sections 2101 through 2108.

2113.3.1 Vertical reinforcing reinforcement. For chimneys up to 40 inches (1016 mm) wide, four No. 4 continuous vertical bars anchored in the foundation shall be placed in the concrete between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Section 2103.12. Grout shall be prevented from bonding with the flue liner so that the flue liner is free to move with thermal expansion. For chimneys greater than 40 inches (1016 mm) wide, two additional No. 4 vertical bars shall be provided for each additional 40 inches (1016 mm) in width or fraction thereof.

2113.3.2 Horizontal reinforcing reinforcement. Vertical reinforcement shall be placed enclosed within 1/4-inch (6.4 mm) ties, or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 18 inches (457 mm) o.c. in concrete, or placed in the bed joints of unit masonry, at a minimum of every 18 inches (457 mm) of vertical height. Two such ties shall be provided at each bend in the vertical bars.

Reason: More clear, better English.

S231-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2111.2-S-BUCKLEY.doc

S232-12

2111.11

Proponent: Jim Buckley, Buckley Rumford Co., representing Masonry Alliance for Codes and Standards and Clay Flue Lining Institute (buckley@rumford.com)

Revise as follows:

2111.11 Fireplace clearance. Any portion of a masonry fireplace located in the interior of a building or within the *exterior wall* of a building shall have a clearance to combustibles of not less than 2 inches (51 mm) from the front faces and sides of masonry fireplaces and not less than 4 inches (102 mm) from the back faces of masonry fireplaces. The airspace shall not be filled, except with noncombustible insulation or to provide fireblocking in accordance with Section 2111.12.

Exceptions:

- 1. Masonry fireplaces *listed* and labeled for use in contact with combustibles in accordance with UL 127 and installed in accordance with the manufacturer's installation instructions are permitted to have combustible material in contact with their exterior surfaces.
- 2. When masonry fireplaces are constructed as part of masonry or concrete walls, combustible materials shall not be in contact with the masonry or concrete walls less than 12 inches (306 mm) from the inside surface of the nearest firebox lining.
- 3. Exposed combustible *trim* and the edges of sheathing materials, such as wood siding, flooring and drywall, are permitted to abut the masonry fireplace sidewalls and hearth extension, in accordance with Figure 2111.11, provided such combustible *trim* or sheathing is a minimum of 12 inches (306 mm) from the inside surface of the nearest firebox lining.
- 4. Exposed combustible mantels or *trim* is permitted to be placed directly on the masonry fireplace front surrounding the fireplace opening, provided such combustible materials shall not be placed within 6 inches (153 mm) of a fireplace opening. Combustible material directly above and within 12 inches (305 mm) of the fireplace opening shall not project more than 1/8 inch (3.2 mm) for each 1-inch (25 mm) distance from such opening. Combustible materials located along the sides of the fireplace opening that project more than 11/2 inches (38 mm) from the face of the fireplace shall have an additional clearance equal to the projection.

Reason: To allow noncombustible insulation in clearance to combustible spaces. It clears up confusion with the reference to fireblocking (which can be noncombustible insulation) and is what builders do anyway.

S232-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2111.11-S-BUCKLEY.doc

S233–12 2111.12

Proponent: Jim Buckley, Buckley Rumford Co., representing Masonry Alliance for Codes and Standards and Clay Flue Lining Institute (buckley@rumford.com)

Revise as follows:

2111.12 Fireplace fireblocking. All spaces between fireplaces and floors and ceilings through which fireplaces 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 fireplace.

Reason: To make the language the same as in Section 2113.20. "Chimney" is replaced by "fireplace" as is appropriate in the fireplace section.

S233-12					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
	5				2111.12-S-BUCKLEY.doc

S234–12 2112.2, 2112.5, Chapter 35 (NEW)

Proponent: Timothy N. Seaton, B.S.C.E, Empire Masonry Heaters LLC (tseaton@masonryheater.com)

Revise as follows:

2112.2 Installation. Masonry heaters shall be installed in accordance with this section and comply with one of the following:

- 1. Masonry heaters shall comply with the requirements of ASTM E 1602; or
- 2. Masonry heaters shall be *listed* and labeled in accordance with UL 1482 or EN 15250 and installed in accordance with the manufacturer's installation instructions.

2112.5 Masonry heater clearance. Combustible materials shall not be placed within 36 inches (765 mm) of the outside surface of a masonry heater in accordance with NFPA 211, Section 8-7 (clearances for solid fuel-burning appliances), and the required space between the heater and combustible material shall be fully vented to permit the free flow of air around all heater surfaces.

Exceptions:

- 1. When the masonry heater wall thickness is at least 8 inches (203 mm) thick of solid masonry and the wall thickness of the heat exchange channels is at least 5 inches (127 mm) thick of solid masonry, combustible materials shall not be placed within 4 inches (102 mm) of the outside surface of a masonry heater. A clearance of at least 8 inches (203 mm) shall be provided between the gas-tight capping slab of the heater and a combustible ceiling.
- 2. Masonry heaters *listed* and labeled in accordance with UL 1482 or EN 15250 and installed in accordance with the manufacturer's instructions.

Add new standard to Chapter 35 as follows:

EN

EN 15250 - Slow heat release appliances fired by solid fuel - Requirements and test methods

Reason: UL 1482, *Solid-Fuel Type Room Heaters*, was created to evaluate wood stoves and similar appliances. It does not address thermal mass storage devices of masonry construction such as masonry heaters and contains significant deficiencies in evaluating them. Specifically, UL 1482 stipulates fueling the appliance until temperature equilibrium is reached at which point the safety clearances are verified. This is not an appropriate end of test for masonry heaters and cannot in testing application actually be clearly reached. While UL 1482 may eventually be modified to specifically address masonry heaters, in 2007 the European standard EN 15250, *Slow heat release appliances fired by solid fuel. Requirements and test method*, was finalized specifically to address masonry heaters and similar devices and has since been adopted by 37 countries in Europe and elsewhere. Since Europe is the original source of virtually all masonry heater technology and since IBC already references European Union standards elsewhere, it is appropriate to reference this standard here. EN 15250 stipulates the same allowable temperature elevations of adjacent combustible materials as UL 1482 but uses an appropriate test fueling method.

S234-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				2112.2-S-SEATON.doc

S235–12 2112.5, Table 2112.1 (NEW), Chapter 35 (NEW)

Proponent: Timothy N. Seaton, B.S.C.E., Empire Masonry Heaters LLC

Revise as follows:

2112.5 Masonry heater clearance. Combustible materials shall not be placed within 36 inches (765 mm) of the outside surface of a masonry heater in accordance with NFPA 211, Section 8-7 (clearances for solid fuel-burning appliances), and the required space between the heater and combustible material shall be fully vented to permit the free flow of air around all heater surfaces.

Exceptions:

- 1. Where the masonry heater wall thickness is at least 8 inches (203 mm) thick of solid masonry and the wall thickness of the heat exchange channels is at least 5 inches (127 mm) thick of solid masonry, combustible materials shall not be placed within 4 inches (102 mm) of the outside surface of a masonry heater. A clearance of at least 8 inches (203 mm) shall be provided between the gas-tight capping slab of the heater and a combustible ceiling. or when the wall thicknesses are similarly 4 inches (102 mm) at the firebox and 2 ½ inches (64 mm) at the heat exchange channel but are lined with at least the inner 2 inches (51 mm) and 1 inch (25 mm) respectively of firebrick(ASTM C27 or ASTM C1261) or refractory equivalent, clearances shall be according to Table 2112.5
- Where masonry heaters *listed* and labeled in accordance with UL 1482 and installed in accordance with the manufacturer's instructions <u>clearances will be as listed</u>.

CONTROLLING	MINIMUM	MASONRY	CLEARANCES FROM COMBUS			TIBLE		
STANDARD	HEATE	R WALL		WALLS	<u> </u>		CEILINGS	
PROVISIONS	THICK	<u>NESS</u>						
	<u>Firebox</u>	<u>Channels</u>	<u>Unprotected</u>	<u>Non-</u> <u>combustible</u> <u>wall surface</u> <u>material^b</u>	Protective shield [©] (from shield)	Both surface ^b and shield ^b (from shield)	<u>Unprotected</u>	Protective shield ^e (from shield)
2112.5 ASTM E 1602 (with NFPA 211)			<u>36" (914</u> <u>mm)</u>	<u>As per NF</u>	PA 211 Sectio	<u>n 12.6</u>	<u>As per NFPA</u> <u>12.</u>	<u>211 Section</u> 6
	<u>8" (203</u> <u>mm)</u>	<u>5" (127</u> <u>mm)</u>		<u>4" (102 m</u>	<u>im)</u>		<u>8" (203</u>	<u>mm)</u>
2112.5.1 ASTM E 1602 (with Exception 1)	4" (100 mm) [including 2" (50 mm) firebrick lining ^a]	2.5" (64 mm) [including 1" (25 mm) firebrick lining ^a }	<u>10" (250</u> <u>mm)</u>	<u>6" (150 mm)</u>	<u>5" (127</u> <u>mm)</u>	<u>3" (75</u> <u>mm)</u>	<u>10" (260</u> <u>mm)</u>	<u>5" (127</u> <u>mm)</u>
2112.5.2 UL 1482/EN 15250 (with Exception 2)	<u>As per ma</u>	nufacturer		<u>As per lis</u>	ting		<u>As per l</u>	listing

TABLE 2112.5 MASONRY HEATER CLEARANCES TO COMBUSTIBLE MATERIALS

a. "Firebrick lining" is a lining constructed of firebrick conforming to ASTM C27 or C1261 or refactor equivalent.

b. <u>"Non-combustible wall surface material" is a wall covering facing the masonry heater made from non-combustible material (Fire Class A) and having at least a 30 minute Fire Resistance Rating</u>

c. <u>"Protective shield" is a non-combustible protective shield placed between the masonry heater and the wall, which extends sideways beyond the heater, and is separated from the wall by at least 1.25 inches (30 mm) and from the floor and ceiling by at least 2 inches (50 mm). The clearance is measured from the shield.</u>

Add new standard to Chapter 35 as follows:

EN

EN 15250-2007 Slow Heat Release Appliances Fired by Solid Fuel - Requirements and Test Methods

Reason: North American masonry heater technology is virtually all sourced in Europe where the devices have been built for centuries. In conformance with typical European standards, ASTM E1602, *Standard Guide for Construction of Solid Fuel Burning Masonry Heaters,* does not stipulate masonry heater wall thickness nor relate it to clearances to combustibles. In contrast to masonry fireplace construction and operation, masonry heater wall thickness does not necessarily relate to surface temperature but instead to the time it takes for the heat to begin radiating from the surface and to the total time radiation will occur. For this reason thicker wall construction may in fact be more dangerous with overfiring situations than thinner wall construction.

Until recent IBC and IRC code revisions, all minimum masonry heater clearances were 4" (102 mm) to surface wall or protective shield as per ASTM E1602. I can locate no documented examples of wall ignition from masonry heaters of any wall thickness at this clearance or under ASTM E1602 as the sole ruling clearance standard.

In the recent IBC/IRC code revisions "NFPA 211, Section 8-7 (clearances for solid fuel-burning appliances)" (*sic*) was made the ruling standard for masonry heater clearances instead of ASTM E1602 even though this former standard was created for wood stoves and similar appliances and had no real application to masonry heaters. This standard stipulates 36" clearance to combustible materials with possible reduction to 12" with approved reduction methods. These clearances may be realistic for metal stoves and similar appliances but are unnecessarily restrictive for masonry heaters which in contrast by definition cannot exceed 230° F (110° C) surface temperatures in normal operation (ASTM E1602 Section 3.2.14).

The recent IBC/IRC revisions created two exceptions to the NFPA 211 rule; 1) for lab tested and listed devices, and 2) for masonry heaters with thick firebox and heat channel walls which by European practice are only used for masonry heaters with large heat storage intended to be fired at very long intervals. This latter class of masonry heaters is built increasingly rarely in Europe as the energy codes were written and tightened there and lower output and more responsive masonry heating was required. The same change in code structure is occurring here in North America, and the 36" clearance stipulation for other than thick walled masonry heaters is making masonry heater construction in new projects and particularly in renovation projects unnecessarily complex and expensive. The typical masonry heater sold is custom in design and cannot support laboratory safety testing.

I am not proposing removing existing code clearance provisions though they have not been lab safety tested and verified (as the code provisions for masonry fireplaces have not). The existing safety tests, UL127 and UL1482 were created for manufactured metal appliances and limited in their application to masonry devices. Instead I am proposing IBC adopt building code provisions from Europe for masonry heater clearances where such clearances have been verified through decades and centuries of use. There is no overall European Union document for code built (as opposed to listed) masonry heater clearances. I am attaching the prevailing Austrian standard TRVB 105:1986, *Technical Regulations for Preventive Fire Protection: Fireplaces for Solid Fuels* as a more conservative European example. I propose these clearances, which are more restrictive than ASTM E1602, be adopted for masonry heaters not covered by the existing IBC language under an expanded Exception 1. Please note that in this Austrian standard "fireplaces" refers collectively to iron stoves, open fireplaces, and masonry heaters.

Note also that the ASTM C27 and C1261 firebrick citation is borrowed from existing IBC/IRC fireplace provisions. C1261 is no longer listed in the ASTM standards volume and may not have been renewed.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S235-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2112.5 #1-S-SEATON.doc

S236-12

2112.5

Proponent: Timothy N. Seaton, B.S.C.E., Empire Masonry Heaters LLC (tseaton@masonryheater.com)

Revise as follows:

2112.5 Masonry heater clearance. Combustible materials shall not be placed within 36 inches (765 <u>914</u> mm) <u>or the distance of the allowed reduction method</u> of <u>from</u> the outside surface of a masonry heater in accordance with NFPA 211, Section 8-7 (clearances for solid fuel-burning appliances), <u>12.6 Clearances</u> from Solid Fuel-Burning Appliances, and the required space between the heater and combustible material shall be fully vented to permit the free flow of air around all heater surfaces.

Exceptions:

- When the masonry heater wall thickness is at least 8 inches (203 mm) thick of solid masonry and the wall thickness of the heat exchange channels is at least 5 inches (127 mm) thick of solid masonry, combustible materials shall not be placed within 4 inches (102 mm) of the outside surface of a masonry heater. A clearance of at least 8 inches (203 mm) shall be provided between the gas-tight capping slab of the heater and a combustible ceiling.
- 2. Masonry heaters *listed* and labeled in accordance with UL 1482 and installed in accordance with the manufacturer's instructions.

Reason: 1) Metric conversion is incorrect; 2) NFPA 211 citation is incorrect; and 3) NFPA 211 Section 12.6 allows clearances under 36" with stipulated distance reduction strategies.

S236-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2112.5 #2-S-SEATON.doc

S237–12 2113.19

Proponent: Jim Buckley, Buckley Rumford Co., representing Masonry Alliance for Codes and Standards and Clay Flue Lining Institute (buckley@rumford.com)

Revise as follows:

2113.19 Chimney clearances. Any portion of a masonry chimney located in the interior of the building or within the *exterior wall* of the building shall have a minimum airspace clearance to combustibles of 2 inches (51 mm). Chimneys located entirely outside the *exterior walls* of the building, including chimneys that pass through the soffit or cornice, shall have a minimum airspace clearance of 1 inch (25 mm). The airspace shall not be filled, except to provide <u>noncombustible insulation and</u> fireblocking in accordance with Section 2113.20.

Exceptions:

- 1. Masonry chimneys equipped with a chimney lining system *listed* and labeled for use in chimneys in contact with combustibles in accordance with UL 1777, and installed in accordance with the manufacturer's instructions, are permitted to have combustible material in contact with their exterior surfaces.
- Where masonry chimneys are constructed as part of masonry or concrete walls, combustible materials shall not be in contact with the masonry or concrete wall less than 12 inches (305 mm) <u>8 inches (204 mm)</u> from the inside surface of the nearest flue lining.
- Exposed combustible *trim* and the edges of sheathing materials, such as wood siding, are permitted to abut the masonry chimney sidewalls, in accordance with Figure 2113.19, provided such combustible *trim* or sheathing is a minimum of 12 inches (305 mm) <u>8 inches (204 mm)</u> from the inside surface of the nearest flue lining. Combustible material and *trim* shall not overlap the corners of the chimney by more than 1 inch (25 mm).

Reason: To allow non ombustible insulation in clearance to combustible spaces. It clears up confusion with the reference to fireblocking (which can be noncombustible insulation) and is what builders do anyway.

Changing the 12" of solid masonry to 8" where chimneys can be in contact with combustible trim or framing in a masonry wall is consistent with the relative wall thicknesses, historic experience and recent engineering studies.

S237-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2113.9-S-BUCKLEY.doc

S238-12

202, 722.5.1, 722.5.1.1, 722.6.1.4, 722.5.1.4.1, 722.5.1.4.5, 722.5.2, 722.5.2.1, 722.5.2.2.1, 1615.3.2, 1809.11, 2205.1, 2205.2 (NEW), 2205.2.1 (NEW), 2205.2.1.2 (NEW), 2205.2.2 (NEW), 2203.1, 2203.2, 2206.1, 2206.2, 2206.2.1 (NEW), NEW),

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction (bmanley@steel.org)

Revise as follows:

SECTION 202 DEFINITIONS

STEEL <u>MEMBER</u> <u>ELEMENT</u>, STRUCTURAL. Any steel structural member of a building or structure consisting of a rolled steel structural shape rolled shapes, pipe, hollow structural sections, plates, bars, sheets, rods or steel castings other than cold-formed steel, or steel joist members.

Revise as follows:

722.5.1 Structural steel columns. The *fire-resistance ratings* of <u>structural</u> steel columns shall be based on the size of the element and the type of protection provided in accordance with this section.

722.5.1.1 General. These procedures establish a basis for determining the *fire resistance* of column assemblies as a function of the thickness of fire-resistant material and, the weight, *W*, and heated perimeter, *D*, of <u>structural</u> steel columns. As used in these sections, *W* is the average weight of a structural steel column in pounds per linear foot. The heated perimeter, *D*, is the inside perimeter of the fire-resistant material in inches as illustrated in Figure 722.5.1(1).

722.5.1.4 Concrete-protected columns. The *fire resistance* of structural steel columns protected with concrete, as illustrated in Figure 722.5.1(6) (a) and (b), shall be permitted to be determined from the following expression:

$$R = R_o(1 + 0.03_m)$$

where:

$$R_{o} = 10 (W/D)^{0.7} + 17 (h^{1.6}/k_{c}^{0.2}) \times [1 + 26 \{H/p_{c}c_{c}h(L+h)\}^{0.8}]$$

As used in these expressions:

- R = Fire endurance at equilibrium moisture conditions (minutes).
- R_{o} = Fire endurance at zero moisture content (minutes).
- m = Equilibrium moisture content of the concrete by volume (percent).
- W = Average weight of the <u>structural</u> steel column (pounds per linear foot).
- D = Heated perimeter of the <u>structural</u> steel column (inches).
- *h* = Thickness of the concrete cover (inches).
- k_c = Ambient temperature thermal conductivity of the concrete (Btu/hr ft °F).
- H = Ambient temperature thermal capacity of the structural steel column = 0.11W (Btu/ ft °F).
- p_c = Concrete density (pounds per cubic foot).
- c_c = Ambient temperature specific heat of concrete (Btu/lb °F).
- L = Interior dimension of one side of a square concrete box protection (inches).

(Equation 7-14)

722.5.1.4.1 Reentrant space filled. For wide-flange <u>structural</u> steel columns completely encased in concrete with all reentrant spaces filled [Figure 722.5.1(6)(c)], the thermal capacity of the concrete within the reentrant spaces shall be permitted to be added to the thermal capacity of the steel column, as follows:

$$H = 0.11 W + (p_c c_c / 144) (b_f d - A_s)$$

where:

- b_f = Flange width of the <u>structural</u> steel column (inches).
- d = Depth of the <u>structural</u> steel column (inches).
- A_s = Cross-sectional area of the steel column (square inches).

FIGURE 721.5.1(5)

WIDE FLANGE STRUCTURE STRUCTURAL STEEL COLUMNS WITH SPRAYED FIRE-RESISTANT MATERIALS

(No change to figure)

722.5.1.4.5 Masonry protection. The *fire resistance* of structural steel columns protected with concrete masonry units or clay masonry units as illustrated in Figure 722.5.1(7), shall be permitted to be determined from the following expression:

 $R = 0.17 (W/D)^{0.7} + [0.285 (T_e^{1.6}/K^{0.2})] [1.0 + 42.7 \{(A_s/d_m T_e)/(0.25p + T_e)\}^{0.8}]$

where:

- *R* = *Fire-resistance rating* of column assembly (hours).
- W = Average weight of <u>structural</u> steel column (pounds per foot).
- D = Heated perimeter of structural steel column (inches) [see Figure 722.5.1(7)].
- T_e = Equivalent thickness of concrete or clay masonry unit (inches) (see Table 722.3.2 Note a or Section 722.4.1).
- K = Thermal conductivity of concrete or clay masonry unit (Btu/hr · ft · °F) [see Table 722.5.1(3)].
- A_s = Cross-sectional area of <u>structural</u> steel column (square inches).
- d_m = Density of the concrete or clay masonry unit (pounds per cubic foot).
- p = Inner perimeter of concrete or clay masonry protection (inches) [see Figure 722.5.1(7)].

722.5.2 Structural steel beams and girders. The *fire resistance ratings* of <u>structural</u> steel beams and girders shall be based upon the size of the element and the type of protection provided in accordance with this section.

722.5.2.1 Determination of fire resistance. These procedures establish a basis for determining resistance of structural steel beams and girders which differ in size from that specified in *approved* fire-resistance-rated assemblies as a function of the thickness of fire-resistant material and the weight (W) and heated perimeter (D) of the beam or girder. As used in these sections, W is the average weight of a structural steel member <u>structural steel element</u> in pounds per linear foot (plf). The heated perimeter, D, is the inside perimeter of the fire-resistant material in inches as illustrated in Figure 722.5.2.

722.5.2.2.1 Minimum thickness. The use of Equation 7-17 is subject to the following conditions:

- 1. The weight-to-heated-perimeter ratio for the substitute beam or girder (*W*2/*D*2) shall not be less than 0.37.
- 2. The thickness of fire protection materials calculated for the substitute beam or girder (*T1*) shall not be less than 3/8 inch (9.5 mm).
- 3. The unrestrained or restrained beam rating shall not be less than 1 hour.
- 4. When used to adjust the material thickness for a restrained beam, the use of this procedure is

(Equation 7-15)

(Equation 7-16)

limited to <u>structural</u> steel sections classified as compact in accordance with the AISC *Specification for Structural Steel Buildings*, (AISC 360-05).

Revise as follows:

1615.3.2 Structural steel, open web steel joist or joist girder, or composite steel and concrete frame structures. Frame structures constructed with a structural steel frame or a frame composed of open web steel joists, joist girders with or without other structural steel elements <u>structural steel elements</u> or a frame composed of composite steel or composite steel joists and reinforced concrete elements shall conform to the requirements of this section.

Revise as follows:

1809.11 Steel grillage footings. Grillage footings of structural steel shapes <u>structural steel elements</u> shall be separated with *approved* steel spacers and be entirely encased in concrete with at least 6 inches (152 mm) on the bottom and at least 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

Revise as follows:

2203.1 Identification. Identification of structural steel members structural steel elements shall comply with the requirements contained in AISC 360. Identification of cold-formed steel members shall comply with the requirements contained in AISI S100. Identification of cold-formed steel light-frame construction shall also comply with the requirements contained in AISI S200. Other steel furnished for structural load-carrying purposes shall be properly identified for conformity to the ordered grade in accordance with the specified ASTM standard or other specification and the provisions of this chapter. Steel that is not readily identifiable as to grade from marking and test records shall be tested to determine conformity to such standards.

2203.2 Protection. Painting of structural steel members <u>structural steel elements</u> shall comply with the requirements contained in AISC 360. Painting of open-web steel joists and joist girders shall comply with the requirements of SJI CJ-1.0, SJI JG-1.1, SJI K-1.1 and SJI LH/DLH-1.1. Individual structural members and assembled panels of cold-formed steel construction shall be protected against corrosion in accordance with the requirements contained in AISI S100. Protection of cold-formed steel light-frame construction shall also comply with the requirements contained in AISI S200.

2205.1 General. The design, fabrication and erection of *structural steel <u>elements in</u>* for buildings, and structures, and portions thereof shall be in accordance with AISC 360. Where required, the seismic design of structural steel structures shall be in accordance with the additional provisions of Section 2205.2.

2205.2 Seismic design. Where required, the seismic design, fabrication and erection of buildings, structures, and portions thereof shall be in accordance with Sections 2205.2.1 or 2205.2.2, as applicable.

2205.2.1 Seismic requirements for structural steel structures <u>Structural steel seismic-force</u> resisting systems. The design, fabrication and erection of structural steel structures to resist seismic forces seismic-force resisting systems shall be in accordance with the provisions of Section 2205.2.1 2205.2.1.1 or 2205.2.2 2205.2.1.2, as applicable.

2205.2.1 <u>2205.2.1.1</u> Seismic Design Category B or C. Structural steel Structures assigned to Seismic Design Category B or C shall be of any construction permitted in Section 2205. Where a response modification coefficient, R, in accordance with ASCE 7, Table 12.2-1 is used for the design of structural steel structures assigned to Seismic Design Category B or C, the structures shall be designed and detailed in accordance with the requirements of AISC 341.

Exception: The response modification coefficient, R, designated for "Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems" in ASCE 7, Table 12.2-1 shall be permitted for systems designed and detailed in accordance with AISC 360, and need not be designed and detailed in accordance with AISC 341.

2205.2.2 Seismic Design Category D, E or F. Structural steel Structures assigned to Seismic Design Category D, E or F shall be designed and detailed in accordance with AISC 341, except as permitted in ASCE 7, Table 15.4-1.

2205.2.2 Structural steel elements. The design, fabrication and erection of *structural steel elements* in seismic-force resisting systems other than those covered in Section 2205.2.1, including struts, collectors, chords and foundation elements, shall be designed and detailed in accordance with AISC 341 if:

- 1. The structure is assigned to Seismic Design Category D, E or F, except as permitted in ASCE 7, Table 15.4-1.
- 2. A response modification coefficient, R, greater than 3 in accordance with ASCE 7, Table 12.2-1 is used for the design of the structure assigned to Seismic Design Category B or C.

2206.1 General. Systems of structural steel <u>structural steel elements</u> acting compositely with reinforced concrete shall be designed in accordance with AISC 360 and ACI 318, excluding ACI 318 Chapter 22. Where required, the seismic design of composite steel and concrete systems shall be in accordance with the additional provisions of Section 2206.2.

2206.2 Seismic design. Where required, the seismic design, fabrication and erection of composite steel and concrete systems shall be in accordance with the additional provisions of this section.

2206.2 <u>2206.2.1</u> Seismic requirements for composite structural steel and concrete construction. Where a response modification coefficient, R, in accordance with ASCE 7, Table 12.2-1 is used for the design of systems of structural steel acting compositely with reinforced concrete, the structures shall be designed and detailed in accordance with the requirements of AISC 341.

Reason: This comprehensive proposal not only makes a number of editorial modifications for clarification purposes, it also introduces into Chapter 22 the term and associated requirements for "structural steel elements" and carries that change throughout the remainder of the IBC, as necessary. Note that the Chapter 17 proposal introducing this term is handled in a separate, companion proposal. Please refer to it for additional background.

The purpose of introducing this new term and its associated requirements is to ensure that the wide range of structural steel components in buildings, structures and portions thereof are appropriately covered for design, fabrication and erection. Concerns have been expressed by the structural engineering community regarding the limited definition of *structural steel* contained in AISC 360-10:

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

Section 2.1 of AISC COSP goes on to list many items that are considered structural steel, and Section 2.1 identifies those items that are specifically excluded from the definition. However, these provisions in AISC COSP are intended to provide a default separation of scope between the work of the structural steel fabricator and erector, and the entity providing miscellaneous iron and steel.

Thus, the AISC COSP provides a definition of *structural steel* for default trade practices. Upon reflection, this is not an ideal definition for use in a model building code. To rectify this situation, this proposal introduces the defined term "structural steel element". The specific change from "member" to "element" was to get away from the confusion caused by the difference between the general term, "steel structural member", and the specific AISC-related term, "structural steel member", used throughout the code. Also, language was added clarifying the types of rolled product that fall under this category of steel construction.

Once the definition was settled upon, the new term was integrated into Section 2205. In Section 2205.1, the intent is for all structural steel elements to be designed, fabricated and erected in accordance with AISC 360. Within the seismic design section, the distinction was drawn between structural steel seismic-force resisting systems, which refer to the sixteen structural steel systems currently listed in ASCE 7-10, Table 12.2-1, and structural steel elements that work as struts, collectors, chords and foundation elements in seismic-force resisting systems composed primarily of other structural materials. These structural steel elements are intended to be designed and detailed in accordance with AISC 341, if they are used in a structural in a high seismic area (SDC D, E or F) or they are utilized in a system that relies heavily on non-elastic energy dissipation, in this case chosen to be a system with a response modification coefficient, R, greater than 3.

The remainder of this proposal simply carries the newly defined term through the rest of the IBC.

 $\label{eq:cost_cost_cost_cost} \textbf{Cost Impact:}. No impact to the cost of construction is anticipated.$

S238-11

Public Hearing: (Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF 202-STEEL MEMBER, STRUCTURAL-G-MANLEY

S239–12 2204.1, 2204.2, 2204.2.1

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

2204.1 Welding. The details of design, workmanship and technique for welding, inspection of welding and qualification of welding operators personnel shall conform to the requirements of the specifications listed in Sections 2205, 2206, 2207, 2208, 2210 and 2211. For special inspection of welding, see shall be provided where required by Section 1705 1705.2.

2204.2 Bolting. The design, installation and inspection of bolts shall be in accordance with the requirements of the specifications listed in Sections 2205, 2206, 2207, 2210 and 2211. For special *inspection* of the installation of high-strength bolts shall be provided where required by see Section 1705. 1705.2.

2204.2.1 <u>2204.3</u> Anchor rods. Anchor rods shall be set in accordance with the *construction documents*. The protrusion of the threaded ends through the connected material shall fully engage the threads of the nuts, but shall not be greater than the length of the threads on the bolts.

Reason: These changes are editorial in nature and include the following:

- Clarification of the relationship between the standards referenced in Chapter 22 and the requirements for special inspection in Chapter 17
- Deletion of the term "operators" in favor of the term "personnel". The term "operators" excludes welders and tack welders as defined by AWS D1.1. "Personnel" is the more inclusive term.
- Modification of the hierarchy with regard to Anchor Rods. Anchor rods are not bolts. They are rods. They should not be a subsection of bolting, but rather stand on their own.

S239-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					2204 1-S-MANI EY doc

S240–12 1604.3.3, 2203.2, 2207.1, 2207.1.1 (NEW), 2207.2, 2207.3, 2207.4, 2207.5,

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute, representing Steel Joist Institute (bmanley@steel.org)

Revise as follows:

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

2203.2 Protection. Painting of structural steel members shall comply with the requirements contained in AISC 360. Painting of open-web steel joists and joist girders shall comply with the requirements of SJI CJ-4.0, SJI JG-1.1, SJI K-1.1 and SJI LH/DLH-1.1. Individual structural members and assembled panels of cold-formed steel construction shall be protected against corrosion in accordance with the requirements contained in AISI S100. Protection of cold-formed steel light-frame construction shall also comply with the requirements contained in AISI S200.

2207.1 General. The design, manufacture and use of open web steel joists and joist girders shall be in accordance with one of the following Steel Joist Institute (SJI) specifications:

- 1. SJI-CJ-1.0
- 2. SJI-K-1.1
- 3. SJI-LH/DLH-1.1
- 4. SJI-JG-1.1

2207.1.1 Seismic design. Where required, the seismic design of buildings shall be in accordance with the additional provisions of Section 2205.2 or 2211.6.

2207.2 Design. The *registered design professional* shall indicate on the *construction documents* the steel joist and/or steel joist girder designations from the specifications listed in Section 2207.1 and shall indicate the requirements for joist and joist girder design, layout, end supports, anchorage, non-SJI standard bridging, bridging termination connections and bearing connection design to resist uplift and lateral loads. These documents shall indicate special requirements as follows:

- 1. Special loads including:
 - 1.1. Concentrated loads;
 - 1.2. Nonuniform loads;
 - 1.3. Net uplift loads;
 - 1.4. Axial loads;
 - 1.5. End moments; and
 - 1.6. Connection forces.
- 2. Special considerations including:
 - 2.1. Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder are as indicated in the SJI catalog) that differ from those defined by the SJI specifications listed in Section 2207.1;
 - 2.2. Oversized or other nonstandard web openings; and
 - 2.3. Extended ends.
- 3. <u>Live load</u> deflection criteria for live and total loads for non-SJI standard joists and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1.

2207.3 Calculations. The steel joist and joist girder manufacturer shall design the steel joists and/or steel joist girders in accordance with the current SJI specifications and load tables <u>listed in Section 2207.1</u> to support the load requirements of Section 2207.2. The *registered design professional* may <u>shall be</u> <u>permitted to</u> require submission of the steel joist and joist girder calculations as prepared by a *registered*

design professional responsible for the product design. If requested by the *registered design professional*, the steel joist manufacturer shall submit design calculations with a cover letter bearing the seal and signature of the joist manufacturer's *registered design professional*. In addition to standard the design calculations submitted under this seal and signature, submittel of the following shall be included:

- 1. Non-SJI standard Bridging details design that differs from the SJI specifications listed in Section <u>2207.1</u> (e.g.for cantilevered conditions, net uplift, etc.).
- 2. Connection details design for:
 - 2.1. Non-SJI standar Connections that differ from the SJI specifications listed in Section 2207.1 (e.g.flushframed or framed connections);
 - 2.2. Field splices; and
 - 2.3. Joist headers.

2207.4 Steel joist drawings. Steel joist placement plans shall be provided to show the steel joist products as specified on the *construction documents* and are to be utilized for field installation in accordance with specific project requirements as stated in Section 2207.2. Steel joist placement plans shall include, at a minimum, the following:

- 1. Listing of all applicable loads as stated in Section 2207.2 and used in the design of the steel joists and joist girders as specified in the *construction documents*.
- Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder configurations are as indicated in the SJI catalog) that differ from those defined by the SJI specifications listed in Section 2207.1.
- 3. Connection requirements for:
 - 3.1. Joist supports;
 - 3.2. Joist girder supports;
 - 3.3. Field splices; and
 - 3.4. Bridging attachments.
- Live and total load deflection criteria for live and total loads for non-SJI standard joists and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1.
- 5. Size, location and connections for all bridging.
- 6. Joist headers.

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

2207.5 Certification. At completion of manufacture, the steel joist manufacturer shall submit a *certificate of compliance* in accordance with Section 1704.2.5.2 stating that work was performed in accordance with *approved construction documents* and with SJI standard specifications listed in Section 2207.1.

Reason: This code change is primarily editorial in nature with the intent to clarify and streamline the requirements for steel joists. Major changes include the following:

- Correction of short titles in Section 2207.1, 1604.3.3 and 2203.2 to reflect the appropriate short title listing in Chapter 35 and correction of SJI address in Chapter 35.
- Deletion of reference to the SJI catalog it is not an adopted reference.
- Deletion of reference to the load tables; they are now incorporated into the relevant SJI specifications.
- Elimination of the vague terms "nonstandard", "non SJI standard", and "standard" used throughout the section. These terms are not defined. To clarify what is intended, a reference to the requirements found in the SJI specifications listed in Section 2207.1 is substituted.

Addition of "joist girders" to Section 2207.2, Item 3 and Section 2207.4, Item 4 for consistency.

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

5240-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2207.1-S-MANLEY.doc

6240 42

S241-12 2207.4

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute, representing Steel Joist Institute (bmanley@steel.org)

Revise as follows:

2207.4 Steel joist drawings. Steel joist placement plans shall be provided to show the steel joist products as specified on the *construction documents* and are to be utilized for field installation in accordance with specific project requirements as stated in Section 2207.2. Steel placement plans shall include, at a minimum, the following:

- 1. Listing of all applicable loads as stated in Section 2207.2 and used in the design of the steel joists and joist girders as specified in the *construction documents*.
- 2. Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder configurations are as indicated in the SJI catalog).
- 3. Connection requirements for:
 - 3.1. Joist supports;
 - 3.2. Joist girder supports;
 - 3.3. Field splices; and
 - 3.4. Bridging attachments.
- 4. Deflection criteria for live and total loads for non-SJI standard joists.
- 5. Size, location and connections for all bridging.
- 6. Joist headers.

Steel joist placement plans do not require the seal and signature of the <u>The</u> joist manufacturer's registered design professional shall not be required to sign and seal the steel joist placement plans.

Reason: This code change is editorial in nature, with the intent of correcting the grammar of the sentence.

S241-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2207.4-S-MANLEY.doc

S242-12 2207.4

Proponent: George R. Stevenson, Jr., S.E., Structural Concepts, Inc., representing Structural Engineers Association of Arizona (gstevenson@scice.com)

Revise as follows:

2207.4 Steel joist drawings. Steel joist placement plans shall be provided to show the steel joist products as specified on the *construction documents* and are to be utilized for field installation in accordance with specific project requirements as stated in Section 2207.2. Steel placement plans shall include, at a minimum, the following:

- 1. Listing of all applicable loads as stated in Section 2207.2 and used in the design of the steel joists and joist girders as specified in the *construction documents*.
- 2. Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder configurations are as indicated in the SJI catalog).
- 3. Connection requirements for:
 - 3.1. Joist supports;
 - 3.2. Joist girder supports;
 - 3.3. Field splices; and
 - 3.4. Bridging attachments.
- 4. Deflection criteria for live and total loads for non-SJI standard joists.
- 5. Size, location and connections for all bridging.
- 6. Joist headers.

Steel joist placement plans do not require the seal and signature of the joist manufacturer's registered design professional. If required by the registered design professional in responsible charge, the steel joist manufacturer shall submit steel joist drawings bearing the seal and signature of the joist manufacturer's registered design professional.

Reason: The sentence deleted above was first included in the 2006 IBC and has caused widespread havoc for structural engineers checking submittals for steel joists since that time. For many decades, it has been customary and necessary for the registered design professional in responsible charge (or engineer of record - EOR) to specify that the joist manufacturer provide structural calculations and joist drawings signed and sealed by their registered design professional. Since 2006, the deleted sentence has commonly been cited by joist suppliers as code-sanctioned grounds why they no longer need to provide signed and sealed joist drawings even if the EOR has specified the requirement. This proposed modification will clarify the code so as to not interfere with submittal requirements specified by the EOR. The language is consistent with that in section 2207.3 for required seals on joist calculations.

As background information, the verification of specified joist loading is one of the most important items to be checked in a joist submittal. The joist loading is typically clearly shown on the joist drawings as required by section 2207.4.1. But, per current code, the joist drawings need not be sealed by the joist engineer; the joist engineer only seals the calculations, which do not clearly show joist loading. Because the calculations and joist drawings are not both sealed by the joist engineer, there is no link between them and it is very difficult for the EOR to determine if the joist engineer used the correct loading by looking at the calculations only, which are typically printouts of some proprietary calculation software. This leads to a safety issue because the joist design cannot be adequately reviewed or verified by the EOR.

Cost Impact: The code change proposal will not increase the cost of construction. No additional work is required.

S242-11				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				2207.4-S-STEVENSON

S243–12 2209.1

Proponent: Victor D. Azzi, P.E., Consulting Structural Engineer, representing the Rack Manufacturers Institute (victorazzi@comcast.net)

Revise as follows:

2209.1 Storage racks. The design, testing and utilization of industrial steel storage racks made of coldformed 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 provisions of Section 15.5.3 of ASCE 7, except that the mapped acceleration parameters, S_s -and S_1 , shall be determined in accordance with Section 1613.3.1.

Reason: The new USGS maps, and the mapped acceleration parameters included in IBC Section 1613.3.1, are included in the new 2011 edition of the RMI/ANSI MH 16.1 standard, as well as in the ASCE 7-2010 and Supplement 1. The new RMI Standard, which is included by reference in the ASCE 7, also includes clarification of Load Combinations (including vertical seismic effects), Redundancy Factors, Minimum Seismic Force for Above-Grade Installations, Beam-to-Column Rotational Capacity and Testing, and Periodic Inspection.

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

Analysis: This code change proposal references RMI_standard MH16.1, which is already referenced in this code. However, the proposed change to code text is written to correlate with a new edition of the standard MH16.1-11 rather than the edition presently referenced in the code, which is the -08 edition. The update to this standard will be considered by the Administrative Code Committee during the 2013 Code Development Cycle. Should this code change proposal be approved, but the update to the standard not be approved, the code text will revert to the text as it appears in the 2012 Edition of the Code.

S243-12					
Public Hearing: C	committee:	AS	AM	D	
A	ssembly:	ASF	AMF	DF	2209.1-S-AZZI.doc

S244–12 2210.1.1.3 (NEW), Chapter 35 (NEW)

Proponent: Thomas Sputo, Ph.D., P.E., S.E., Steel Deck Institute

Add new text as follows:

2210.1.1.3 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be permitted to be designed and constructed in accordance with SDI-C.

Add new standard to Chapter 35 as follows:

SDI

SDI-C-2011 Standard for Composite Steel Floor Deck Slabs

Reason: This Standard contains provisions for the design and construction of composite steel deck-slabs of concrete on composite steel deck, and reflects current design and construction industry practices.

The 2012 IBC contains no provisions for the design of composite slabs on steel deck. The previous reference standard that was contained in the 2009 IBC was deleted from the 2012 IBC. Designers and code officials currently must rely on Section 104.11 of the IBC to use this very common structural system. Adding this Standard to the 2015 IBC would fill this gap.

This Standard is an update to the previous 2006 version of this Standard, and was developed and approved through a consensus process under ANSI guidelines, and complies with ICC CP 28. This Standard, along with all other Steel Deck Institute (SDI) Standards, will be available for free download from the SDI website for all parties.

For review purposes, the SDI C-2011 Standard that is being proposed is available for download and review from this website: http://www.sputoandlammert.com/standard.html

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S244-11

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
· · · · ·				2210.1.1.3 (NEW)-S-SPUTO

S245–12 2201.1, 2203.1, 2203.2, 2211.1, 2211.4, Table 2506.2, Table 2507.2, Chapter 35

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

2201.1 Scope. The provisions of this chapter govern the quality, design, fabrication and erection of steel used structurally in buildings or structures <u>construction</u>.

2203.1 Identification. Identification of structural steel members shall comply with the requirements contained in AISC 360. Identification of cold-formed steel members shall comply with the requirements contained in AISI S100. Identification of cold-formed steel light-frame construction shall also comply with the requirements contained in AISI S200 <u>or AISI S220</u>, as applicable. Other steel furnished for structural load-carrying purposes shall be properly identified for conformity to the ordered grade in accordance with the specified ASTM standard or other specification and the provisions of this chapter. Steel that is not readily identifiable as to grade from marking and test records shall be tested to determine conformity to such standards.

2203.2 Protection. Painting of structural steel members shall comply with the requirements contained in AISC 360. Painting of open-web steel joists and joist girders shall comply with the requirements of SJI CJ-1.0, SJI JG-1.1, SJI K-1.1 and SJI LH/DLH-1.1. Individual structural members and assembled panels of cold-formed steel construction shall be protected against corrosion in accordance with the requirements contained in AISI S100. Protection of cold-formed steel light-frame construction shall also comply with the requirements contained in AISI S200 <u>or AISI S220</u>, as applicable.

2211.1 General. The design and installation of structural members and nonstructural members utilized in cold-formed steel light-frame construction where the specified minimum base steel thickness is between 0.0179 inches (0.455 mm) and not greater than 0.1180 inches (2.997 mm) shall be in accordance with AISI S200 and Sections 2211.2 through 2211.7, or AISI S220, as applicable.

2211.4 <u>Structural</u> wall stud design. <u>Structural</u> wall studs shall be designed in accordance with either AISI S211 or AISI S100.

Revise as follows:

GYPSUM BOARD MATERIALS AND ACCESSORIES				
MATERIAL	<u>STANDARD</u>			
Steel studs, load-bearing Cold-formed steel studs and track, structural	AISI S200 and ASTM C955, Section 8			
Steel studs, nonload-bearing-Cold-formed steel studs and track, nonstructural	AISI S220 and ASTM C645, Section 10			

TABLE 2506.2

(Portions of Table not shown remain unchanged)

TABLE 2507.2 LATH, PLASTERING MATERIALS AND ACCESSORIES

MATERIAL	STANDARD
Steel studs and track Cold-formed steel studs and	ASTM C 645 AISI S200 and; ASTM C 955, Section
track, structural	8
Cold-formed steel studs and track, nonstructural	AISI S200 and ASTM C645, Section 10

(Portions of Table not shown remain unchanged)

Add new standard to Chapter 35 as follows:

AISI

AISI S220—11 North American Standard for Cold-formed Steel Framing-Nonstructural Members

Reason: This proposal represents the results of a major effort to synchronize and coordinate the industry standards related to coldformed steel framing. ASTM Committees C11 and A05, and AISI have been working within the steel framing industry on this "Code Synchronization" effort, the goal of which is to organize and maintain a single path for the building code requirements of cold-formed steel light frame construction products. To this end, a new document, AISI S220, was developed to contain all the necessary requirements for nonstructural products. AISI S220 represents a clarification and coordination of industry requirements. The Steel Framing Industry Association (SFIA), the Steel Stud Manufacturers Association (SSMA), the Association of the Wall and Ceiling Industry (AWCI), and the Gypsum Association (GA) all participated in this effort.

The proper integration of AISI S220 into the IBC requires the following changes:

- Section 2201.1: The scope of this chapter now includes products that are non-structural. Therefore, the statement has been simplified to reflect the broad spectrum of steel construction.
- Section 2203: AISI S220, Section A6.5 includes requirements that cover the identification and protection of nonstructural cold-formed steel framing.
- Section 2211.1: Because of the addition of the reference for nonstructural cold-formed steel framing, the lower limit of the minimum base thickness has been deleted.
- Section 2211.4: The charging language to AISI S211 has been clarified to reflect the distinction between AISI S211 and AISI S220.
- Table 2506.2: The material column has been clarified to refer to "structural" and "nonstructural" CFS studs and track. Additionally, AISI S200 and AISI S220 have been incorporated into the table as the primary references. Only ASTM C645 Section 10, and ASTM C955 Section 8, which cover the requirements for the Penetration Test for screws, have been retained. These sections provide a procedure for evaluating the member's ability to pull the head of a screw below the surface of gypsum sheathing. At this time, AISI S220 does not include this test. Future editions may include it, allowing for the eventual deletion of the specific references to ASTM C645 and C955. AISI S200 and AISI S220 incorporate the material and manufacturing provisions previously included in ASTM C955 and ASTM C645 respectively. Limiting the specific references to ASTM C645 Section 10 and C955 Section 8 removes the "dual paths to code compliance", which has caused confusion in the cold-formed steel framing industry.
- Table 2507.2: Entries match what is contained in Table 2506.2.
- Chapter 35: Reflects the necessary changes to the referenced standards.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S245-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				2201.1-S-MANLEY.doc

S246–12 2301.2, 2308.1, 2309 (NEW)

Proponent: Paul Coats, PE, CBO, American Wood Council (pcoats@awc.org)

Revise as follows:

2301.2 General design requirements. The design of structural elements or systems, constructed partially or wholly of wood or wood-based products, shall be in accordance with one of the following methods:

- 1. Allowable stress design in accordance with Sections 2304, 2305 and 2306.
- 2. Load and resistance factor design in accordance with Sections 2304, 2305 and 2307.
- 3. Conventional light-frame construction in accordance with Sections 2304 and 2308.

Exception: Buildings designed in accordance with the provisions of the AF&PA WFCM shall be deemed to meet the requirements of the provisions of Section 2308.

- 4. WFCM in accordance with Section 2309
- 4<u>5</u>. The design and construction of log structures shall be in accordance with the provisions of ICC 400.

2308.1 General. The requirements of this section are intended for *conventional light-frame construction*. Other methods are permitted to be used, provided a satisfactory design is submitted showing compliance with other provisions of this code. Interior nonload-bearing partitions, ceilings and curtain walls of *conventional light-frame construction* are not subject to the limitations of this section. Alternatively, compliance with AF&PA WFCM shall be permitted subject to the limitations therein and the limitations of this code. Detached one- and two-family dwellings and multiple single-family dwellings (townhouses) not more than three *stories above grade plane* in height with a separate *means of egress* and their accessory structures shall comply with the *International Residential Code*.

SECTION 2309 WOOD FRAME CONSTRUCTION MANUAL

2309.1 WFCM. Structural design in accordance with the WFCM shall be permitted for buildings in any use group subject to the limitations of Section 1.1.3 of the WFCM and the load assumptions contained therein. Structural elements beyond these limitations shall be designed in accordance with accepted engineering practice.

Reason: The WFCM is a consensus document that contains both engineering criteria and engineered prescriptive provisions for wood frame construction. It is an ANSI standard developed by technical committees organized by the American Wood Council and it is already referenced in the code for the design of wood frame structures within its scope.

Item #1 revises the manner in which the WFCM is referenced by removing its association with conventional constructions provisions of 2308. The proposed revision in 2301.2 recognizes WFCM as a separate design method.

Item #2 removes the reference to WFCM as an alternative in Section 2308.1 because it is no longer needed and may lead to confusion about its applicability in accordance with its own applicability limits rather than the limits for conventional construction listed in 2308.2.

Item #3 incorporates reference to WFCM under a new 2309 section, and states clearly that the WFCM may be used for buildings of any use group that fit within the WFCM's applicability limits for building size, configuration, and loads as set out in Section 1.1.3 of the standard.

While WFCM provisions are intended primarily for detached one-and two-family dwellings due to the floor live load assumption associated with those occupancies, many of the WFCM provisions for specific geographic wind, seismic, and snow loads may remain applicable for other buildings. For example, wind provisions for sizing of roof sheathing, wall sheathing, fastening schedule, uplift straps, shear anchorage, shear wall lengths, and wall studs for out of plane wind loads are included in WFCM and are applicable for other use groups within the load limitations of the WFCM tables. Similarly, roof rafter size and spacing for heavy snow, and shear wall lengths and anchorage for seismic are applicable within the load limitations of the WFCM tables. Applications outside the scope of the WFCM tablated requirements, such as floor joist design for 60 psf loading and design of supporting gravity elements for the additional floor live load is beyond the applicability of the WFCM and must be designed in accordance with accepted engineering practice. This parallels the approach taken in Section R301.1.3 of the IRC, which permits unconventional

elements of one and two-family dwellings to be designed per the IBC. This change will expand the availability of engineered but prescriptive options for design of wood frame commercial buildings.

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

S246-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2301.2-S-COATS.doc

S247–12 202 (NEW), 2303.1.12 (NEW), Chapter 35 (New)

Proponent: Edward L. Keith, APA – The Engineered Wood Association (ed.keith@apawood.org)

Add new text as follows:

<u>CROSS-LAMINATED TIMBER.</u> A prefabricated engineered-wood product consisting of at least three layers of solid-sawn lumber or structural composite lumber where the adjacent layers are cross-oriented and bonded with structural adhesive to form a solid wood element.

Add new text as follows:

2303.1.12 Cross-laminated timber. Cross-laminated timber (CLT) shall conform to ANSI/APA PRG 320. Cross-laminated timber shall be identified by grade, thickness, and mill name or identification number by marks provided by an approved testing or grading agency indicating conformance to the referenced standard.

Add new standard to Chapter 35 as follows:

APA

ANSI/APA PRG 320-2011 Standard for Performance-Rated Cross-Laminated Timbers

Reason: While new to the North America, cross-laminated timber (CLT) construction is a well established building system in Europe. This system is made up of solid wood slabs up to 52 feet long, 9 feet wide, and 12 inches thick. Cross-laminated like plywood from lumber planks, CLT has a minimum of 3 layers. (Think plywood on a grand scale!)

These timbers come in a number of configurations suitable for wall, roof and/or floor applications. Due to their makeup, these wall-size timbers can be used in heavy timber construction and have exceptional in plane (shear walls and bracing) and out of plane (wind) strength and stiffness. Having essentially no inside cavities and being solid throughout, air infiltration and inner-wall condensation are essentially eliminated. Being wall sized, these timbers came to the jobsite with all openings pre-cut and erection times are just a fraction of those for conventional construction.

In parallel with the research and development work being conducted in North America, the APA completed the development of an ANSI product standard. A National Design Specification (NDS) supplement is currently under development and several test projects are underway in North America.

Additional information is available at:

http://www.woodworks.org/files/PDF/Presentations/SE-Nov-2010/Mohammad.pdf

S247-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
			202-CROSS LAMINATED TIMBER (NEW)-G-KE	ITH

S248–12 202 (NEW), 2303.1.12 (NEW), Chapter 35 (NEW)

Proponent: Brad Douglas, American Wood Council

Add new text as follows:

SECTION 202 DEFINITIONS

ENGINEERED WOOD RIM BOARD. A full-depth structural composite lumber, wood structural panel, structural glued laminated timber, or pre-fabricated wood l-joist member designed to transfer horizontal (shear) and vertical (compression) loads, provide attachment for diaphragm sheathing, siding and exterior deck ledgers, and provide lateral support at the ends of floor or roof joists or rafters.

Add new text as follows:

2303.1.12 Engineered wood rim board. Engineered wood rim boards shall conform to ANSI/APA PRR 410 or shall be evaluated in accordance with ASTM D 7672. Structural capacities shall be in accordance with ANSI/APA PRR 410 or established in accordance with ASTM D 7672. Rim boards conforming to ANSI/APA PRR 410 shall be marked in accordance with that standard.

Add new standards to Chapter 35 as follows:

ANSI

ANSI/APA PRR 410-2011 Standard for Performance-Rated Engineered Wood Rim Boards

ASTM

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ASTM D 7672-2011e1 Standard Specifications for Evaluating Structural Capacities of Rim Board Products and Assemblies

Reason: Engineered rim board is a key structural element in many engineered wood floor applications where both structural load path through the perimeter member and dimensional change compatibility are design considerations. Two new consensus standards address products intended for engineered wood rim board applications. While both ANSI/APA PRR 410 and ASTM D7672 standards address the fundamental requirements for testing and evaluation of engineered rim board, PRR 410 also includes performance categories for engineered wood products used in engineered rim board applications. Under PRR 410, products are assigned a grade based on performance category (e.g. categories based on structural capacity) and will bear a mark in accordance with the grade. In contrast, ASTM D7672 is applicable for determination of product specific rim board performance (i.e. structural capacities) for engineered wood products that may be recognized in manufacturer's literature or product evaluation reports.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S248-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
			202-ENGINEERED WO	OD RIM BOARD (NEW)-G-COATS

S249–12 202 (NEW), 2303.1.2 (NEW), Chapter 35 (NEW)

Proponent: Edward L. Keith, P.E., APA – The Engineered Wood Association (ed.keith@apawood.org)

Add new text as follows:

SECTION 202 DEFINITIONS

ENGINEERED WOOD RIM JOIST/RIM BOARD. A full-depth structural composite lumber, wood structural panel, structural glued laminated timber, or pre-fabricated wood I-joist member designed to transfer horizontal (shear) and vertical (compression) loads, provide attachment for diaphragm sheathing, siding and exterior deck ledgers, and provide lateral support at the ends of floor or roof joists or rafters.

Add new text as follows:

2303.1.12 Engineered wood rim joist/rim board. Engineered wood rim joists shall conform to ANSI/APA PRR 410 or shall be evaluated in accordance with ASTM D 7672. Engineered wood rim joists conforming to ANSI/APA PRR 410 shall be identified by marks provided by an approved testing or grading agency indicating conformance to the referenced standard.

Add new standard to Chapter 35 as follows:

APA

ANSI/APA PRR 410-2011-Standard for Performance Rated Engineered Wood Rim Boards

ASTM

ASTM D 7672-2011e1-Standard Specification for Evaluating Structural Capacities of Rim Board Products and Assemblies

Reason: With the acceptance of engineered wood floor joists and beams into modern building systems, it had become increasingly important to match the physical properties of the various wood systems used in parallel load paths. The rim joist is a good example in that a solid sawn lumber rim joist should not be used in conjunction with engineered wood floor joists. The engineered wood floor joists are often dry when they are placed in the building system and subject to very little shrinkage as they reach equilibrium moisture content with the completed building system. As such it is imperative that a rim joist product with similar physical properties be used in conjunction with the engineered wood floor joists.

Lumber is normally delivered to the jobsite at a moisture content of from 16 to 18%. As the lumber rim joist dries out and reaches equilibrium moisture content of 8 – 10%, it can shrink by as much as ½". As the lumber rim joist shrinks away from the top of the engineered wood framing all of the vertical loads carried by the rim joist are effectively redistributed to the floor joists and other framing members, not designed for the extra load. For this reason, as well as the resource utilization advantages of engineered wood products, engineered wood rim joists have been produced and sold as compatible sizes to other popular engineered wood products, such as prefabricated wood I-joists. Up until now each of these rim joist products has been manufactured to proprietary standards or no standards at all. The building official was left without any guidance from the building code on the acceptability of these very common produces. Two new consensus-based standards, the ANSI/APA PRR 410 and ASTM D7672, have been developed by industry to correct this discrepancy. The ANSI/APA PRR 410 Standard was developed to provide a vehicle whereby commodity engineered wood rim joists can be manufactured and the ASTM D7672 Standard provides procedures for testing and establishing the structural capacities of proprietary rim joist products.

Voting to accept this consensus-based standard will make the building officials' job easier, provide for better and safer structures to the consumer, promote the use of "Green" materials as well as reducing the regulatory burden for the commodity product manufacturers.

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

S249-12				
Public Hearing:	Committee:	AS	AM	D
-	Assembly:	ASF	AMF	DF

202-ENGINEERED WOOD RIM JOIST-RIM BOARD (NEW)-G-KEITH

S250–12 202 (NEW), 2303.1.4 (NEW), Chapter 35 (NEW)

Proponent: Sam Francis, representing American Wood Council (sfrancis@awc.org)

Add new definition as follows:

SECTION 202 DEFINITIONS

CROSS-LAMINATED TIMBER. A prefabricated engineered wood product consisting of at least three layers of solid-sawn lumber or *structural composite lumber* where the adjacent layers are cross-oriented and bonded with structural adhesive to form a solid wood element.

Add new text as follows:

2303.1.4 Structural glued cross-laminated timber. Cross-laminated timbers shall be manufactured and identified as required in ANSI/APA PRG 320-2011.

Add new standard to Chapter 35 as follows:

ANSI

ANSI/APA PRG 320-2011 Standard for Performance-Rated Cross-Laminated Timber

Reason: Cross-Laminated Timber (CLT) is a new product in North America. First developed in Europe nearly 20 years ago, it is used extensively in Europe. A new North American product manufacturing standard, ANSI/APA PRG 320-2011, has just been completed. This large section, engineered wood product should be defined by the code, and it should conform to the newly developed consensus manufacturing standard.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S250-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				2303.1.4 (NEW)-S-FRANCIS.doc

S251–12 2303.1.8.1

Proponent: Stephen C. Shields, Arch Wood Protection, A Lonza Company, representing self (steve_shields@lonza.com)

Revise as follows:

2303.1.8.1 Identification. Wood required by Section 2304.11 to be preservative treated shall bear the quality *mark* of an inspection agency that maintains continuing supervision, testing and inspection over the quality of the *preservative-treated wood*. Inspection agencies for *preservative-treated wood* shall be *listed* by an accreditation body that complies with the requirements of the American Lumber Standards Treated Wood Program, or equivalent. The quality *mark* shall be on a stamp or *label* affixed to the *preservative-treated wood*, and shall include the following information:

- 1. Identification of treating manufacturer.
- 2. Type of preservative used.
- 3. Minimum preservative retention (pcf).
- 43. End use for which the product is treated.
- 54. AWPA standard to which the product was treated.
- 65. Identity of the accredited inspection agency.

Reason: This change will simplify treated wood quality marking by removing information that is no longer of value.

With many different preservatives now in commercial use, retentions are no longer meaningful and have become confusing for consumers and building inspectors. The traditional 0.25 pounds per cubic foot (pcf) retention, which was at one time universally recognized for above ground treatment, is now only rarely found on commercially treated wood. For treated wood used in exposed, above ground applications (Use Category 3B) in the American Wood Protection Association Standards, various preservatives are listed with minimum retention requirements of 0.013, 0.019, 0.20, 0.04, 0.06, 0.07, 0.10, 0.15, 0.19, 0.206, 0.25, 0.40 and 8.0 pounds per cubic foot.

The existing (remaining) requirements for identification of the type of preservative, a description of the end use, the AWPA standard to which the product was treated and the identity of the approved inspection agency provide all of the information needed so that (1) inspection agency personnel can verify that the product has been manufactured to the retention and penetration required by the referenced standard and (2) a building inspector or consumer can verify that the product has been produced to the recognized standard under a recognized third party quality supervision program and that the product is being used in an application consistent with the end use description.

This change would not prohibit including the preservative retention on a label should a producer desire to do so, it would simply remove it from the listed of mandatory items required by the code. This would also allow producers to remove the retention reference as they redesign product labels from time to time in the normal course of business.

S251-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					2303.1.8.1-S-SHIELDS.doc

S252–12 202, 2303.2

Proponent: AI Godwin, CBO, CPM, Aon Fire Protection Engineering (al.godwin@aon.com)

Revise as follows:

2303.2 Fire-retardant-treated wood. <u>*Fire-retardant-treated wood* shall be in accordance with Sections 2303.2.1 through 2303.2.9.</u> Fire-retardant-treated wood is any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E 84 or UL 723, 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, 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.</u>

SECTION 202 DEFINITIONS

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

Fire-retardant-treated wood. Any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E 84 or UL 723, 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, the flame front shall not progress more than 10½ feet (3200 mm) beyond the centerline of the burners at any time during the test. Pressure treated lumber and plywood that exhibit reduced surface-burning characteristics and resist propagation of fire.

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

Reason: There are actually two definitions of Fire retardant treated wood. One in Section 202 and a more detail definition in the wood Section 2303.2, which states:

"*Fire-retardant-treated* wood is any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E 84 or UL 723, 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, the flame front shall not progress more than 10 ½ feet (3200 mm) beyond the centerline of the burners at any time during the test."

At this time, they do not match. This will correct that issue and place the definition language from Section 2303.2 within Section 202 where it belongs.

Depending on the outcome of this proposal, a Group B proposal would be as follows: Revise R802.1.3 as follows:

R802.1.3 Fire-retardant-treated wood. Fire-retardant treated wood (FRTW) shall be in accordance with Section R802.1.3.1 and R802.1.3.8. is any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E 84 or UL 723, a listed flame spread index of 25 or less and shows no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. In addition, the flame front shall not progress more than 10.5 feet (3200 mm) beyond the center line of the burners at any time during the test.

Revise R202as follows:

FIRE-RETARDANT-TREATED WOOD. Any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E 84 pr UL723, a listed flame spread index of 25 or less and shows no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. In addition, the flame front shall not progress more than 10.5 feet (3200 mm) beyond the center line of the
burners at any time during the test. Pressure-treated lumber and plywood that exhibit reduced surface-burning characteristics and resist propagation of fire.

Other means during manufacturing. A process where the wood raw material is treated with a fire-retardant formulation while undergoing creation as a finished product.

Pressure process. A process for treating wood using an initial vacuum followed by the introduction of pressure above atmosphere.

Cost Impact: This code change proposal will not increase the cost of construction since the provisions already exist in the code.

S252-12				
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF
				202-FIRE RETARDANT TREATED WOOD-G-GODWIN.doc

S253-12 2303.2, Chapter 35 (NEW)

Proponent: Marcelo M. Hirschler, GBH International (gbhint@aol.com)

Revise as follows:

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any <u>homogeneous</u> wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E 84 or UL 723, 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, 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 <u>complies with the requirements of ASTM E 2768 and is listed</u>.

Add new standard to Chapter 35 as follows:

ASTM

E2768-2011 Standard Test Method for Extended Duration Surface Burning Characteristics of Building Materials (30 min Tunnel Test)

Reason: ASTM has now issued a test method, ASTM E2768, which contains the three requirements discussed in section 2303.2, namely that a product be tested in accordance with ASTM E84 or UL 723, and exhibit a flame spread index of 25 or less, show no evidence of significant progressive combustion when the test is continued for 30 minutes (i.e. an additional 20-minute period over the standard ASTM E84 duration of 10 minutes) and that the flame front not progress more than 101/2 feet (3200 mm) beyond the centerline of the burners at any time during the test.

Note that products listed as fire-retardant treated wood to UL 723 or to ASTM E84 (with the additional requirements shown above) will be able to continue to be listed to ASTM E2768 without having to be retested as the ASTM E2768 test method contains all of those requirements. Therefore, this code proposal is basically simple clarification.

The addition of the requirement that fire-retardant treated wood must be a "homogeneous" product is necessary to ensure that products that are coated or only partially impregnated with chemicals are not considered "fire-retardant treated wood" as they are not.

Note that there also needs to be consistency between the definition of fire-retardant treated wood and the requirements in this Chapter 23. At the last cycle it was established that it is important that the code not place a requirement regarding the means of manufacture and the definition at present in Chapter 2 discusses purely "pressure treated wood". A separate proposal has been made to change the definition. The two changes can be made independently.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S253-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	,				2303.2-S-HIRSCHLER.doc

S254–12 2303.4.3

Proponent: Larry Wainright, Qualtim, representing Structural Building Components Association (lwainright@qualtim.com)

Revise as follows:

2303.4.3 Truss submittal package. The truss submittal package provided by the truss manufacturer shall consist of each individual truss design drawing, the truss placement diagram, the permanent individual truss member restraint/bracing method and details and any other structural details germane to the trusses; and, as applicable, the cover/truss index sheet. The submittal package shall be submitted to the registered design professional in responsible charge for final approval prior to fabrication of trusses.

Reason: The purpose of this proposal is to help close the gap in communication that many times exists whereby the RDP does not get the truss submittal package for review to ensure the truss package meets the intent of the building design. The RDP should always have the opportunity to review these prior to fabrication. The language in this proposal is taken from the North Carolina Building Code where the issue of RDP approval has been thoroughly vetted.

S254-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				2303.4.3-S-WAINRIGHT.doc

S255-12 2304.6

Proponent: Jay Crandell, ARES Consulting, representing Foam Sheathing Committee (jcrandell@aresconsulting.biz)

Revise as follows:

2304.6 Wall sheathing. Where wall sheathing is used or required and except as provided for in Section 1405 for weatherboarding or where stucco construction that complies with Section 2510 is installed, enclosed buildings shall be sheathed with one of the materials of the nominal thickness specified in Table 2304.6, foam plastic insulation wall sheathing in accordance with Section 2603 or any other approved material of equivalent strength or durability.

Reason: Foam plastic insulation wall sheathing is a commonly used sheathing material on wood frame walls and provides a means for energy code compliance and also, when approved, water resistive barrier compliance. Its inclusion in Section 2304.6 is necessary to ensure its appropriate use and provide guidance for enforcement by reference to requirements in Section 2603. In addition, Section 2304.6 as currently written requires that "wall sheathing" be used at the exclusion of other accepted wood construction practices that do not use wall sheathing. One example is post-frame buildings which often rely on metal panel diaphragms for weather resistance and bracing without use of wall sheathing. It is important to recognize that Section 2304 does not just apply to conventional wood frame construction using wall sheathing and should not exclusively require use of wall sheathing. Finally, the words "equivalent strength or durability" are stricken because the performance requirements for sheathing will depend on its purpose and application (e.g., all sheathing must have strength to resist wind load, but not all sheathing materials are used as bracing and should not be required to be equivalent on that attribute). Using the term "approved material" adequately conveys that appropriate attributes for sheathing must be provided on the basis of the intended application.

S255-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	,				2304.6-S-CRANDELL.doc

S256-12 Table 2304.6.1

Proponent: Edward L. Keith, P.E., APA – The Engineered Wood Association (ed.keith@apawood.org)

Revise as follows:

TABLE 2304.6.1MAXIMUM NOMINAL DESIGN WIND SPEED, V_{asd} PERMITTED FOR WOOD STRUCTURALPANEL WALL SHEATHING USED TO RESIST WIND PRESSURES a,b,c

b. The table is based on wind pressures acting toward and away from building surfaces in accordance with Section 30.7 of ASCE 7. Lateral requirements shall be in accordance with Section 2305 or Section 2308. <u>The table was developed based on the requirement that the specified wood structural panels would alone resist 100% of the applied wind load. Evaluation includes stud strength, nail withdrawal, nail head pull-through, and the sheathing deflection criteria of *l*/120 in accordance with Table 1604.3, where *l* = distance between studs.</u>

(Portions of table and footnotes not shown remain unchanged)

Reason: This code change is proposed to clarify the basis on which Table 2304.6.1 was developed and approved so as to provide guidance for any materials that are intended to establish equivalency to this table in accordance with Section 104.11 of the IBC.

S256-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				T2304.6.1-S-KEITH.doc

S257–12 2301.2, 2308.2.1, Table 2304.9.1, 2304.7.2.1(NEW), 2304.7.2.1.1 (NEW), Figure 2304.7.2.1.1 (NEW)

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

Revise as follows:

2301.2 General design requirements. The design of structural elements or systems, constructed partially or wholly of wood or wood-based products, shall be in accordance with one of the following methods:

- 1. Allowable stress design in accordance with Sections 2304, 2305 and 2306.
- 2. Load and resistance factor design in accordance with Sections 2304, 2305 and 2307.
- 3. Conventional light-frame construction in accordance with Sections 2304 and 2308.

Exception: Buildings designed in accordance with the provisions of the AF&PA WFCM <u>and</u> <u>Section 2304.7.2.1</u> shall be deemed to meet the requirements of the provisions of Section 2308.

4. The design and construction of log structures shall be in accordance with the provisions of ICC 400.

2308.2.1 Nominal design wind speed greater than 100 mph (3-second gust). Where V_{asd} as determined in accordance with Section 1609.3.1 exceeds 100 mph (3-second gust), the provisions of either AF&PA WFCM, or the ICC 600 are permitted to be used. Wind speeds in Figures 1609A, 1609B, and 1609C shall be converted in accordance with Section 1609.3.1 for use with AF&PA WFCM or ICC 600. Section 2304.7.2.1 shall apply to roof sheathing attachment when using the AF&PA WFCM or ICC 600.

FASTENING SCHEDULE				
CONNECTION	FASTENING ^{a,m}	LOCATION		
31. Wood structural panels and particleboard ^b				
Subfloor, roof and wall sheathing (to framing)				
Where V_{at} equals or exceeds 130 mph. wood				
structural panel roof sheathing shall be fastened				
in accordance with Section 2304.7.2.1				
Single floor (combination subfloor-underlayment				
to framing)				
	1			

TABLE 2304.9.1 FASTENING SCHEDULE

(Portions of Table not shown remain unchanged)

2304.7.2.1 Wood structural panel roof sheathing attachment. Where V_{ult} equals or exceeds 130 mph, wood structural panels used as roof sheathing shall be installed with joints staggered and fastened in accordance with Section 2304.7.2.1.1.

2304.7.2.1.1 Sheathing fastenings. Wood structural panel sheathing shall be fastened to roof framing with 8d annular ring-shank nails at 6 inches on center at edges and 6 inches on center at intermediate framing. Ring-shank nails shall have the following minimum dimensions:

- 1. 0.113 inch nominal shank diameter
- 2. Ring diameter of 0.012 over shank diameter

- 3. 16 to 20 rings per inch
- 4. 0.280 inch full round head diameter
- 5. 2 inch nail length

Where roof framing with a specific gravity, $0.42 \le G < 0.49$ is used, spacing of ring-shank fasteners shall be 4 inches on center in nailing zone 3 in accordance with Figure 2304.7.2.1.1 where V_{ult} is 130 mph or greater.

Exceptions:

- 1. Where roof framing with a specific gravity, $0.42 \le G < 0.49$ is used, spacing of ring-shank fasteners shall be permitted at 12 inches on center at intermediate framing in nailing zone 1 for any V_{ult} and in nailing zone 2 for V_{ult} less than or equal to 140 mph in accordance with Figure 2304.7.2.1.1.
- 2. Where roof framing with a specific gravity, G ≥ 0.49 is used, spacing of ring-shank fasteners shall be permitted at 12 inches on center at intermediate framing in nailing zone 1 for any V_{utt} and in nailing zone 2 for V_{utt} less than or equal to 150 mph in accordance with Figure 2304.7.2.1.1.
- 3. Where roof framing with a specific gravity, $G \ge 0.49$ is used, 8d common or 8d hot dipped galvanized box nails at 6 inches on center at edges and 6 inches on center at intermediate framing shall be permitted for V_{ult} less than or equal to 120 mph in accordance with Figure 2304.7.2.1.1.
- 4. Where roof diaphragm requirements necessitate a closer fastener spacing.



FIGURE 2304.7.2.1.1 ROOF SHEATHING NAILING ZONES

Reason: This proposed modification, if approved, will significantly improve the performance of wood structural panel roofs when subjected to high wind loads. It does so at a minimal to negligible cost which provides an extremely generous benefit/cost ratio. The requirements are based on hundreds of true wood structural panel tests. Extensive roof sheathing fastening tests at Clemson University (Reinhold 2000 – 2002, McKinley 2001) and at the International Hurricane Center – Florida International University (Reinhold, Alvarez 2003) compared the Mean Failure Pressure in psf for roof sheathing panels using both the 8d common and the 8d ring shank nails spaced at 6 inches as prescribed by the code. Sheathing consisted of 5/8 inch thick plywood attached to nominal 2x4 Southern Yellow Pine rafters.

The results of these tests were as follows:

(1) Mean ultimate uplift capacity for panels attached with 8d common nails at 6 inch spacing: 126 pounds per square foot

(2) Mean ultimate uplift capacity for panels attached with 8d ring shank nails at 6 inch spacing: 292 pounds per square foot This shows a 131% improvement in performance when 8d ring shank nails are used instead of the currently prescribed 8d common nails.

Requiring the use of 8d ring shank nails would result in an almost negligible increase in cost. While variations will occur regionally, it's estimated that the cost increase will be less than \$10 for 2000 square foot roof.

S257-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2301 2-S-STAFFORD doc

S258–12 2302.1, Table 2304.7(4)

Proponent: John Mulder, Intertek Testing Services, NA, Inc., representing International Standards Organization Technical Committee 77, *Products in Fibre-reinforced Cement,* and self

Revise as follows:

2302.1 Definitions. For the purposes of this chapter, and as used elsewhere in this code the following terms are defined in Chapter 2:

FIBER-CEMENT PRODUCTS

TABLE 2304.7(4) ALLOWABLE SPAN FOR WOOD STRUCTURAL PANEL COMBINATION SUBFLOOR-UNDERLAYMENT (SINGLE FLOOR)^{a, b} (Panels Continuous Over Two or More Spans and Strength Axis Perpendicular to Supports)

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kN/m₂.

a. Spans limited to value shown because of possible effects of concentrated loads. Allowable uniform loads based on deflection of 1/360 of span is 100 pounds per square foot except allowable total uniform load for 11/8-inch wood structural panels over joists spaced 48 inches on center is 65 pounds per square foot. Panel edges shall have approved tongue-and-groove joints or shall be supported with blocking, unless 1/4-inch minimum thickness wood panel-type or fiber-cement underlayment or 11/2 inches of approved cellular or lightweight concrete is placed over the subfloor, or finish floor is 3/4-inch wood strip.

(Portions of table not shown remain unchanged)

Reason: A revision to Table 2304.7(4) is proposed to include "fiber-cement underlayment". The term "fiber-cement products" is proposed to be included in the definitions here consistent with the definition published in the Terminology Standard ASTM C1154-06, *Standard Terminology for Non-Asbestos Fiber-Reinforced Cement Products* (see attached Standard) and also proposed for revision in Chapter 2 of the IBC code. The current footnote does not clearly describe the allowable type of permitted underlayment. The inclusion of references to "wood panel-type" and "fiber-cement" clarifies the types of recognized products permitted in this type of Code-compliant subfloor/underlayment application (see attached ICC-ES ESR-1381[reference Section 4.3], ESR-2280[reference Section 4.2]). "See the ICC-ES website (http://www.icc-es.org/) to gain access to the referenced ESR reports. "

Cost Impact: The code change proposal will not increase the cost of construction because the proposed addition of fiber-cement underlayment to the table footnote only provides for the choice and use of a type of underlayment currently used in this type of application and permitted in Evaluation Service Reports.

S258-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	2302.1-T2304.7(4)-S-MULDER.doc

S259–12 2304.9.1, Table 2304.9.1

Proponent: Erin Ashley, URS Corporation, representing Department of Homeland Security, Federal Emergency Management Agency (erin.ashley@urscom), John Ingargiola, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@fema.dhs.gov)

Revise as follows:

2304.9.1 Fastener requirements. Connections for wood members shall be designed in accordance with the appropriate methodology in Section 2301.2. For conventional light frame construction in accordance with Section 2308, connections for wood members are permitted to be in accordance with Table 2304.9.1. The number and size of fasteners connecting wood members shall not be less than that set forth in Table 2304.9.1 for any method of wood construction.

TABLE 2304.9.1 FASTENING SCHEDULE

	CONNECTION	FASTENING ^{a,m,<u>a</u>}	LOCATION				
g. This table is subject to the limitations stated in Section 2308.2							

(Portions of table and footnotes not shown remain unchanged)

Reason: The proposed addition of the footnote identifies the limitations of Table 2304.9.1 **Fastening Schedule**, which are set forth several sections later in the code as a condition of the table addressing *conventional light-frame construction*. Limitations to *conventional light-frame construction* are located in Section 2308.2, but these limitations are not directly referred to in Section 2304.9 **Connectors and fasteners** and not currently referenced for the entire Table 2304.9.1.

S259-12				
Public Hearing: Committee	e: AS	AM	D	
Assembly:	ASF	AMF	DF	
			2304.9.1-S-ASHLEY-INGARGI	OLA.doc

S260-12 2304.9.6

Proponent: Jay Crandell, ARES Consulting, representing Foam Sheathing Committee (jcrandell@aresconsulting.biz)

Revise as follows:

2304.9.6 Load path. Where wall framing members are not continuous from foundation sill to roof, the members shall be secured to ensure a continuous load path. Where required, sheet metal clamps, ties or clips shall be formed of galvanized steel <u>not less than 0.0179 inch (0.45 mm) minimum thickness</u> or other *approved* corrosion-resistant material not less than 0.040 inch (1.01 mm) nominal thickness <u>capable of resisting the applied loads</u>.

Reason: The code needs to allow thinner steel based on performance to, when possible, avoid interference of uplift straps with fastening/installation of interior and exterior finishes and sheathings. AISI Standard S105 Product Data permits minimum steel thickness of 0.0179 inches thick for structural and non-structural applications. In addition, 24CFR Section 3280.305 also permits uplift straps of minimum 26 gage (0.0179 inch thick) for manufactured homes even in the highest of wind zones. The current minimum 0.040 inch thickness requirement is not consistent with existing industry consensus standards and needs to be changed such that minimum required steel thickness is governed by performance needed for a specific application.

5260-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				2304.9.6-S-CRANDELL.doc

S261–12 Table 2304.9.1

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

TABLE 2304.9.1 FASTENING SCHEDULE

CONNECTION	FASTENING ^{a,m}	LOCATION			
12. Rim joist to top plate, or	8d (21/2" × 0.131") at 6" o.c.				
other framing below	3" × 0.131" nail at 6" o.c.	toenail			
	3" 14 gage staple at 6" o.c.				

(Portions of table not shown remain unchanged)

Reason: The current code language does not explicitly require connections at perimeter joists to a foundation sill ("mudsill") in the case where a framed floor is built over a crawlspace without cripple-walls (the foundation walls extend to the underside of the floor framing).

This item was first introduced in the 1994 UBC to provide a more complete lateral load path to resist earthquake or wind forces. The original intent surely was to provide for lateral strength in all buildings constructed over a raised foundation: not just cases where cripple walls are present, and not to exclude connections along the sides of the building where framing is parallel to the foundation or cripple wall below.

Lack of connection along joists to the parallel supporting members is considered a deficiency under the 2009 IEBC (for buildings with more than one floor above). IEBC Section A304.1.4 requires supplementation of the joist-to-mudsill or joist-to-top plate connection if existing connectors are not present at 6" on center. The current IBC language for this connection requirement allows construction that is immediately in need of strengthening under the IEBC.

Cost Impact: Negligible cost for new construction; Substantial savings in possible retrofit costs in the case where the deficient connection would have to be supplemented to meet IEBC requirements; Immense savings over losing a building in an earthquake due to an incomplete load path.

S261-12					
Public Hearing:	Committee:	AS	AM AME	D	
	Assembly.	AOI		Ы	T2304.9.1#1-S-KERR.doc

S262-12 Table 2304.9.1

Proponent: Thor Matteson, S.E., representing self

Revise as follows:

TABLE 2304.9.1 FASTENING SCHEDULE

Connection	Fastening	Location
11. Blocking between joists or rafters to foundation sill,	3 - 8d common $(2^{1}/_{2}" \times 0.131")$	Toe-nail
girder, beam, top plate, or other framing below	3 - 3" × 0.131" nails	
	3 - 3″14 gage staples	

(Portions of table not shown remain unchanged)

Reason: The current code language does not explicitly require connections at blocking to a foundation sill ("mudsill") in the case where a framed floor is built over a crawlspace without cripple-walls (the foundation walls extend to the underside of the floor framing).

This item was first introduced in the 1994 UBC to provide a more complete lateral load path to resist earthquake or wind forces. The original intent surely was to provide for lateral strength in all buildings constructed over a raised foundation, not just cases where cripple walls are present.

Lack of connection to the mudsill is considered a deficiency under the 2009 IEBC (for buildings with more than one floor above). IEBC Section A304.1.3 requires supplementation of the blocking-to-mudsill connection if existing connectors are not present at 6" on center. The current IBC language for this connection requirement allows construction that is immediately in need of strengthening under the IEBC.

Cost Impact: Negligible cost for new construction; Substantial savings in possible retrofit costs in the case where the deficient connection would have to be supplemented to meet IEBC requirements; Immense savings over losing a building in an earthquake due to an incomplete load path.

S262-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				T2304.9.1 #1-S-MATTESON

S263-12 Table 2304.9.1

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa.se.com)

Revise as follows:

FASTENING SCHEDULE				
CONNECTION	FASTENING ^{a,m}	LOCATION		
11. Blocking between joists or	3 - 8d common (21/2" × 0.131")			
rafters to top plate, or other	3 - 3" × 0.131" nails	toenail		
framing below	3 - 3" 14 gage staples			

TABLE 2304.9.1

(Portions of table not shown remain unchanged)

Reason: The current code language does not explicitly require connections at blocking to a foundation sill ("mudsill") in the case where a framed floor is built over a crawlspace without cripple-walls (the foundation walls extend to the underside of the floor framing).

This item was first introduced in the 1994 UBC to provide a more complete lateral load path to resist earthquake or wind forces. The original intent surely was to provide for lateral strength in all buildings constructed over a raised foundation, not just cases where cripple walls are present.

Lack of connection to the mudsill is considered a deficiency under the 2009 IEBC (for buildings with more than one floor above). IEBC Section A304.1.3 requires supplementation of the blocking-to-mudsill connection if existing connectors are not present at 6" on center. The current IBC language for this connection requirement allows construction that is immediately in need of strengthening under the IEBC.

Cost Impact: Negligible cost for new construction; Substantial savings in possible retrofit costs in the case where the deficient connection would have to be supplemented to meet IEBC requirements; Immense savings over losing a building in an earthquake due to an incomplete load path.

S263-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				T2304 9 1#2-S-KERR doc

S264–12 Table 2304.9.1

Proponent: Thor Matteson, Structural Engineer, representing self (thor2304@shearwalls.com)

Revise as follows:

TABLE 2304.9.1 FASTENING SCHEDULE

CONNECTION	FASTENING ^{a,m}	LOCATION
12. Rim Perimeter joist (end-joist, or band, rim,	8d (2 ¹ / ₂ " × 0.131") at 6" o.c.	
or header joist) to foundation sill, girder, beam,	3" × 0.131" nail at 6" o.c.	Toenail
top plate, or other framing below	3"14 gage staple at 6" o.c.	
(Dertians of table not above remain unaborged)		

(Portions of table not shown remain unchanged.)

Reason: The current code language does not explicitly require connections at perimeter joists to a foundation sill ("mudsill") in the case where a framed floor is built over a crawlspace without cripple-walls (the foundation walls extend to the underside of the floor framing). The current code also does not define "Rim joist". Carpenters in different regions use different terms for various framing members. In some areas the term "Rim joist" may mean any perimeter floor framing member; in other areas it may exclude perimeter joists that run parallel to footings or walls below (such members are commonly called "End joists").

This item was first introduced in the 1994 UBC to provide a more complete lateral load path to resist earthquake or wind forces. The original intent surely was to provide for lateral strength in all buildings constructed over a raised foundation: not just cases where cripple walls are present, and not to exclude connections along the sides of the building where framing is parallel to the foundation or cripple wall below.

Lack of connection along joists to the parallel supporting members is considered a deficiency under the 2009 IEBC (for buildings with more than one floor above). IEBC Section A304.1.4 requires supplementation of the joist-to-mudsill or joist-to-top plate connection if existing connectors are not present at 6" on center. The current IBC language for this connection requirement allows construction that is immediately in need of strengthening under the IEBC.

This change would also make the Code more usable (internationally especially) where terminology varies for different framing components.

Cost Impact: Negligible cost for new construction; Substantial savings in possible retrofit costs in the case where the deficient connection would have to be supplemented to meet IEBC requirements; Immense savings over losing a building in an earthquake due to an incomplete load path.

S264-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				T2304.9.1-S-MATTESON #2

S265–12 Table 2304.9.1

Proponent: Charles S. Bajnai, Chesterfield County, VA, representing ICC Building Code Action Committee (bajnaic@chesterfield.gov)

Delete and substitute as follows:

TABLE 2304.9.1 FASTENING SCHEDULE

Table 2304.9.1 FASTENING SCHEDULE

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPACING AND
	ELEMENTS	FASTENER	LOCATION
		ROOF	
<u>1</u>	Blocking between ceiling joists or	<u>3-8d common (2.5" x 0.131"); or</u>	at each end, toenail
	rafters to top plate	<u>3-10d box (3" x 0.128"); or</u>	
		<u>3-3" x 0.131" nails; or</u>	
		<u>3-3" 14 gage staples, 7/16" crown</u>	
<u>2</u>	Ceiling joists to top plate	<u>3-8d common (2.5" x 0.131"); or</u>	<u>per joist, toenail</u>
		<u>3-10d box (3" x 0.128"); or</u>	
		<u>3-3" x 0.131" nails; or</u>	
		<u>3-3" 14 gage staples, 7/16" crown</u>	
<u>3</u>	Ceiling joist not attached to parallel	<u>3-16d common (3.5" x 0.162"); or</u>	<u>Face nail</u>
	rafter, laps over partitions (no	<u>4-10d box (3" x 0.128"); or</u>	
	thrust) (see Section 2308.10.4.1,	<u>4-3" x 0.131" nails; or</u>	
	Table 2308.10.4.1)	4-3" 14 gage staples, 7/16" crown	
<u>4</u>	Ceiling joist attached to parallel	Per table 2308.10.4.1	Face nail
	rafter (heel joint) (see Section		
	2308.10.4.1, Table 2308.10.4.1)		
<u>5</u>	Collar tie to rafter	<u>3-10d common (3" x 0.148"); or</u>	<u>Face nail</u>
		<u>4-10d box (3" x 0.128"); or</u>	
		<u>4-3" x 0.131" nails; or</u>	
		4-3" 14 gage staples, 7/16" crown	
<u>6</u>	Rafter or roof truss to top plate	<u>3-10 common (3" x 0.148"); or</u>	<u>Toenail^c</u>
	(See Section 2308.10.1, Table	<u>3-16d box (3.5" x 0.135"); or</u>	
	<u>2308.10.1)</u>	<u>4-10d box (3" x 0.128"); or</u>	
		<u>4-3" x 0.131 nails; or</u>	
		4-3" 14 gage staples, 7/16" crown	
<u>7</u>	Roof rafters to ridge valley or hip	2-16d common (3.5" x 0.162"); or	End nail
	rafters; or, roof rafter to 2-inch ridge	<u>3-10d box (3" x 0.128"); or</u>	
	<u>beam</u>	<u>3-3" x 0.131" nails; or</u>	
		<u>3-3" 14 gage staples, 7/16" crown;</u>	
		<u>or</u>	
		<u>3-10d common (3.5" x 0.148"); or</u>	Toenail
		<u>3-16d box (3.5" x 0.135"); or</u>	
		<u>4-10d box (3" x 0.128"); or</u>	
		<u>4-3" x 0.131" nails; or</u>	
		<u>4-3" 14 gage staples, 7/16" crown</u>	
		WALL	-
<u>8</u>	Stud to stud (not at braced wall	<u>16d common (3.5" x 0.162");</u>	24" o.c. face nail
	panels)	<u>10d box (3" x 0.128"); or</u>	16" o.c. face nail

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPACING AND
	ELEMENTS	FASTENER	LOCATION
		<u>3" x 0.131" nails; or</u>	
		3-3" 14 gage staples, 7/16" crown	
<u>9</u>	Stud to stud and abutting studs at	<u>16d common (3.5" x 0.162"); or</u>	<u>16" o.c. face nail</u>
	intersecting wall corners (at braced		
	<u>wall panels)</u>	<u>16d box (3.5" x 0.135"); or</u>	12" o.c. face nail
		3" x 0.131" nails; or	12" o.c. face nail
		3-3" 14 gage staples, 7/16" crown	
10	Built-up header (2-inch to 2-inch	16d common (3.5" x 0.162"); or	16" o.c. each edge,
	header)		face nail
		<u>16d box (3.5" x 0.135")</u>	12" o.c. each edge,
			face nail
11	Continuous header to stud	4-8d common (2.5" x 0.131"); or	Toenail
_		4-10d box (3" x 0.128")	
12	Top plate to top plate	16d common (3.5" x 0.162"); or	16" o.c. face nail
		10d box (3" x 0.128"); or	12" o.c. face nail
		<u>3" x 0.131" nails; or</u>	
		<u>3" 14 gage staples, 7/16" crown</u>	
<u>13</u>	I op plate to top plate, at end joints	8-16d common (3.5" x 0.162"); or	⊢ace nail on each side
		<u>12-10d box (3" x 0.128"); or</u>	of end joint (minimum
		$\frac{12-3^{\circ} \times 0.131^{\circ} \text{ nails; or}}{12-3^{\circ} \times 1.131^{\circ} \text{ nails; or}}$	24 [°] lap splice length
	B <i>H</i> B <i>H B <i>H</i> B <i>H</i> B <i>H</i> B <i>H</i> B <i>H B <i>H</i> B <i>H B <i>H</i> B <i>H B <i>H</i> B <i>H B <i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i></i>	<u>12-3" 14 gage staples, 7/16" crown</u>	each side of end joint)
<u>14</u>	Bottom plate to joist, rim joist, band	<u>16d common (3.5" x 0.162"); or</u>	<u>16" o.c. face nail</u>
	joist or blocking (not at braced wall	<u>16d box (3.5" x 0.135"); or</u>	<u>12" o.c. face nail</u>
	panels)	$\frac{3^{"} \times 0.131^{"} \text{ nails; or}}{3^{"} \times 0.131^{"} \text{ nails; or}}$	
		<u>3" 14 gage staples, 7/16" crown</u>	
<u>15</u>	Bottom plate to joist, rim joist, band	<u>2-16d common (3.5" x 0.162"); or</u>	<u>16" o.c. face nail</u>
	joist or blocking at braced wall	<u>3-16d box (3.5" x 0.135"); or</u>	
	panels	$\frac{4-3^{"} \times 0.131^{"} \text{ nails; or}}{4-3^{"} \times 0.131^{"} \text{ nails; or}}$	
		<u>4-3" 14 gage staples, //16" crown</u>	·
<u>16</u>	Stud to bottom plate	<u>4-8d common (2.5" x 0.131"); or</u>	loenail
		<u>4-10d box (3" x 0.128"); or</u>	
		$\frac{4-3^{"} \times 0.131^{"} \text{ nails; or}}{4-3^{"} \times 0.131^{"} \text{ nails; or}}$	
		4-3" 14 gage staples, 7/16" crown;	
		<u>or</u>	
		<u>2-16d common (3.5" x 0.162"); or</u>	End nail
		<u>3-10d box (3" x 0.128"); or</u>	
		$\frac{3-3^{n} \times 0.131^{n} \text{ nails; or}}{3-3^{n} \times 0.131^{n} \text{ nails; or}}$	
	-	<u>3-3" 14 gage staples, //16" crown</u>	
<u>17</u>	I op or bottom plate to stud	2-16d common (3.5" x 0.162"); or	End nail
		<u>3-100 DOX (3" X U.128"); Or</u>	
		$\frac{3-3^{\circ} \times 0.131^{\circ}}{2.0^{\circ}}$ nalls; or	
40	-	<u>3-3° 14 gage staples, //16° crown</u>	
<u>18</u>	I op plates, laps at corners and	2-16d common (3.5" x 0.162"); or	Face nall
	Intersections	<u>3-100 DOX (3" X U.128"); or</u>	
		<u>3-3" X U.131" nails; or</u>	
4.0		<u>3-3" 14 gage staples, 7/16" crown</u>	· · ·
<u>19</u>	<u>1" brace to each stud and plate</u>	<u>2-8d common (2.5" x 0.131"); or</u>	Face nail
		<u>2-10d box (3" x 0.128"); or</u>	
		<u>2-3" x 0.131" nails; or</u>	
		2-3" 14 gage staples, 7/16" crown	
		1.2.8d common (2.5" v 0.121"); or	L Eaco nail
<u>20</u>	<u>1" x 6" sheathing to each bearing</u>	$\frac{2-60}{2}$ commutation (2.5 x 0.151), or	Face Hall

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPAC	CING AND			
	ELEMENTS	FASTENER	LO	CATION			
<u>21</u>	<u>1" x 8" and wider sheathing to each</u>	<u>3-8d common (2.5" x 0.131"); or</u>	Face nail				
	bearing	<u>3-10d box (3" x 0.128")</u>					
		<u>FLOOR</u>					
22	Joist to sill, top plate, or girder	3-8d common (2.5" x 0.131"); or	Toenail				
		3-10d box (3" x 0.128"); or					
		<u>3-3" x 0.131" nails; or</u>					
		3-3" 14 gage staples, 7/16" crown					
<u>23</u>	Rim joist, band joist, or blocking to	8d common (2.5" x 0.131"); or	<u>6" o.c., to</u>	enail			
	sill or top plate	<u>10d box (3" x 0.128"); or</u>					
		<u>3" x 0.131" nails; or</u>					
		<u>3" 14 gage staples, 7/16" crown</u>					
<u>24</u>	<u>1" x 6" subfloor or less to each joist</u>	<u>2-8d common (2.5" x 0.131"); or</u>	Face nail				
		<u>3-10d box (3" x 0.128")</u>					
<u>25</u>	2" subfloor to joist or girder	<u>2-16d common (3.5" x 0.162")</u>	Face nail				
<u>26</u>	2" planks (plank & beam – floor &	2-16d common (3.5" x 0.162")	At each b	earing, face			
	roof)		<u>nail</u>				
<u>27</u>	Built-up girders and beams, 2-inch	<u>20d common (4" x 0.192")</u>	<u>32″ o.c., 1</u>	face nail at top			
	lumber layers		and botto	om staggered			
		10d how (2" x 0 120"); or	On oppos	<u>sile sides</u>			
		$\frac{100 \text{ D0X} (3 \text{ X} 0.128), 01}{2" \times 0.121" \text{ poile: or}}$	<u>24 0.C. 18</u>	ace hall at top			
		$\frac{3 \times 0.131}{3}$ Halls, 01 3" 14 gage staples 7/16" crown	and bollo	ite sides			
		And	Eace nail	at ends and			
		2-20d common (4" x 0 192"); or	at each s	nlice			
		$\frac{2}{3}$ -10d box (3" x 0.128"); or		<u>piloo</u>			
		3-3" x 0.131" nails: or					
		3-3" 14 gage staples. 7/16" crown					
28	Ledger strip supporting joists or	3-16d common (3.5" x 0.162"); or	At each je	oist or rafter,			
	rafters	4-10d box (3" x 0.128"); or	face nail	<u> </u>			
		4-3" x 0.131" nails; or					
		4-3" 14 gage staples, 7/16" crown					
<u>29</u>	Joist to band joist or rim joist	3-16d common (3.5" x 0.162"); or	End nail				
		<u>4-10d box (3" x 0.128"); or</u>					
		<u>4-3" x 0.131" nails; or</u>					
		<u>4-3" 14 gage staples, 7/16" crown</u>					
<u>30</u>	Bridging to joist	<u>2-8d common (2.5" x 0.131"); or</u>	Each end	<u>l, toenail</u>			
		<u>2-100 DOX (3" X 0.128"); OF</u>					
		$\frac{2-3 \times 0.131}{2.2}$ 14 gags staples 7/16" grown					
	Wood structural papels (WSP) su	<u>2-3 14 gage staples, 7/10 crown</u>	ing to fram	ning and			
	particleboard wall sheathing to framing ^a						
			Edges	Intermediate			
			(inches)	supports			
				(inches)			
<u>31</u>	<u>3/8" – 1/2"</u>	6d common or deformed (2" x	<u>6</u>	12			
1		0.113") (subfloor and wall)					
		8d box or deformed (2.5" x 0.113")	<u>6</u>	<u>12</u>			
		(root)		40			
		2 3/8" X 0.113" nail (subfloor and	<u>6</u>	<u>12</u>			
		1 ³ / ³ 16 gage staple 7/16" crown	4	8			
1		<u>, ,,, io gago otapio, i/ io otowii</u>	1 -	I ≚			

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPA	CING AND
	ELEMENTS	<u>FASIENER</u>		
			4	0
		<u>2 3/8 X 0.113 mail (roor)</u>	4	ō
		$1\frac{3}{4}$ " 16 gage staple, 7/16" crown	3	6
		(roof)	-	-
32	19/32" – 3/4"	8d common (2.5" x 0.131"); or	6	12
		6d deformed (2" x 0.113)	_	
		2 3/8" x 0.113" nail; or	4	<u>8</u>
		2" 16 gage staple, 7/16" crown		
<u>33</u>	<u>7/8" – 1 1/4"</u>	<u>10d common (3" x 0.148"); or</u>	<u>6</u>	<u>12</u>
		8d deformed (2.5" x 0.131")		
	Othe	er exterior wall sheathing	1	1
<u>34</u>	<u>1/2" fiberboard sheathing</u>	<u>1 ¹/₂" galvanized roofing nail (7/16"</u>	<u>3</u>	<u>6</u>
		head diameter; or		
		<u>6d common (2" x 0.113"); or</u>		
		$\frac{1 \frac{1}{4}}{16}$ gage staple with 7/16" or 1"		
0.5		<u>Crown</u>	0	
<u>35</u>	25/32 fiberboard sneatning	<u>1 % galvanized rooting nail (7/16</u>	<u>3</u>	<u>6</u>
		diameter nead); or		
		$\frac{80 \text{ common (2.5 x 0.131); of}}{11/3 16 \text{ gass stanle with } 7/16^3 \text{ or } 1^3$		
		<u>1 ½ To gage staple with 7/To_or_i</u>		
	Wood structural papels of	<u>crown</u> combination subfloor underlayment t	o framina	
36	3/4" and less	8d common (2.5" x 0.131"); or6d		12
<u>30</u>		deformed (2" x 0 113")	<u> </u>	12
37	7/8" – 1"	8d common (2.5" x 0.131"); or	6	12
		8d deformed (2 1/2" x 0.131")		
<u>38</u>	<u>1 1/8" – 1 1/4"</u>	10d common (3" x 0.148"); or 8d	<u>6</u>	<u>12</u>
		<u>deformed (2 1/2" x 0.131")</u>		
	<u>P</u>	anel Siding to Framing		40
<u>39</u>	<u>¹/2 OF IESS</u>	6d corrosion-resistant siding (1 7/8"	<u>6</u>	<u>12</u>
		<u>× 0.106°); or</u>		
40	5/9"	0.099)	6	10
40	5/8	$\times 0.128$ "); or	<u>o</u>	12
		$\frac{60.120}{1.00}$, or 8d corrosion-resistant casing (2.1/2")		
		x 0 113")		
	<u> </u>	Interior Paneling	l	1
41	1/4"	4d casing (11/2" × 0.080"): or	6	12
<u> </u>	- <u></u>	4d finish (11/2" × 0.072")	-	
42	3/8"	6d casing (2" × 0.099"): or	6	12
	<u></u>	6d finish (Panel supports at 24	-	<u> </u>
		inches)		
a.	Nails spaced at 6 inches at intermediate support	ts where spans are 48 inches or more. For nailin	ng of wood str	uctural panel and
<u>41</u> <u>42</u> <u>a.</u>	<u>1/4</u> " <u>3/8</u> " Nails spaced at 6 inches at intermediate support	4d casing (11/2" × 0.080"); or 4d finish (11/2" × 0.072") 6d casing (2" × 0.099"); or 6d finish (Panel supports at 24 inches) ts where spans are 48 inches or more. For nailing	6 6 Ig of wood str	12 12 uctural panel and

<u>casing.</u> <u>Spacing shall be 6 inches on center on the edges and 12 inches on center at intermediate supports for nonstructural</u> <u>applications. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise</u> b. marked).

c. Where a rafter is fastened to an adjacent parallel ceiling joist in accordance with this schedule and the ceiling joist is fastened to the top plate in accordance with this schedule, the number of toenails in the rafter shall be permitted to be reduced by one nail.

Reason: The ICC Building Code Action Committee sought to reformat and correlate the current fastening schedule for wood frame construction in Chapter 23 with the current fastening schedule in the IRC. The organization of the IRC table was thought to be easier to use, and it was generally acknowledged that it may help users of both codes if the tables more closely resembled each other in format and content.

Descriptions of specified fastening and their capacities in the IBC and IRC tables were compared. In developing the proposed new table, the committee tried to make as few technical changes as possible while reorganizing and reformatting the IBC table to look more like the IRC table. Care was taken to retain, for the most part, all fastening alternatives currently in the IBC, while at the same time adding appropriate alternatives that appear in the IRC for the same connection, if they were missing.

To attain complete coordination between the two tables was not possible because certain technical changes that would have been required were beyond the chosen scope of the committee's work. However, the proposed table is much closer to the IRC table and the committee will look at the IRC table in the Group B changes to attempt further correlations between the two.

When inconsistencies or apparent anomalies were discovered between tables or within the IBC table itself, in general the following principles were applied:

a. attempt to establish a reference common nail specification for each connection where it appeared to be lacking; b. provide box nails alternatives, if lacking, where possible

c. retain all current alternatives for power-driven and staple alternatives (though in a few cases the number or size of fastener was adjusted to be consistent with the IRC or to achieve consistency within the IBC table itself based on other entries);

d. in creating box nail alternatives where they currently are missing, for simplicity assume 10d box nails (3" x 0.128") to be equivalent to 3" x 0.131" power-driven fasteners;

e. take into account calculated connection capacities. (These were also compared to the engineered connections specified in the AWC Wood Frame Construction Manual for like connections.)

Finally, this proposed IBC table is much cleaner and more complete than the current table. Besides adding many fastener alternatives, many detailed and difficult-to-use footnotes in the current table were eliminated since their content was incorporated directly into the proposed table.

The following three tables are provided: i) the proposed IBC Table 2304.9.1 with an additional column of notes explaining how it correlates to the existing IBC table, ii) the existing IBC Table 2304.9.1 with an additional column of notes explaining how it correlates to the proposed IBC table, and iii) the existing IRC table, shown for reference.

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPACING AND	Notes:		
	ELEMENTS	FASTENER	LOCATION			
	ROOF					
1	Blocking between ceiling joists or rafters to top plate	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	at each end, toenail	-Nailing from IBC Row 11. -10d box equivalent to 8d common added.		
2	Ceiling joists to top plate	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	per joist, toenail	-Nailing from IBC Row 15. -10d box equivalent to 8d common added. -Correct power driven number from 5 to 3.		
3	Ceiling joist not attached to parallel rafter, laps over partitions (no thrust) (for parallel rafter case see Section 2308.10.4.1, Table 2308.10.4.1)	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	Face nail	-Nailing from IBC Row 17. -10d box equivalent to power driven nail size added.		
4	Ceiling joist attached to parallel rafter (heel joint) (see Section 2308.10.4.1, Table 2308.10.4.1)	Per table 2308.10.4.1	Face nail	-Nailing from IBC Row 18.		
5	Collar tie to rafter	3-10d common (3" x 0.148"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	Face nail	-Nailing from IBC Row 26. -10d box equivalent to power driven nail size added.		
6	Rafter or roof truss to top plate (See Section 2308.10.1, Table 2308.10.1)	3-10 common (3" x 0.148"); or 3-16d box (3.5" x 0.135"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131 nails; or 4-3" 14 gage staples, 7/16" crown	Toenail ^c	-Nailing from IRC Row 5. -10d box equivalent to power driven nail size added.		

<u>Proposed</u> Table 2304.9.1 with additional column of explanation:

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPACING AND	Notes:
7	Roof rafters to ridge valley or hip	2-16d common (3 5" x 0 162").	End nail	-Nailing from IBC
1.	rafters; or, roof rafter to 2-inch	or		Rows 27 and 28.
	ridge beam	3-10d box (3" x 0.128"); or		-10d box equivalent to
		3-3" x 0.131" nails; or		power driven nail size
		3-3" 14 gage staples, 7/16"		added.
		3-10d common (3.5" x 0.148"):	Toenail	-Nailing from IBC
		or		Rows 27 and 28.
		3-16d box (3.5" x 0.135"); or		-10d box equivalent to
		4-10d box (3" x 0.128"); or		power driven nail size
		4-3" x 0.131" nails; or		added.
		crown		toenailing of rafter in
				Row 6 added.
		WALL	·	
8	Stud to stud (not at braced wall panels)	16d common (3.5" x 0.162");	24" o.c. face nail	-Nailing from IBC Row 9.
	,	10d box (3" x 0.128"); or	16" o.c. face nail	-10d box equivalent to
		3" x 0.131" nails; or		power driven nail size
		3-3" 14 gage staples, 7/16"		added.
		CIOWIT		power driven nail to be
				equivalent to the
				specified common nail.
9	Stud to stud and abutting studs	16d common (3.5" x 0.162"); or	16" o.c. face nail	-Nailing from IBC Row
	at intersecting wall corners (at			23.
	braced wall panels)			-16d box equivalent
		16d box (3.5" x 0.135"); or		Trom IRC Row 8.
		3" x 0 131" nails: or		
		3-3" 14 gage staples, 7/16"		
		crown		
10	Built-up header (2-inch to 2-inch header)	16d common (3.5" x 0.162"); or	16" o.c. each edge, face nail	-Nailing from IBC Row 14.
	,	16d box (3.5" x 0.135")	12" o.c. each edge, face	-16d box equivalent
			nail	added but at 12" o.c.
				spacing.
11	Continuous header to stud	4-8d common (2.5" x 0.131"); or	Toenail	-Nailing from IBC Row
		4-10d box (3" x 0.128")		16. 10-1 h
				-100 DOX equivalent to
12	Top plate to top plate	16d common (3.5" x 0.162"); or	16" o.c. face nail	-Nailing from IBC Row
	- F F F F			10 except that 16d
				common specified in
				lieu of 16d box to align
				sizes
				-10d box equivalent to
				power driven sizes
		10d box (3" x 0 129"); or	12" o o faco poil	added.
1		3" x 0 131" nails: or		
1		3" 14 gage staples, 7/16" crown		
13	Top plate to top plate, at end	8-16d common (3.5" x 0.162");	Face nail on each side of	-Nailing from IBC Row
	joints	Or	end joint (minimum 24" lap	10.
		12-100 DOX (3" X 0.128"); or 12-3" x 0 131" nails: or	splice length each side of	-10d box equivalent to
1		12-3" 14 gage staples. 7/16"		added.
		crown		
14	Bottom plate to joist, rim joist,	16d common (3.5" x 0.162"); or	16" o.c. face nail	-Nailing from IBC Row
	band joist or blocking (not at			6 except that 16d
1	braceu wali paneis)			of 16d box
1				-16d box equivalent
				added at 12" o.c.

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPACING AND	Notes:
	ELEMENTS	FASTENER		
		100 DOX (3.5 X 0.135); 01 3" x 0 131" nails: or	12 O.C. Tace hall	
		3" 14 gage staples, 7/16" crown		
15	Bottom plate to joist, rim joist,	2-16d common (3.5" x 0.162");	16" o.c. face nail	-Nailing from IBC Row
	band joist or blocking at braced	or		6; 16d common
	wall panels	3-160 DOX (3.5" X 0.135"); OF		equivalent added
		4-3" 14 gage staples, 7/16"		
		crown		
16	Stud to bottom plate	4-8d common (2.5" x 0.131"); or	Toenail	-Nailing per IBC Row
		4-10d box (3" x 0.128"); or		8. 10d box equivalent to
		4-3" 14 gage staples. 7/16"		8d common added.
		crown; or		
		2-16d common (3.5" x 0.162");	End nail	-Nailing per IBC Row
		or 2 10d box (2" x 0 128"); or		8.
		3-3" x 0 131" nails: or		power driven sizes
		3-3" 14 gage staples, 7/16"		added.
		crown		
17	Top or bottom plate to stud	2-16d common (3.5" x 0.162");	End nail	-Nailing per IBC Row
		or 3-10d box (3" x 0 128"): or		7. -10d box equivalent to
		3-3" x 0.131" nails; or		power driven sizes
		3-3" 14 gage staples, 7/16"		added.
40	The states laws at some so a	crown	Face and	Na iliana any IDO David
18	I op plates, laps at corners and intersections	2-16d common (3.5" X 0.162");	Face nall	-Nalling per IBC Row
		3-10d box (3" x 0.128"); or		-10d box equivalent to
		3-3" x 0.131" nails; or		power driven sizes
		3-3" 14 gage staples, 7/16"		added.
10	1" brace to each stud and plate	Crown 2-8d common (2.5" x 0.131"): or	Face nail	-Nailing per IBC Row
13		2-10d box (3" x 0.128"): or		20.
		2-3" x 0.131" nails; or		-10d box equivalent to
		2-3" 14 gage staples, 7/16"		8d common added.
20	1" x 6" sheathing to each bearing	Crown 2-8d common (2.5" x 0.131"): or	Face nail	Nailing per IRC Row
20	1 x 0 sheathing to each bearing	2-10d box (3" x 0.128")		21.
				-10d box equivalent to
				8d common added.
21	1" x 8" and wider sheathing to	3-8d common (2.5" x 0.131"); or	Face nail	-Nailing per IRC Rows
	each bealing	3-100 D0x (3 × 0.128)		Rows 4 21 and 22
				-10d box equivalent to
				8d common added.
		FLOOR		
	laista all tas slats as sind.		Teeneil	Nailina francisco DO D
22	Joist to sill, top plate, or girder	3-80 common (2.5" X 0.131"); or 3-10d box (3" x 0.128"); or	ioenali	-ivalling from IBC Row
		3-3" x 0.131" nails; or		-10d box equivalent to
		3-3" 14 gage staples, 7/16"		8d common added.
		crown		
23	Rim joist, band joist or blocking	8d common (2.5" x 0 131") [.] or	6" o.c., toenail	-Nailing from IBC Row
	to sill or top plate	10d box (3" x 0.128"); or	,	12.
		3" x 0.131" nails; or		-10d box equivalent to
24	1" v 6" oubfloor or loos to sack	3" 14 gage staples, 7/16" crown	Easo pail	8d common added.
24	i x o subiloor of less to each	2-ou common (2.5 x 0.131); or 3-10d box (3" x 0.128")	Face Hall	-ivaliing from IBC ROW
	10.01			-10d box equivalent to
				8d common added
25	2" subfloor to joist or girder	2-16d common (3.5" x 0.162")	Face nail	-Nailing from IBC Row
1		1		Э.

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPACING AND		Notes:
	ELEMENTS	FASTENER	LOCATION		
26	2" planks (plank & beam – floor & roof)	2-16d common (3.5" x 0.162")	At each be	earing, face nail	-Nailing from IBC Row 25.
27	Built-up girders and beams, 2- inch lumber layers	20d common (4" x 0.192")	32" o.c., face nail at top and bottom staggered on opposite sides		-Nailing from IBC Row 24. -10d box equivalent to power driven nail size added.
		10d box (3" x 0.128"); or 3" x 0.131" nails; or 3" 14 gage staples, 7/16" crown	24" o.c. fac bottom sta opposite s	ce nail at top and ggered on ides	
		And: 2-20d common (4" x 0.192"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	Face nail a each splice	at ends and at e	-Nailing from IBC Row 24. -10d box equivalent to power driven nail sizes added.
28	Ledger strip supporting joists or rafters	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	At each joist or rafter, face nail		-Nailing from IBC Row 30. -10d box equivalent to power driven nail size added.
29	Joist to band joist or rim joist	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	End nail		-Nailing from IBC Row 29. -10d box equivalent to power driven nail size added.
30	Bridging to joist	2-8d common (2.5" x 0.131"); or 2-10d box (3" x 0.128"); or 2-3" x 0.131" nails; or 2-3" 14 gage staples, 7/16" crown	Each end, toenail		-Nailing from IBC Row 2. -10d box equivalent to 8d common nail added
	Wood structural panels (WSP), s	subfloor, roof and interior wall sho	eathing to f	raming and	
			Edges (inches)	Intermediate supports	
31	3/8" – 1/2"	6d common or deformed (2" x 0.113") (subfloor and wall)	6	12	-Nailing from IBC Row 31.
		8d box or deformed (2.5" x 0.113") (roof)	6	12	-Nailing from IBC Row 31 footnote "L".
		2 3/8" x 0.113" nail (subfloor and wall)	6	12	-Nailing from IBC Row 31.
		1 ³ ⁄ ₄ " 16 gage staple, 7/16" crown (subfloor and wall)	4	8	-Nailing from IBC Row 31 and footnote "o".
		2 3/8 x 0.113" nail (roof)	4	8	-Nailing from IBC Row 31 and footnote "n".
20	40/20" 2/4"	1 3 ⁴ / ⁷ 16 gage staple, 7/16 ⁷ crown (roof)	3	6	-Nailing from IBC Row 31 and footnote "o".
32	19/32 - 3/4	ou common (2.5" X 0.131"); or 6d deformed (2" X 0.113) 2 3/8" X 0.113" poilt or	0	12	-Nailing from IBC Row 31.
33	7/8" – 1 1/4"	2" 16 gage staple, 7/16" crown 10d common (3" x 0 148"); or	6	8	31 and footnote "p".
	Ot	8d deformed (2.5" x 0.131") her exterior wall sheathing			31 and footnote "e".
34	1/2" fiberboard sheathing ^b	1 ½" galvanized roofing nail (7/16" head diameter; or 6d common (2" x 0.113"); or 1 ¼" 16 gage staple with 7/16" or 1" crown	3	6	-Nailing from IBC Row 33 and footnote "g" and "h" and "i".
35	25/32" fiberboard sheathing ^b	1 $\frac{3}{4}$ " galvanized roofing nail (7/16" diameter head); or 8d common (2.5" x 0.131"); or 1 $\frac{1}{2}$ " 16 gage staple with 7/16"	3	6	-Nailing from IBC Row 33 and footnote "g" and "h" and "i".

	DESCRIPTION OF BUILDING	NUMBER AND TYPE OF	SPACING AND		Notes:
		or 1" crown	-		
	Wood structural panels.	combination subfloor underlaym	ent to fran	ning	
36	3/4" and less	8d common (2.5" x 0.131"); or6d deformed (2" x 0.113")	6	12	-Nailing from IBC Row 31 and footnote "e" and IRC Row 39 for common nail size.
37	7/8" – 1"	8d common (2.5" x 0.131"); or 8d deformed (2 ½" x 0.131")	6	12	-Nailing from IBC Row 31 and footnote "e" and IRC Row 40 for common nail size.
38	1 1/8" – 1 ¼"	10d common (3" x 0.148"); or 8d deformed (2 ½" x 0.131")	6	12	-Nailing from IBC Row 31 for common and deformed nail size.
		Panel Siding to Framing			
39	1/2" or less	6d corrosion-resistant siding (1 7/8" × 0.106"); or 6d corrosion-resistant casing (2" × 0.099")	6	12	-Nailing from IBC Row 32 and footnote "f".
40	5/8"	8d corrosion-resistant siding (2 3/8" × 0.128"); or 8d corrosion-resistant casing (2 1/2" × 0.113")	6	12	-Nailing from IBC Row 32 and footnote "f".
41	1/4"	4d casing (11/2" × 0.080"); or 4d finish (11/2" × 0.072")	6	12	-Nailing from IBC Row 34 and footnote "j".
42	3/8"	6d casing (2" × 0.099"); or 6d finish (Panel supports at 24 inches)	6	12	-Nailing from IBC Row 34 and footnote "k".

a. Nails spaced at 6 inches at intermediate supports where spans are 48 inches or more. For nailing of wood structural panel and particleboard diaphragms and shear walls, refer to Section 2305. Nails for wall sheathing are permitted to be common, box or casing.

 Spacing shall be 6 inches on center on the edges and 12 inches on center at intermediate supports for nonstructural applications. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise marked).

c. Where a rafter is fastened to an adjacent parallel ceiling joist in accordance with this schedule and the ceiling joist is fastened to the top plate in accordance with this schedule, the number of toenails in the rafter shall be permitted to be reduced by one nail.

Current (Existing) Table 2304.9.1 with additional column indicating new location:

CONNECTION	FASTENING ^{a, m}	LOCATION	Notes:
1. Joist to sill or girder	3-8d common (2 ½ " x 0.131") 3-3" x 0.131" nails 3-3" 14 gage staples	toenail	to new row 22
2. Bridging to joist	2-8d common (2 ½" x 0.131") 2-3" x 0.131" nails 2-3" 14 gage staples	toenail each end	to new row 30
3. 1" x 6" subfloor or less to each joist	2-8d common (2 ½" x 0.131")	face nail	to new row 24
4. Wider than 1" x 6" subfloor to each joist	3-8d common (2 ½" x 0.131")	face nail	deleted from table, wider condition addressed by row 21
5. 2" subfloor to joist or girder	2-16d common (3 1/2" x 0.162")	Blind and face nail	to new row 25
6. sole plate to joist or blocking	16d (3 ½" x 0.135") at 16" o.c. 3" x 0.131" nails at 8 o.c. 3" 14 gage staples at 12" o.c.	typical face nail	to new row 14
Sole plate to joist or blocking at braced wall panel	3-16d (3 ½" x 0.135") at 16" o.c. 4-3" x 0.131" nails at 16" o.c. 4-3" 14 gage staples at 16" o.c.	braced wall panels	to new row 15
7. Top plate to stud	2-16d common (3 ½" x 0.162") 3-3" x 0.131" nails 3-3" 14 gage staples	end nail	to new row 17
8. Stud to sole plate	4-8d common (2 ½" x 0.131") 4-3" x 0.131" nails 3-3" 14 gage staples	toenail	to new row 16 and 17

CONNECTION	FASTENING ^{a, m}	LOCATION	Notes:
	2-16d common (3 1/2" x 0.162")	end nail	to new row 16 and 17
	3-3" x 0.131" nails		
	3-3" 14 gage staples		
9. Double studs	16d $(3 \frac{1}{2} \times 0.135)$ at 24" o.c.	face nail	to new rows 8 and 9
	3" 14 nage staple at 8" o.c.		
10. Double top plates	$16d (3 \frac{1}{2}) \times 0.135") at 16" o.c.$	typical face nail	to new rows 12
	3" x 0.131" nail at 12" o.c.	typical lace that	
	3" 14 gage staple at 8" o.c.		
Double top plates	8-16d common (3 ¹ / ₂ " x 0.162")	lap splice	
	12-3" x 0.131" nails		
11 Placking between iniste or	12-3 14 gage staples		to pour row 1
rafters to top plate	3-3" x 0 131" nails	toenail	to new row r
	3-3" 14 gage staples	toeriai	
12. Rim joist to top plate	8d (2 ½" x 0.131") at 6" o.c.		to new row 23
	3" x 0.131" nail at 6" o.c.	toenail	
	3" 14 gage staple at 6" o.c.		
13. Top plates, laps and	2-16d common (3 1/2" x 0.162")		to new row 18
intersections	3-3" x 0.131" nails	face nail	
	3-3" 14 gage staples	4.07	
14. Continuous header, two pieces	16d common (3 ½ 0.162")	16" o.c. along edge	to new row 10
15. Ceiling joists to plate	3-8d common (2 ½" x 0.131")		to new row 2
	5-3" x 0.131" nails	toenail	
	5-3" 14 gage staples		
16 Continuous header to stud	4-8d common (2 ½" x 0 131")	toenail	to new row 11
47.0 11			
17. Celling joists, laps over partitions	$3-160$ Common ($3 \frac{1}{2}$ X 0.162)	face hall	to new rows 3 and 4
2308 10 4 1)	4_3 x 0 131" nails		
2000.10.4.1)	4-3" 14 gage staples		
18. Ceiling joists to parallel rafters	3-16d common (3 ½" x 0.162")	face nail	to new row 4
(see Section 2308.10.4.1, Table	minimum, Table 2308.10.4.1		
2308.10.4.1)	4-3" x 0.131" nails		
	4-3" 14 gage staples		
19. Rafter to plate (see Section	3-8d common (2 ½" x 0.131")	Face nail	to new row 6
2308.101, Table 2308.10.1)	3-3" x 0.131" nails		
20 1" diagonal braza ta cach atud	3-3 14 gage staples		to now row 10
and plate	$2-3^{\circ} \times 0.131^{\circ}$ naile	Face fian	10 new 10w 19
	3-3" 14 gage staples		
21. 1" x 8" sheathing to each	3-8d common (2 ½" x 0.131")	face nail	to new row 21
bearing			
22. Wider than 1" x 8" sheathing to	3-8d common (2 1/2" x 0.131")	face nail	to new row 21
each bearing			
23. Built-up corner studs	16d common (2 ½" x 0.131")	24" o.c.	to new row 9
	3" X 0.131" nails	16 [°] 0.C.	
24. Ruilt up girder and beams	3 14 gage staples	face pail at top and	to new row 27
24. Built-up girder and beams	$3^{\circ} \times 0.131^{\circ}$ nails @ 24" o c	bottom staggered on	10 new 10w 21
	3" 14 gage staples @ 24" o.c.	opposite sides	
	2-20d common (4" x 0.192")	face nail at ends and at	to new row 27
	3-3" x 0.131" nails @ 24" o.c.	each splice	
	3-3" 14 gage staples @ 24" o.c.		
25. 2" planks	16d common (3 ½" x 0.162")	at each bearing	to new row 26
26. Collar tig to rofter	$2.10d \text{ common } (2" \times 0.149")$	face pail	to now row F
20. Collar tie to rafter	3-100 common (3° X 0.148°) 4-3" x 0.131" paile	lace nall	to new row 5
	4-3 14 dage staples		
27. Jack rafter to hip	3-10d common (3" x 0 148")	toenail	to new row 7
	4-3" x 0.131" nails		
	4-3" 14 gage staples		

CONNECTION	FASTENING ^{a, m}		LOCATION	Notes:
	2-16d common (3	½" x 0.162")	face nail	to new row 7
	3-3" x 0.131" nails	5		
	3-3" 14 gage staples			
28. Roof rafter to 2-by ridge beam	2-16d common (3 1/2" x 0.162")		toenail	to new row 7 except 10d
	3-3" x 0.131" nails	6		common is specified for
	3-3" 14 gage staples			toe-nail case to match
				jack to hip nailing.
	2-16d common (3	½" x 0.162")	face nail	to new row 7
	3-3" x 0.131" hails			
29 loist to band joist	3-16d common (3	$\frac{163}{162}$ 162")	face nail	to new row 29
29. 30131 10 Dana joist	4-3" x 0 131" nails	72 X 0. 102)		10 11610 1010 23
	4-3" 14 gage stap	les		
30. Ledger strip	3-16d common (3	¹ / ₂ " x 0.162")	face nail at each joist	to new row 28
5	4-3" x 0.131" nails	\$,	
	4-3" 14 gage stap	les		
31. Wood structural panels and	1/2" and less	6d ^{c,I}		to new row 31
particleboard		2 3/8" x 0.113"		
Subfloor, roof and wall sheathing (to		nail"		
framing)	40/00" += 3/"	1 % 16 gage	-	
	19/32 to %			to new rows 22 22
		2 3/6 X 0.113		10 116W 10WS 32-33
		2" 16 gage ^p		
	7/8" to 1"	8d ^c		
	1 1/8" to 1 1/4"	10d ^d or 8d ^e		to new rows 36, 37, 38
Single floor (combination subfloor-	³ ⁄ ₄ " and less	6d ^e		
underlayment to framing)	7/8" to 1"	8d ^e		
	1 1/8" to 1 1/4"	10d ^a or 8d ^e		
32. Panel siding (to framing)	¹ / ₂ " or less	6d'		to new rows 39 and 40
	5/8"	8d'		
33. Fiberboard sheathing ⁹	1/2"	No. 11 gage		to new row 34
		roofing nail ⁿ		
		6d common nail		
		(2" x 0.113")		
		No. 16 gage		
	25/32"			to new row 35
	25/32 NO. 11 gage			
		8d common nail		
		(2" x 0.113")		
		No. 16 gage		
		staple ⁱ		
34. Interior paneling	1/4"	4d ¹		to new row 41
	3/8"	6d ^ĸ		to new row 42

For SI: 1 inch = 25.4 mm.

a. common or box nails are permitted to be used except where otherwise stated.

- b. Nails spaced at 6 inches on center at edges, 12 inches at intermediate supports except 6 inches at supports where spans are 48 inches or more. For nailing of wood structural panel and particleboard diaphragms and shear walls, refer to Section 2305. Nails for wall sheathing are permitted to be common, box or casing.
- c. Common or deformed shank (6d-2" x 0.113"; 8d-2 1/2" x 0.131"; 10d-3" x 0.148").
- d. Common (6d-2" x 0.113"; 8d-2 ½" x 0.131"; 10d-3" x 0.148").
- e. Deformed shank (6d-2" x 0.113"; 8d-2 1/2" x 0.131"; 10d-3" x 0.148").
- f. Corrosion-resistant siding (6d-1 7/8 x 0.106"; 8d-2 3/8" x 0.128") or casing (6d-2" x 0.099"; 8d-2 ½" x 0.113") nail.
- g. Fasteners spaced 3 inches on center at exterior edges and 6 inches on center at intermediate supports, when used as structural sheathing. Spacing shall be 6 inches on center on the edges and 12 inches on center at intermediate supports for nonstructural applications.
- h. Corrosion-resistant roofing nails with 7/16-inch-diameter head and d1 1 1/2"-inch length for ½-inch sheathing and 1 ¾-inch length for 25/32-inch sheathing.
- i. Corrosion-resistant staples with nominal 7/16-inch crown or 1-inch crown and 1 ¼-inch length for ½-inch sheathing and 1 ½inch length for 25/32-inch sheathing. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise marked).
- j. Casing (1 1/2" x 0.080") or finish (1 1/2" x 0.072") nails spaced 6 inches on panel edges, 12 inches at intermediate supports
- k. Panel supports at 24 inches. Casing or finish nails spaced 6 inches on panel edges, 12 inches at intermediate supports.
- I. For roof sheathing applications, 8d nails (2 1/2" x 0.113") are the minimum required for wood structural panels.
- m. Staples shall have a minimum crown width of 7:16 inch.

n. For roof sheathing applications, fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports.

- o. Fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports for subfloor and wall sheathing and 3 inches on center at edges, 6 inches at intermediate supports for roof sheathing.
- p. Fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports.

(The 2012 IRC fastener schedule is shown below for reference)

ITEM	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER ^{a, b, c}	SPACING OF FASTENERS
		Roof	
1	Blocking between joists or rafters to top plate, toe nail	3-8d (2 ¹ / ₂ " × 0.113 ")	
2	Ceiling joists to plate, toe nail	3-8d (2 ¹ / ₂ " × 0.113 ")	
3	Ceiling joists not attached to parallel rafter, laps over parti- tions, face nail	3-10d	-
4	Collar tie to rafter, face nail or $1^{1}/_{4}$ " × 20 gage ridge strap	3-10d (3 "× 0.128")	
5	Rafter or roof truss to plate, toe nail	3-16d box nails (3 ¹ / ₂ " × 0.135 ") or 3-10d common nails (3 " × 0.148 ")	2 toe nails on one side and 1 toe nail o opposite side of each rafter or truss ¹
6	Roof rafters to ridge, valley or hip rafters: toe nail face nail	4-16d (3 ¹ / ₂ " × 0.135") 3-16d (3 ¹ / ₂ " × 0.135")	
		Wall	
7	Built-up studs-face nail	10d (3 " × 0.128 ")	24 " o.c.
8	Abutting studs at intersecting wall corners, face nail	16d (3 ½" x 0.135")	12 " o.c.
9	Built-up header, two pieces with 1/2" spacer	$16d (3^{1}/_{2} \times 0.135)$	16 "o.c. along each edge
10	Continued header, two pieces	16d (3 ¹ / ₂ "× 0.135 ")	16 "o.c. along each edge
11	Continuous header to stud, toe nail	4-8d (2 ¹ / ₂ "× 0.113")	
12	Double studs, face nail	10d (3 " × 0.128 ")	24 "o.c.
13	Double top plates, face nail	10d (3 " × 0.128 ")	24 ″o.c.
14	Double top plates, minimum 24-inch offset of end joints, face nail in lapped area	8-16d (3 ¹ / ₂ " × 0.135 ")	
15	Sole plate to joist or blocking, face nail	16d (3 ¹ / ₂ "× 0.135 ")	16 "o.c.
16	Sole plate to joist or blocking at braced wall panels	3-16d (3 ¹ / ₂ " × 0.135")	16 ″o.c.
17	Stud to sole plate, toe nail	3-8d (2 ¹ / ₂ " × 0.113 ") or 2-16d (3 ¹ / ₂ " × 0.135 ")	-
18	Top or sole plate to stud, end nail	$2-16d (3^{1}/_{2}" \times 0.135")$	
19	Top plates, laps at corners and intersections, face nail	2-10d (3 "× 0.128")	
20	1 "brace to each stud and plate, face nail	2-8d (2 ¹ / ₂ " × 0.113") 2 staples 1 ³ / ₄ "	
21	$1~^{\prime\prime}\times6~^{\prime\prime}$ sheathing to each bearing, face nail	2-8d (2 ¹ / ₂ " × 0.113") 2 staples 1 ³ / ₄ "	
22	$1~^{\prime\prime}\times8~^{\prime\prime}$ sheathing to each bearing, face nail	2-8d (2 ¹ / ₂ " × 0.113") 3 staples 1 ³ / ₄ "	
23	Wider than 1 $'' \times 8$ '' sheathing to each bearing, face nail	3-8d (2 ¹ / ₂ " × 0.113") 4 staples 1 ³ / ₄ "	1
		Floor	
24	Joist to sill or girder, toe nail	3-8d (2 ¹ / ₂ " × 0.113")	
25	Rim joist to top plate, toe nail (roof applications also)	8d (2 ¹ / ₂ "× 0.113")	6 ″ o.c.
26	Rim joist or blocking to sill plate, toe nail	8d (2 ½ "× 0.113")	6 " o.c.
27	$1\ "\times 6\ "$ subfloor or less to each joist, face nail	2-8d (2 ¹ / ₂ " × 0.113") 2 staples 1 ³ / ₄ "	E
28	2 "subfloor to joist or girder, blind and face nail	$2-16d (3^{1}/_{2}" \times 0.135")$	· · · · · · · · · · · · · · · · · · ·
29	2 "planks (plank & beam - floor & roof)	2-16d (3 ¹ / ₂ " × 0.135")	at each bearing
30	Built-up girders and beams, 2-inch lumber layers	10d (3 " × 0.128 ")	Nail each layer as follows: 32 " o.c. at top and bottom and staggered. Two nails at ends and at each splice.
31	Ledger strip supporting joists or rafters	3-16d (3 ¹ / ₂ " × 0.135 ")	At each joist or rafter

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

(continued)

2012 INTERNATIONAL RESIDENTIAL CODE®

6.6	in an an an an an an an an		SPAC	ING OF FASTENERS
ITEM DESCRIPTION OF BUILDING MATERIALS		DESCRIPTION OF FASTENER ^{b.c.e}	Edges (inches) ⁱ	Intermediate supports ^{c, e} (inches)
	Wood structural panels, subfloor, r	oof and interior wall sheathing to framing and particleboard	wall sheathin	g to framing
32	³ / ₈ " - ¹ / ₂ "	6d common (2 " \times 0.113 ") nail (subfloor wall) ¹ 8d common (2 ¹ / ₂ " \times 0.131 ") nail (roof) ^f	6	12 ^g
33	¹⁹ / ₃₂ " - 1 "	8d common nail (2 ¹ / ₂ " × 0.131 ")	6	12 ^g
34	1 ¹ / ₈ " = 1 ¹ / ₄ "	10d common (3 " × 0.148 ") nail or 8d ($2^{1}/_{2}$ " × 0.131 ") deformed naìl	6	12
		Other wall sheathing ^h		
35	¹ / ₂ " structural cellulosic fiberboard sheathing	$^{1\!/_{2}}$ " galvanized roofing nail, $^{7\!/_{16}}$ " crown or 1 " crown staple 16 ga., $1^{1\!/_{4}}$ " long	3	6
36	²⁵ / ₃₂ " structural cellulosic fiberboard sheathing	$1^{3}\!/_{4}{}''$ galvanized roofing nail, ${}^{7}\!/_{16}{}''$ crown or $1{}''$ crown staple 16 ga., $1^{1}\!/_{2}{}''$ long	3	6
37	¹ / ₂ "gypsum sheathing ^d	$1^{1/_2}$ " galvanized roofing nail; staple galvanized, $1^{1/_2}$ " long; $1^{1/_4}$ screws, Type W or S	7	7
38	⁵ / ₈ "gypsum sheathing ^d	$1^{3/4}$ " galvanized roofing nail; staple galvanized, $1^{5/8}$ " long; $1^{5/8}$ " screws, Type W or S	7	7
10.00	Wood	structural panels, combination subfloor underlayment to fra	ming	
39	$^{3}/_{t}$ " and less	6d deformed (2 " × 0.120 ") nail or 8d common ($2^{1}/_{2}$ " × 0.131 ") nail	6	12
40	⁷ / ₈ "- 1 "	8d common (2 ¹ / ₂ " × 0.131 ") nail or 8d deformed (2 ¹ / ₂ " × 0.120 ") nail	6	12
41	1 ¹ / ₈ "-1 ¹ / ₄ "	10d common (3 " \times 0.148 ") nail or 8d deformed (2 ¹ / ₂ " \times 0.120 ") nail	6	12

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

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

a. All nails are smooth-common, box or deformed shanks except where otherwise stated. Nails used for framing and sheathing connections shall have minimum average bending yield strengths as shown: 80 ksi for shank diameters of 0.192 inch (20d common nail), 90 ksi for shank diameters larger than 0.142 inch but not larger than 0.177 inch, and 100 ksi for shank diameters of 0.142 inch or less.

b. Staples are 16 gage wire and have a minimum $^{7\!/}_{16}\text{-inch on diameter crown width.}$

c. Nails shall be spaced at not more than 6 inches on center at all supports where spans are 48 inches or greater.

d. Four-foot by 8-foot or 4-foot by 9-foot panels shall be applied vertically.

e. Spacing of fasteners not included in this table shall be based on Table R602.3(2).

f. For regions having basic wind speed of 110 mph or greater, 8d deformed (2¹/_z "× 0.120) nails shall be used for attaching plywood and wood structural panel roof sheathing to framing within minimum 48-inch distance from gable end walls, if mean roof height is more than 25 feet, up to 35 feet maximum.

g. For regions having basic wind speed of 100 mph or less, nails for attaching wood structural panel roof sheathing to gable end wall framing shall be spaced 6 inches on center. When basic wind speed is greater than 100 mph, nails for attaching panel roof sheathing to intermediate supports shall be spaced 6 inches on center for minimum 48-inch distance from ridges, eaves and gable end walls; and 4 inches on center to gable end wall framing.

h. Gypsum sheathing shall conform to ASTM C 1396 and shall be installed in accordance with GA 253. Fiberboard sheathing shall conform to ASTM C 208.

i. Spacing of fasteners on floor sheathing panel edges applies to panel edges supported by framing members and required blocking and at all floor perimeters only. Spacing of fasteners on roof sheathing panel edges applies to panel edges supported by framing members and required blocking. Blocking of roof or floor sheathing panel edges perpendicular to the framing members need not be provided except as required by other provisions of this code. Floor perimeter shall be supported by framing members or solid blocking.

j. Where a rafter is fastened to an adjacent parallel ceiling joist in accordance with this schedule, provide two toe nails on one side of the rafter and toe nails from the ceiling joist to top plate in accordance with this schedule. The toe nail on the opposite side of the rafter shall not be required.

S265-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				T2304.9.1-S-BAJNAI-BCAC.doc

S266–12 Table 2304.9.1

Proponent: Paul Coats, American Wood Council (pcoats@awc.org)

Revise as follows:

TABLE 2304.9.1 FASTENING SCHEDULE

CONNECTION		FASTENING ^{a,m}	LOCATION
33. Fiberboard sheathing ^g	1/2"	No. 11 gage roofing nail ^h 6d common nail (2" × 0.113") No. 16 gage staple ⁱ	
	25/32"	No. 11 gage roofing nail ^h 8d common nail (21/2" × 0.131") No. 16 gage staple ⁱ	

(Portions of table not shown remain unchanged)

Reason: Recommended fastening for fiberboard sheathing no longer includes 6d common or 8d common nails. Removal of these common nail fastener sizes coordinates with revisions made in the 2008 Special Design Provisions for Wind and Seismic (SDPWS) and 2012 Wood Frame Construction Manual (WFCM) referenced in this code and applicable for design of structural fiberboard shear walls. Specified roofing nails and staples incorporate a larger head/crown size per footnotes h and i for increased head pull-through resistance.

S266-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				T2304.9.1-S-COATS.doc

S267–12 Table 2304.9.1

Proponent: Jay Crandell, ARES Consulting, representing the Foam Sheathing Committee of the American Chemistry Council- Plastics Division (jcrandell@aresconsulting.biz)

Revise as follows:

TABLE 2304.9.1 FASTENING SCHEDULE

	TASTENING SCHEDOLE	
7. Top plate to stud	2 - 16d common (3 ¹ / ₂ " × 0.162") 3 - 3" × 0.131" nails 3 - 3" 14 gage staples	end nail
	$\frac{4 - 8d \text{ common } (21/2" \times 0.131")}{4 - 3" \times 0.131" \text{ nails}}$ $\frac{3 - 3" + 4 \text{ gage staples}}{3 - 3" + 4 \text{ gage staples}}$	<u>toenail</u>

(Portions of table not shown remain unchanged)

Reason: The code already provides a toenail connection option for the stud to bottom plate connection (see Item 8 in the same table). This code change proposal makes requirements consistent for connection of the stud to the top plate. Toe nail connections provide a better uplift load path than end nails, so this option should be provided for both ends of the stud, not just at the bottom end of the stud.

S267-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				T2304.9.1-S-CRANDELL.doc

S268-12

2304.11, 2304.11.1, 2304.11.2, 2304.11.2.1, 2304.11.2.2, 2304.11.2.3, 2304.2.4, 2304.11.2.5, 2304.11.2.6, 2304.11.2.7, 2304.11.3, 2304.11.4, 2304.11.4.1, 2304.11.4.2, 2304.11.5, 2304.11.6, 2304.11.7

Proponent: Dennis Pitts, American Wood Council, (dpitts@awc.org)

Revise as follows:

2304.11 Protection against decay and termites. Wood shall be protected from decay and termites in accordance with the applicable provisions of Sections 2304.11.1 through <u>2304.11.9</u> <u>2304.11.7</u>.

2304.11.1 General. Where required by this section, protection from decay and termites shall be provided by the use of naturally durable or *preservative-treated wood*.

2304.11.2 Wood used above ground 2304.11.1 Location requiring water-borne preservatives. Wood used above ground in the locations specified in Sections 2304.11.2.1 2304.11.1.1 through 2304.11.2.7 2304.11.1.5, 2304.11.3 and 2304.11.5 shall be naturally durable wood or *preservative-treated wood* using water-borne preservatives, in accordance with AWPA U1 (Commodity Specifications A or F) for above-ground use.

2304.11.2.1 2304.11.1.1 Joists, girders and subfloor. Where Wood joists or the bottom of a wood structural floor without joists are closer than 18 inches (457 mm), or wood girders are closer than 12 inches (305 mm) to the exposed ground in crawl spaces or unexcavated areas located within the perimeter of the building foundation, the floor construction (including posts, girders, joists and subfloor) shall be of naturally durable or *preservative-treated wood*.

2304.11.2.2 2304.11.1.2 Wood supported by exterior foundation walls. Wood framing members, including wood sheathing, that rest on <u>are in contact with</u> exterior foundation walls and are less than 8 inches (203 mm) from exposed earth shall be of naturally durable or *preservative-treated wood*.

2304.11.2.3 2304.11.1.3 Exterior walls below grade. Wood framing members and furring strips attached directly to in direct contact with the interior of exterior masonry or concrete walls below grade shall be of naturally durable or *preservative-treated wood*.

2304.11.2.4 2304.11.1.4 Sleepers and sills. Sleepers and sills on a concrete or masonry slab that is in direct contact with earth shall be of naturally durable or *preservative-treated wood*.

2304.11.2.6 <u>2304.11.1.5</u> Wood siding. Clearance between wood siding and earth on the exterior of a building shall not be less than 6 inches (152 mm) or less than 2 inches (51 mm) vertical from concrete steps, porch slabs, patio slabs and similar horizontal surfaces exposed to the weather except where siding, sheathing and wall framing are of naturally durable or *preservative-treated wood*.

2304.11.2 Other locations. Wood used in the locations specified in Sections 2304.11.2.1 through 2304.11.2.5 shall be naturally durable wood or *preservative treated wood* in accordance with AWPA U1. Preservative treated wood used in interior locations shall be protected with two coats of urethane, shellac, latex epoxy, or varnish unless waterborne preservatives are used. Prior to application of the protective finish, the wood shall be dried in accordance with the manufacturer's recommendations.

2304.11.2.5 <u>2304.11.2.1</u> Girder ends. The ends of wood girders entering exterior masonry or concrete walls shall be provided with a 1/2-inch (12.7 mm) air space on top, sides and end, unless naturally durable or preservative-treated wood is used.

2304.11.2.7 2304.11.2.2 Posts or columns. Posts or columns supporting permanent structures and supported by a concrete or masonry slab or footing that is in direct contact with the earth shall be of naturally durable or *preservative-treated wood*.

Exceptions:

- Posts or columns that are either not exposed to the weather or located in basements or cellars, are supported by concrete piers or metal pedestals projected at least 1 inch (25 mm) above the slab or deck and 6 8 inches (152 mm) above exposed earth, and are separated therefrom by an impervious moisture barrier.
- 2. Posts or columns in enclosed crawl spaces or unexcavated areas located within the periphery of the building, supported by a concrete pier or metal pedestal at a height greater than 8 inches (203 mm) from exposed ground, and are separated therefrom by an impervious moisture barrier.

2304.11.5 2304.11.2.3 Supporting member for permanent appurtenances. Naturally durable or *preservative-treated wood* shall be utilized for those portions of wood members that form the structural supports of buildings, balconies, porches or similar permanent building appurtenances where such members are exposed to the weather without adequate protection from a roof, eave, overhang or other covering to prevent moisture or water accumulation on the surface or at joints between members.

Exception: When a building is located in a geographical region where experience has demonstrated that climatic conditions preclude the need to use durable materials where the structure is exposed to the weather.

2304.11.3 2304.11.2.4 Laminated timbers. The portions of glued-laminated timbers that form the structural supports of a building or other structure and are exposed to weather and not fully protected from moisture by a roof, eave or similar covering shall be pressure treated with preservative or be manufactured from naturally durable or *preservative-treated wood*.

2304.11.2.5. Supporting members for permeable floors and roofs. Wood structural members that support moisture-permeable floors or roofs that are exposed to the weather, such as concrete or masonry slabs, shall be of naturally durable or *preservative-treated wood* unless separated from such floors or roofs by an impervious moisture barrier.

2304.11.4 <u>2304.11.3</u> Wood in contact with the ground or fresh water. Wood used in contact with the ground (exposed earth) in the locations specified in Sections 2304.11.4.1 and 2304.11.4.2 shall be naturally durable (species for both decay and termite resistance) or preservative treated using waterborne preservatives in accordance with AWPA U1 (Commodity Specifications A or F) for soil or fresh water use.

Exception: Untreated wood is permitted where such wood is continuously and entirely below the groundwater level or submerged in fresh water.

2304.11.4.1 2304.11.3.1 Posts or columns. Posts and columns supporting permanent structures that are embedded in concrete that is in direct contact with the earth, embedded in concrete that is exposed to the weather or in direct contact with the earth shall be of *preservative-treated wood*.

2304.11.4.2 Wood structural members. Wood structural members that support moisture-permeable floors or roofs that are exposed to the weather, such as concrete or masonry slabs, shall be of naturally durable or *preservative-treated wood* unless separated from such floors or roofs by an impervious moisture barrier.

2304.11.6 <u>2304.11.4</u> Termite protection. In geographical areas where hazard of termite damage is known to be very heavy, wood floor framing in the locations specified in Section 2304.11.1.1 and exposed framing of exterior decks or balconies shall be of naturally 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.

2304.11.7 2304.11.5 Wood used in retaining walls and cribs. Wood installed in retaining or crib walls shall be preservative treated in accordance with AWPA U1 (Commodity Specifications A or F) for soil and fresh water use.

Reason: This code change contains few technical changes but addresses many editorial clean-ups and some_re-organization. The technical change is a delineation of exactly where waterborne preservatives should be required and where they should not. In a reorganization of this section in the 2005 code change cycle, glued laminated and certain exterior applications were lumped under a general section for the purposes of citing the new AWPA U1 standard, but a requirement for waterborne preservatives was inadvertently imposed for all applications in that_reorganization. This proposed code change restores the ability for glued laminated beams and wood in exterior applications to be treated with other-than waterborne preservatives in accordance with the U1 standard. As a precaution, a requirement for the drying of treated wood and its sealing was added where used on the interior of a building (proposed section 2304.11.2).

Other changes are explained as follows:

Existing section 2304.11.1 deletion: This section became superfluous.

Proposed 2304.11.1: Section references are changed, and the specific mention of commodity specifications in the U1 standard was deleted because it is unnecessary.

Proposed 2304.11.1.1: Removing "the floor construction (including posts, girders, joists and subfloor)" makes it clear_that only those floor elements within proximity to exposed ground need to be protected.

Proposed 2304.11.1.2: Better wording to meet current intent.

Proposed 2304.11.1.3: Better wording to meet current intent.

Proposed 2304.11.2: This new section is needed to introduce the subsections for locations where other-than waterborne preservatives are permitted under certain circumstances, as long as treatment is in accordance with the AWPA U1 standard.

Proposed 2304.11.2.2 Exceptions: The first exception was worded incorrectly and would seem to exempt exposed wood from protection; the proposed wording is a fix. With Exception 1 fixed, exception 2 was so similar in requirement that it was combined with Exception 1 and the clearance dimension was changed from 6 to 8 inches to preserve the intent of the deleted exception and be consistent with the clearance required for wood supported by exterior foundation walls in proposed Section 2304.11.1.2.

Proposed 2304.11.2.5: This is not a new section, but is re-titled and moved up in the text from Section 2304.11.4.2 (shown struck-out further down). There is no obvious reason why it must be a subsection of current 2304.11.4.

Proposed 2304.11.3: The requirement that water-borne preservatives be used exclusively has been struck in accordance with the purpose of this change, which indicates those locations where water-borne preservatives must be used up in proposed Section 2304.11.1 and subsections.

Existing section 2304.11.4.1 and 2304.11.4.2 (shown struck out): These were not lost. The current 2304.11.4.2 was moved up to become_proposed 2304.11.2.5, and the current 2304.11.4.1 became 2304.11.3.1 with some editorial rewording for clarity.

S268-12					
Public Hearing:	Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF		
				2304.11-S-PITTS.doc	

S269–12 2304.11.2, 2304.11.3, 2304.11.4, 2304.11.5, 2304.11.7

Proponent: Paul Coats, P.E., CBO, American Wood Council (pcoats@awc.org)

Revise as follows:

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

2304.11.3 Laminated timbers. The portions of glued-laminated timbers that form the structural supports of a building or other structure and are exposed to weather and not fully protected from moisture by a roof, eave or similar covering shall be <u>pressure preservative</u> treated with preservative in accordance with <u>AWPA U1</u> or be manufactured from naturally durable or *preservative-treated wood*.

Unless waterborne preservatives are used, preservative treated glued laminated timbers used in interior locations shall be protected with two coats of urethane, shellac, latex epoxy, or varnish. Prior to application of the protective finish, the wood shall be dried in accordance with the manufacturer's recommendations.

2304.11.4 Wood in contact with the ground or fresh water. Wood used in contact with the ground (exposed earth) in the locations specified in Sections 2304.11.4.1 and 2304.11.4.2 shall be naturally durable (species for both decay and termite resistance) or preservative treated using water-borne preservatives in accordance with AWPA U1 (Commodity Specifications A or F) for soil or fresh water use.

Exception: Untreated wood is permitted where such wood is continuously and entirely below the groundwater level or submerged in fresh water.

2304.11.5 Supporting member for permanent appurtenances. Naturally durable wood or wood that is preservative-treated wood in accordance with AWPA U1 shall be utilized for those portions of wood members that form the structural supports of buildings, balconies, porches or similar permanent building appurtenances where such members are exposed to the weather without adequate protection from a roof, eave, overhang or other covering to prevent moisture or water accumulation on the surface or at joints between members.

Exception: When a building is located in a geographical region where experience has demonstrated that climatic conditions preclude the need to use durable materials where the structure is exposed to the weather.

2304.11.7 Wood used in retaining walls and cribs. Wood installed in retaining or crib walls shall be preservative treated in accordance with AWPA U1 (Commodity Specifications A or F) for soil and fresh water use.

Reason: It is common practice for glued-laminated structural members to be treated with other than water-borne preservatives. In the 2006 IBC, a change was introduced which re-organized the preservative treated wood section and inadvertently imposed a water-borne preservative mandate on glued laminated timbers and exterior applications of other structural members (see code change S51-03/04). The reason for the original change was to bring all applications under the new AWPA U1 standard. This proposed change retains the intent of having the U1 standard apply to all applications but enables current industry practice for the use of other preservatives for exterior and glued-laminated applications. As a precaution against air quality concerns, wording is introduced to require the drying and sealing of glued laminated members when used in interior applications, if water-borne preservatives are not used. Also, the limitation to Commodity Specifications A or F is no longer necessary since the standard now clearly indicates the applicability limits of its various specifications.

S269-12					
Public Hearing:	Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF		
	•				2304.11.2-S-COATS.doc

S270–12 2304.11.2.1

Proponent: Edward L. Keith, P.E., APA The Engineered Wood Association (ed.keith@apawood.org)

Revise as follows:

2304.11.2.1 Joists, girders and subfloor. Where wood joists or the bottom of a wood structural floor without joists are closer than 18 inches (457 mm), or wood girders are closer than 12 inches (305 mm) to the exposed ground in crawl spaces or unexcavated areas located within the perimeter of the building foundation, the floor construction (including posts, girders, joists and subfloor) shall be of naturally durable or *preservative-treated wood*.

Exception: The clearance between the wood floor joists or the bottom of a wood structural floor without joists and the exposed ground shall be permitted to be reduced to 12 inches in areas where all of the following conditions have been met:

- 1. The grade within the perimeter of the foundation slopes away from the floor framing in such a way as to make all of the floor joists, the bottom of a wood structural floor without joists, and girders readily accessible.
- 2. The average distance between grade and the bottom of the wood floor joists or the bottom of a wood structural floor without joists is at least 18", and
- 3. The foundation is ventilated in accordance with Section 1203.3.

Reason: The purpose of this proposal is to reduce the raised-wood floor clearance height only in those areas that doing so will not adversely impact the long-term performance of the floor system. The provisions of this exception accomplish two things:

- It recognizes the method of floor construction whereby the floor joists are face-mounted to the side of the girders and occupy the same depth as the girder. As such, both portions of the system would have the same clearance above grade. If 12 inches of clearance from grade provides sufficient moisture separation for girders, the same distance will provide sufficient moisture separation for floor joists as well.
- 2) Recognizing that at least part of the reason for requiring 18 inches of clearance under floor joists is to provide accessibility to the underside of the floor, permitting the separation to be reduced to 12 inches can only be made under three conditions specified to maintain access to the under-floor area and still provide sufficient ventilation to insure the serviceability of the floor system through moisture control. These conditions are as follows:
 - a. This reduction in separation for the floor joists or the bottom of a wood structural floor without joists is only permitted in foundations where the ground slopes away from the floor in a way such that only readily accessible portions of the floor may meet this reduced separation requirement.
 - b. The minimum average clearance requirement is provided to insure sufficient slope is present to provide the ready accessibility to the portions of the floor meeting the new reduced clearance to grade requirement.

c. This exception is applicable only to those under-floor spaces meeting the ventilation requirements of Section 1203.3. It is anticipated that this exception as written will provide some cost savings to buildings constructed on sloped sites by reducing the crawl space height dictated by a small percent of the floor area while providing similar accessibility and serviceability as is intended by the Section 2304.11.2.1.

It also recognizes the increasingly common low profile floors whereby the floor joists are hung off of the sides of the floor girders. These are becoming increasingly popular as builders discover the availability of engineered wood beams made in I-joist-compatible depths. The use of such beams and girders had the potential to reduce first-floor elevations by 6 to 9 inches with a corresponding savings in labor and construction materials.

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

S270-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2304.11.2.1-S-KEITH.doc
S271–12 2304.11.2.7

Proponent: Dennis Pitts, American Wood Council, (dpitts@awc.org)

Revise as follows:

2304.11.2.7 Posts or columns. Posts or columns supporting permanent structures and supported by a concrete or masonry slab or footing that is in direct contact with the earth shall be of naturally durable or *preservative-treated wood*.

Exception:

- Posts or columns that are either not exposed to the weather or located in basements or cellars, and are supported by concrete piers or metal pedestals projected at least 1 inch (25 mm) above the slab or deck and 6 inches (152 mm) 8 inches (203 mm) above exposed earth, and are separated therefrom by an impervious moisture barrier.
- Posts or columns in enclosed crawl spaces or unexcavated areas located within the periphery of the building, supported by a concrete pier or metal pedestal at a height greater than 8 inches (203 mm) from exposed ground, and are separated therefrom by an impervious moisture barrier.

Reason: The current wording of Exception 1 conflicts with requirements in 2304.11.3 and 2304.11.5 and is technically incorrect. Those sections make it clear that all posts or columns exposed to the weather must be protected, regardless of location. The proposed wording removes any confusion. Exceptions 1 and 2 are very similar and with the proposed clean-up of the first exception, the second is superfluous except for the dimension of 8 inches, which differs from the 6-inch dimension in Exception 1. For consistency with other requirements such as for wood supported by exterior foundation walls in 2304.11.2.2, the 8-inch clearance dimension is preferable.

A comparable revision is made as part of the general cleanup and reorganization of this section proposed by AWC in a separate proposal.

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

S271-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2304.11.2.7-S-PITTS.doc

S272-12 2301.2, 2308

Proponent: Charles S. Bajnai, Chesterfield County, VA, (bajnaic@chesterfield.gov)

Revise as follows:

2301.2 General design requirements. The design of structural elements or systems, constructed partially or wholly of wood or wood-based products, shall be in accordance with one of the following methods:

- 1. Allowable stress design in accordance with Sections 2304, 2305 and 2306.
- 2. Load and resistance factor design in accordance with Sections 2304, 2305 and 2307.
- 3. Conventional light-frame construction in accordance with Sections 2304 and 2308.

Exceptions:

- <u>1.</u> Buildings designed in accordance with the provisions of the AF&PA WFCM shall be deemed to meet the requirements of the provisions of Section 2308.
- 2. Buildings designed in accordance to the prescriptive structural provisions of the International Residential Code, shall be permitted in accordance with Section 2303 limitations.
- 4. The design and construction of log structures shall be in accordance with the provisions of ICC 400.

Delete and substitute as follows:

SECTION 2308 CONVENTIONAL LIGHT-FRAME CONSTRUCTION

SECTION 2308 CONVENTIONAL LIGHT-FRAME CONSTRUCTION

2308.1 General. For purposes of defining the structural requirements for buildings using conventional light-frame construction, the International Residential Code (IRC) shall be permitted to be used. The limitations for height, area, egress and the necessity for sprinklers are not to be modified. If there are differences encountered between the IBC and the IRC, the IBC or accepted engineering practice shall govern.

Other construction methods are permitted to be used in accordance with Section 2301.2.

2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of this section and the International Residential Code subject to the following limitations:

- 1. Buildings shall be limited to a maximum of three stories above grade plane.
- 2. <u>Maximum floor-to-floor height shall not exceed 11 feet, 7 inches in height.</u> Bearing wall height shall not exceed a stud height of 12 feet.
- 3. Live load for floors shall not exceed 40 psf when built on conventional light-frame construction.
- V_{asd} shall be 109 mph or less.
- 5. <u>Structures of Risk Category I, II, or III.</u>
- 6. <u>Seismic Design Category A thru D₂</u>. <u>Structures in SDC E shall conform to the wall bracing</u> requirements of Seismic Design Category D₂ as established in the IRC.

2308.3. Climatic and Geographic Design Criteria. The Climatic and Geographic Design Criteria in IRC Table R301.2(1) and the related maps and tables shall be applied.

Reason: There are intrinsic deficiencies in the IBC, Section 2308.

I. IBC Section 2308 is prescriptive code. It tells the designer everything from anchor bolt location and sizes to floor joist, ceiling joist and header tables. What is the difference between house construction and a small office building constructed of wood? My contention is that there is nothing different. This code change is intended to tell the designer/builder that the requirements for *conventional light-frame construction* are sufficient for the buildings and structure within the scope of the IRC and are equally sufficient for other buildings and structures other than one and two family dwellings.

- II. IBC Section 2308 has numerous technical flaws in its engineering, namely:
 - 1. The wall bracing requirements were set up in the IBC for seismic events, not wind events. What this means is that the building exposed to wind events (more than half of the country) may be adequately strong in its long dimension, but weak/under designed in its narrow dimension. See example at the end of this reason statement.
 - 2. The IBC does not take into consideration that the lateral loads are accumulative from the upper floors down to the lower floors. All of the floors are treated as if they are the same with respect to the amount of bracing required. This is not a true phenomenon.

The IRC addresses this in both the wind and seismic tables – the length of bracing on the lower levels is more than the bracing required on the upper levels.

3. The IBC does not specify the minimum length of bracing required. Instead, it has a table, 2308.9.3.1 which shows "X" for the methods allowed and references back to the location requirement of not more than 12.5 feet from braced wall corners. The IBC tries to control the required length of bracing based on the panel spacing – this is like trying to control gas mileage of a car based on the tire size...they just don't have too much influence on each other.

III. IBC Section 2308 does not allow the engineer/builder to gain the benefits of other advancements that have been added to the IRC, namely:

- 1. There are wall bracing methods in the IRC that should be available to structures using the IBC, namely, continuous
- wall bracing methods. The current IBC provides no benefit for continuously sheathing the exterior of the structure.

The IRC actually allows for stud height of 12 feet. The IBC limitations in Section 2308 limits stud heights to 10 feet.
 In the IBC there is no difference if the wind is ≤90 mph (over half of the country) or <110 mph. By inference then, the

In the IBC there is no difference if the wind is ≤90 mph (over half of the country) or <
 IBC provisions must be designing for the worst case situation for all structures.

The Ad-hoc Committee on Wall Bracing (AHWB) spent over five years getting the 2012 IRC to correctly reflect the technical differences between wind and seismic events. The rules are different and the amount of bracing required is different depending on which event governs.

Unfortunately, the IBC was not in the purview of the AHWB committee. Therefore no effort was put into coordinating the IRC with the IBC. Part of the problem had to deal with the fact that the wind and seismic provisions in the IBC are based on ASCE 7 while the wind and seismic provisions in the IRC are based on prescriptive maps.

This code change is not intended to change or negate any of the height and area nor the egress requirements, in the IBC. Likewise, this code change is not intended to say that all structures using the IRC must now be sprinklered. This code change is intended to say that the prescriptive requirements in the IRC can be used for of structures besides houses.

Why not just copy the IRC tables into the IBC?

Some experts may suggest that if the IBC is flawed then the wall bracing requirements in the IRC should be copied over in their entirety to the IBC. It is nearly impossible to just drop the wind tables and seismic tables directly into the IBC; the two books do not have a hand and glove relationship - they are better off remaining distinct.

The AHWC committee in its effort to be more flexible took the six pages from Section R602.10 in the 2006 IRC and made it into thirty pages in the 2012 IRC. Do we really want to do this again??? Even if we did, copying the IRC wall bracing provisions into the IBC does not rectify the original prescriptive language problem inherent in Section 2308.

Example of the difference between the IRC and the IBC with regard to wall bracing assuming most liberal code allowances:



Example of the difference between the IRC and the IBC with regard to wall bracing assuming most liberal code allowances:



Conclusion:

The length of bracing for this one example shows that if wind is blowing north/south against the long face of the structure, the IBC has under designed the length of bracing required over fifty percent.

This code change is good for the IBC and should be passed.

Conclusion:

The length of bracing for this one example shows that if wind is blowing north/south against the long face of the structure, the IBC has under designed the length of bracing required over fifty percent. This code change is good for the IBC and should be passed.

Cost Impact: The impact of this change could be a significant cost reduction by allowing the prescriptive code to be used on small commercial projects that would otherwise require an engineered design. The exact amount of savings will be dependent on the cost of the architectural/engineering services. The cost of the construction should be the same.

S272-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2301-S-BAJNAI-RICE.doc

S273-12 2308 (NEW)

Proponent: Robert Rice, Josephine County, OR (structdesigner@yahoo.com)

Delete and substitute as follows:

SECTION 2308 CONVENTIONAL LIGHT-FRAME CONSTRUCTION

SECTION 2308 CONVENTIONAL LIGHT-FRAME CONSTRUCTION

2308.1 General. The requirements of this section are intended for *conventional light-frame construction*. Other construction methods are permitted to be used, provided a satisfactory design is submitted showing compliance with other provisions of this code. Interior non-load-bearing partitions, ceilings and curtain walls of *conventional light-frame construction* are not subject to the limitations of section 2308.2. Alternatively, compliance with AF&PA WFCM shall be permitted subject to the limitations therein and the limitations of this code. Detached one- and two-family dwellings and multiple single-family dwellings (townhouses) not more than three *stories above grade plane* in height with a separate *means of egress* and their accessory structures shall comply with the *International Residential Code*.

2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of conventional light-frame construction, subject to the following limitations;

2308.2.1 Stories. Structures of *conventional light-frame construction* shall be limited in story height according to Table 2308.2.1

Seismic Design Category	Allowable Story above grade plane
A and B	Three stories
<u>C</u>	Two Stories
<u>D and E ^a</u>	<u>One story</u>

TABLE 2308.2.1 ALLOWABLE STORY HEIGHT

a. For the purposes of this section, for buildings assigned to Seismic Design Category D or E, cripple walls shall be considered to be a story unless cripple walls are solid blocked and do not exceed 14 inches in height,

2308.2.2 Allowable floor-to-floor height. Maximum floor-to-floor height shall not exceed 11 feet, 7 inches (3531 mm). Exterior bearing wall and interior braced wall heights shall not exceed a stud height of 10 feet (3048 mm).

2308.2.3 Allowable Loads. Loads shall be in accordance with Chapter 16 and shall not exceed the following:

1. Average dead loads shall not exceed 15 psf (718 N/m²) for combined roof and ceiling, exterior walls, floors and partitions.

Exceptions:

 Subject to the limitations of Section 2308.6.10.2, 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. <u>Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance</u> <u>with the provisions of this code.</u>
- 2. Live loads shall not exceed 40 psf (1916 N/m²) for floors.
- 3. Ground snow loads shall not exceed 50 psf (2395 N/m²).

2308.2.4 Allowable wind speed. V_{asd} as determined in accordance with Section 1609.3.1 shall not exceed 100 miles per hour (mph) (44 m/s) (3-second gust).

Exceptions:

- <u>V_{asd} as determined in accordance with Section 1609.3.1 shall not exceed 110 mph (48.4 m/s)</u> (3-second gust) for buildings in Exposure Category B that are not located in a hurricaneprone region.
- Where V_{asd} as determined in accordance with Section 1609.3.1 exceeds 100 mph (3-second gust), the provisions of either AF&PA WFCM or ICC 600 are permitted to be used. Wind speeds in Figures 1609A, 1609B, and 1609C shall be converted in accordance with Section 1609.3.1 for use with AF&PA WFCM or ICC 600.

2308.2.5 Allowable roof span. Roof trusses and rafters shall not span more than 40 feet (12 192 mm) between points of vertical support.

2308.2.6 Risk Category limitation. The use of the provisions for *conventional light-frame construction* in this section shall not be permitted for *Risk Category* IV buildings, as determined by Section 1604.5, assigned to *Seismic Design Category* B, C, D or E.

2308.2.7 Portions exceeding limitations of conventional light-frame construction. When portions of a building of otherwise conventional light-frame construction exceed the limits of Section 2308.2, those portions and the supporting load path shall be designed in accordance with accepted engineering practice and the provisions of this code. For the purposes of this section, the term "portions" shall mean parts of buildings containing volume and area such as a room or a series of rooms. The extent of such design need only demonstrate compliance of the non-conventionally light-framed elements with other applicable provisions of this code and shall be compatible with the performance of the conventional light-framed system.

2308.3 Foundations and footings. Foundations and footings shall be designed and constructed in accordance with Chapter 18. Connections to foundations and footings shall comply with this section.

2308.3.1 Foundation plates or sills. Foundation plates or sills resting on concrete or masonry foundations shall comply with Section 2304.3.1. Foundation plates or sills shall be bolted or anchored to the foundation with not less than 1/2-inch-diameter (12.7 mm) steel bolts or approved anchors spaced to provide equivalent anchorage as the steel bolts. Along *braced wall lines* in structures assigned to *Seismic Design Category* E, steel bolts with a minimum nominal diameter of 5/8 inch (15.9 mm) or approved anchorage shall be used. Bolts shall be embedded at least 7 inches (178 mm) into concrete or masonry.

Bolts shall be spaced not more than 6 feet (1829 mm) apart and there shall be a minimum of two bolts or anchor straps per piece with one bolt or anchor strap located not more than 12 inches (305 mm) or less than 4 inches (102 mm) from each end of each piece. Bolts in *braced wall lines* in structures over two stories above grade shall be spaced not more the 4 feet (1219 mm) o.c.. A properly sized nut and washer shall be tightened on each bolt to the plate.

2308.3.2 Braced wall line sill plate anchorage in *Seismic Design Category* **D** and **E**. Sill plates along *braced wall lines* shall be anchored with anchor bolts with steel plate washers between the foundation sill plate and the nut, or *approved* anchor straps load rated in accordance with Section 1706.1. Such washers

shall be a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16 inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed 1-3/4 inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut.

2308.4 Floor framing. Floor framing shall comply with this section.

2308.4.1 Girders. Girders for single-story construction or girders supporting loads from a single floor shall not be less than 4 inches by 6 inches (102 mm by 152 mm) for spans 6 feet (1829 mm) or less, provided that girders are spaced not more than 8 feet (2438 mm) o.c. Spans for built-up 2-inch girders shall be in accordance with Table 2308.4.1(1) or 2308.4.1(2). Other girders shall be designed to support the loads specified in this code. Girder end joints shall occur over supports.

Where a girder is spliced over a support, an adequate tie shall be provided. The ends of beams or girders supported on masonry or concrete shall not have less than 3 inches (76 mm) of bearing.

TABLE 2308.9.5 TABLE 2308.4.1(1)HEADER AND GIRDER SPANS^a FOR EXTERIOR BEARING WALLS(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir^b and
Required Number of Jack Studs)

(Portions of table not shown remain unchanged)

TABLE 2308.9.6 TABLE 2308.4.1(2)HEADER AND GIRDER SPANS^a FOR INTERIOR BEARING WALLS(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir^b and
Required Number of Jack Studs)

(Portions of table not shown remain unchanged)

2308.4.2 Floor joists. Floor joists shall comply with this section.

2308.4.2.1 Span. Spans for floor joists shall be in accordance with Tables 2308.4.2.1(1) or 2308.4.2.1(2) or the *AF&PA Span Tables for Joists and Rafters*.

2308.4.2.2 Bearing. The ends of each joist shall not have less than 1-1/2 inches (38 mm) of bearing on wood or metal, or not less than 3 inches (76 mm) on masonry, except where supported on a 1-inch by 4-inch (25.4 mm by 102 mm) ribbon strip and nailed to the adjoining stud,

2308.4.2.3 Framing details. Joists shall be supported laterally at the ends and at each support by solid blocking except where the ends of the joists are nailed to a header, band or rim joist or to an adjoining stud or by other means. Solid blocking shall not be less than 2 inches (51mm) in thickness and the full depth of the joist. Joist framing from opposite sides of a beam, girder or partition shall be lapped at least 3 inches (76 mm) or the opposing joists shall be tied together in an approved manner. Joists framing into the side of a wood girder shall be supported by framing anchors or on ledger strips not less than 2 inches by 2 inches (51 mm).

TABLE 2308.8(1) 2308.4.2.1(1)FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES(Residential Sleeping Areas, Live Load = 30 psf, L/Δ = 360)

(Portions of table not shown remain unchanged)

TABLE 2308.8(2) 2308.4.2.1(2)FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES(Residential Living Areas, Live Load = 40 psf, L/Δ = 360)

(Portions of table not shown remain unchanged)

2308.4.2.4 Notches and holes. Notches on the ends of joists shall not exceed one-fourth the joist depth. Notches in the top or bottom of joists shall not exceed one sixth the depth and shall not be located in the middle third of the span. Holes bored in joists shall not be within 2 inches (51 mm) of the top or bottom of the joist and the diameter of any such hole shall not exceed one-third the depth of the joist.

2308.4.3 Engineered wood products. Engineered wood products shall be installed in accordance with manufacturer's recommendations. Cuts, notches and holes bored in trusses, structural composite lumber, structural glue-laminated members or I-joists are not permitted except where permitted by the manufacturer's recommendations or where the effects of such alterations are specifically considered in the design of the member by a *registered design professional*.

2308.4.4 Framing around openings. Trimmer and header joists shall be doubled, or of lumber of equivalent cross section, where the span of the header exceeds 4 feet (1219 mm). The ends of header joists more than 6 feet (1829 mm) long shall be supported by framing anchors or joist hangers unless bearing on a beam, partition or wall. Tail joists over 12 feet (3658 mm) long shall be supported at the header by framing anchors or on ledger strips not less than 2 inches by 2 inches (51 mm by 51 mm).

2308.4.4.1 Openings in floor diaphragms in *Seismic Design Categories* B, C, D and E. Openings in horizontal diaphragms with a dimension perpendicular to the joist that is greater than 4 feet (1219 mm) shall be constructed with metal ties and blocking in accordance with this section and Figure 2308.4.4.1(1). Metal ties shall not be less than 0.058 inch [1.47 mm (16 galvanized gage)] thick by 1-1/2 inches (38 mm) wide with a minimum yield stress of 33,000 psi (227 Mpa). Blocking shall be provided 2 feet minimum beyond headers. Ties shall be attached to blocking with eight 16d common nails on each side of the header-joist intersection.



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

FIGURE 2308.4.4.1(1) OPENINGS IN FLOOR AND ROOF DIAPHRAGMS

Openings in floor diaphragms in Seismic Design Categories D and E shall not exceed a dimension greater than 50 percent of the distance between braced wall lines or an area greater than 25 percent of the area between orthogonal pairs of braced wall lines [see Figure 2308.4.4.1(2)], or shall be designed in accordance with accepted engineering practice.



FIGURE 2308.4.4.1(2) OPENING LIMITATIONS FOR FLOOR AND ROOF DIAPHRAGMS

2308.4.4.2 Vertical offsets in floor diaphragms in Seismic Design Categories D and E. Portions of a floor level shall not be vertically offset such that the framing members on either side of the offset cannot be lapped or tied together in an *approved* manner in accordance with Figure 2308.4.4.2.

Exception: Framing supported directly by foundations need not be lapped or tied directly together.



SECTION VIEW

SECTION VIEW

FIGURE 2308.4.4.2 PORTIONS OF FLOOR LEVEL OFFSET VERTICALLY

2308.4.5 Joists supporting bearing partitions. Bearing partitions parallel to joists shall be supported on beams, girders, doubled joists, walls or other bearing partitions. Bearing partitions perpendicular to joists shall not be offset from supporting girders, walls or partitions more than the joist depth unless such joists are of sufficient size to carry the additional load.

2308.4.6 Lateral support. Floor and ceiling framing with a nominal depth-to-thickness ratio greater than or equal to 5:1 shall have one edge held in line for the entire span. Where the nominal depth-to- thickness ratio of the framing member exceeds 6:1, there shall be one line of bridging for each 8 feet (2438 mm) of span, unless both edges of the member are held in line. The bridging shall consist of not less than 1-inch by 3-inch (25 mm by 76 mm) lumber, double nailed at each end, of equivalent metal bracing of equal rigidity, full-depth solid blocking or other *approved* means. A line of bridging shall also be required at supports where equivalent lateral support is not otherwise provided.

2308.4.7 Structural floor sheathing. Structural floor sheathing shall comply with the provisions of Section 2304.7.1.

2308.4.8 Under-floor ventilation. For under-floor ventilation, see Section 1203.3.

2308.4.9 Floor framing supporting braced wall panels. When braced wall panels are supported by cantilevered floors or are setback from the floor joist support the floor framing shall comply section 2308.6.7.

2308.4.10 Anchorage of exterior means of egress components in Seismic Design Category D and *E*. Exterior egress balconies, exterior exit stairways and similar means of egress components in structures assigned to Seismic Design Category D or E shall be positively anchored to the primary structure at not over 8 feet (2438 mm) o.c. or shall be designed for lateral forces. Such attachment shall not be accomplished by use of toenails or nails subject to withdrawal.

2308.5 Wall construction. Walls of *conventional light-frame construction* shall be in accordance with this section.

2308.5.1 Stud size, height and spacing. The size, height and spacing of studs shall be in accordance with Table 2308.5.1

Studs shall be continuous from a support at the sole plate to a support at the top plate to resist loads perpendicular to the wall. The support shall be a foundation or floor, ceiling or roof diaphragm or shall be designed in accordance with accepted engineering practice.

Exception: Jack studs, trimmer studs and cripple studs at openings in walls that comply with Table 2308.4.1(1) or 2308.4.1(2).

2308.5.2 Framing details. Studs shall be placed with their wide dimension perpendicular to the wall. Not less than three studs shall be installed at each corner of an *exterior wall*.

Exceptions:

- 1. <u>In interior nonbearing walls and partition, studs are permitted to be set with the long dimension parallel to the wall.</u>
- At corners, two studs are permitted, provided wood spacers or backup cleats of 3/8-inchthick (9.5 mm) wood structural panel, 3/8-inch (9.5 mm) Type M "Exterior Glue" particleboard, 1-inch-thick (25 mm) lumber or other approved devices that will serve as an adequate backing for the attachment of facing materials are used. Where fire-resistance ratings or shear values are involved, wood spacers, backup cleats or other devices shall not be used unless specifically approved for such use.

TABLE 2308.5.1SIZE, HEIGHT AND SPACING OF WOOD STUDS

		BEARIN	IG WALLS		NONBEARING	WALLS
<u>STUD</u> <u>SIZE</u> (inche	<u>Laterally</u> <u>unsupported</u> <u>stud height^a <u>(feet)</u></u>	Supporting roof and ceiling only	Supporting one floor, roof and <u>ceiling</u>	<u>a</u> <u>Supporting</u> <u>two floors,</u> <u>roof and</u> <u>ceiling</u>	<u>Laterally</u> <u>unsupported</u> stud height ^a	<u>Spacing</u> (inches)
<u>s)</u>		<u>Spacin</u>	<u>Spacing (inches)</u>			
<u>2 × 3^b</u>	<u>NP</u>	<u>NP</u>	<u>NP</u>	<u>NP</u>	<u>10</u>	<u>16</u>
<u>2 × 4</u>	<u>10</u>	<u>24</u>	<u>16</u>	<u>NP</u>	<u>14</u>	<u>24</u>
<u>3 × 4</u>	<u>10</u>	<u>24</u>	<u>24</u>	<u>16</u>	<u>14</u>	<u>24</u>
<u>2 × 5</u>	<u>10</u>	24	24	<u>NP</u>	<u>16</u>	24
<u>2 × 6</u>	<u>10</u>	<u>24</u>	24	<u>16</u>	20	24

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

NP=Not Permitted

a. Listed heights are distances between points of lateral support placed perpendicular to the plane of the wall. Increases in unsupported height are permitted where justified by an analysis.

b. Shall not be used in exterior walls.

c. Utility-grade studs shall not be spaced more than 16 inches (406 mm) o.c., or support more than a roof and ceiling, or exceed 8 feet (2438 mm) in height for exterior walls and load-bearing walls or 10 feet (3048 mm) for interior non-load-bearing walls.

2308.5.3 Plates and sills. Studs shall have plates and sills according to this section.

2308.5.3.1. Bottom plate or sill. Studs shall have full bearing on a plate or sill. Plates or sills shall not be less than 2 inches (51 mm) nominal in thickness and have a width at least equal to the width of the wall studs.

2308.5.3.2 Top plates. Studs shall be capped with double top plates installed to provide overlapping at corners and at intersections with other partitions. End joints in double top plates shall be offset at least 48 inches (1219 mm), and shall be nailed in accordance with Table 2304.9.1. Plates shall be a nominal 2 inches (51 mm) in depth and have a width at least equal to the width of the studs.

Exception: A single top plate is permitted, provided the plate is adequately tied at joints, corners and intersecting walls by at least the equivalent of 3-inch by 6-inch (76 mm by 152 mm) by 0.036-inch-thick (0.914 mm) galvanized steel connector that is nailed to each wall or segment of wall by six 8d nails or equivalent, provided the rafters, joists or trusses are centered over the studs with a tolerance of not more than 1 inch (25 mm).

Where bearing studs are spaced at 24-inch (610 mm) intervals and top plates are less than two 2- inch by 6-inch (51 mm by 152 mm) or two 3-inch by 4- inch (76 mm by 102 mm) members and where the floor joists, floor trusses or roof trusses that they support are spaced at more than 16-inch (406 mm) intervals, such joists or trusses shall bear within 5 inches (127 mm) of the studs beneath or a third plate shall be installed.

2308.5.4 Nonbearing walls and partitions. In nonbearing walls and partitions, studs shall be spaced not more than 28 inches (711 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 1-1/2-inch (12.7 mm by 38 mm) metal ties with spliced sections fastened with two 16d nails on each side of the joint.

2308.5.5 Openings in walls and partitions. Openings in exterior and interior walls and partitions shall comply with sections 2308.5.5.1 through 2308.5.5.3

2308.5.5.1 Openings in exterior bearing walls. Headers shall be provided over each opening in exterior bearing walls. The size and spans in Table 2308.4.1(1) are permitted to be used for one- and two-family *dwellings*. Headers for other buildings shall be designed in accordance with Section 2301.2, Item 1 or 2. Headers shall be of two pieces of nominal 2-inch (51mm) framing lumber set on edge as permitted by Table 2308.4.1(1) and nailed together in accordance with Table 2304.9.1 or of solid lumber of equivalent size.

Wall studs shall support the ends of the header in accordance with Tables 2308.4.1(1). Each end of a lintel or header shall have a bearing length of not less than 1-1/2 inches (38 mm) for the full width of the lintel.

2308.5.5.2 Openings in interior bearing partitions. Headers shall be provided over each opening in interior bearing partitions as required in Section 2308.5.5.1 The spans in Table 2308.4.1(2) are permitted to be used. Wall studs shall support the ends of the header in accordance with Table 2308.4.1(1) or 2308.4.1(2), as appropriate.

2308.5.5.3 Openings in interior nonbearing partitions. Openings in nonbearing partitions are permitted to be framed with single studs and headers. Each end of a lintel or header shall have a bearing length of not less than 11/2 inches (38 mm) for the full width of the lintel.

2308.5.6 Cripple walls. Foundation cripple walls shall be framed of studs not less in size than the studding above with a minimum length of 14 inches (356 mm), or shall be framed of solid blocking. Where exceeding 4 feet (1219 mm) in height, such walls shall be framed of studs having the size required for an additional *story*. See section 2308.6.5 for cripple wall bracing.

2308.5.7 Bridging. Unless covered by interior or *exterior wall coverings* or sheathing meeting the minimum requirements of this code, stud partitions or walls with studs having a height-to-least-thickness ratio exceeding 50 shall have bridging not less than 2 inches (51 mm) in thickness and of the same width as the studs fitted snugly and nailed thereto to provide adequate lateral support. Bridging shall be placed in every stud cavity and at a frequency such that no stud so braced shall have a height-to-least-thickness ratio exceeding 50 with the height of the stud measured between horizontal framing and bridging or between bridging, whichever is greater.

2308.5.8 Pipes in walls. Stud partitions containing plumbing, heating or other pipes shall be so framed and the joists underneath so spaced as to give proper clearance for the piping. Where a partition containing such piping runs parallel to the floor joists, the joists underneath such partitions shall be doubled and spaced to *permit* the passage of such pipes and shall be bridged. Where plumbing, heating or other pipes are placed in or partly in a partition, necessitating the cutting of the soles or plates, a metal tie not less than 0.058 inch (1.47 mm) (16 galvanized gage) and 11/2 inches (38 mm) wide shall be fastened to each plate across and to each side of the opening with not less than six 16d nails.

2308.5.9 Cutting and notching. In exterior walls and bearing partitions, any wood stud is permitted to be cut or notched to a depth not exceeding 25 percent of its width. Cutting or notching of studs to a depth not greater than 40 percent of the width of the stud is permitted in nonbearing partitions supporting no loads other than the weight of the partition.

2308.5.10 Bored holes. A hole not greater in diameter than 40 percent of the stud width is permitted to be bored in any wood stud. Bored holes not greater than 60 percent of the width of the stud are permitted in nonbearing partitions or in any wall where each bored stud is doubled, provided not more than two such successive doubled studs are so bored. In no case shall the edge of the bored hole be nearer than 5/8 inch (15.9 mm) to the edge of the stud. Bored holes shall not be located at the same section of stud as a cut or notch.

2308.6 Wall Bracing. Buildings shall be provided with exterior and interior braced wall lines as described in Sections 2308.6.1 through 2308.6.9.2.

2308.6.1 *Braced wall lines.* For the purpose of determining the amount and location of bracing required along each story level of a building, *braced wall lines* shall be designated as straight lines through the building plan in both the longitudinal and transverse direction and placed in accordance with Table 2308.6.1 and Figure 2308.6.1. *Braced wall line* spacing shall not exceed the distance specified in Table 2308.6.1. In structures assigned to Seismic Design Category D or E, braced wall lines shall intersect perpendicularly to each other.

2308.6.2 *Braced wall panels. Braced wall panels* shall be placed along *braced wall lines* in accordance with Table 2308.6.1 and Figure 2308.6(1) and specified in Table 2308.6.2(1). A *braced wall panel* must be located at each end of the braced wall line and at the corners of intersecting *braced wall lines* or may begin within the maximum distance from the end of the *braced wall line* in accordance with Table 2308.6(1). *Braced wall panels* in a *braced wall line* shall not be offset from each other by more than 4 feet (1219 mm). Braced wall panels shall be clearly indicated on the plans.



Figure 2308.6(1) BASIC COMPONENTS OF THE LATERAL BRACING SYSTEM

TABLE 2308.1 WALL BRACING REQUIREMENTS

<u>Seismic</u> <u>Design</u> <u>Category</u>	<u>Story</u> <u>Condition</u> (See section	<u>Maximum</u> <u>spacing</u> of braced wall lines	Br	<u>Maximum</u> distance of braced wall panels from each		
	2308.2)		LIB	Bracing Metho	SFB PBS PCP HPS	end of braced wall line
		<u>35'-0"</u>	<u>Each end</u> <u>and</u> ≤25'-0" o.c.	<u>Each end and</u> <u>≤25'-0" o.c.</u>	<u>B,</u> Each end and ≤25'-0" o.c.	<u>12'-6"</u>
<u>A and B</u>		<u>35'-0"</u>	<u>Each end</u> <u>and</u> ≤25'-0" o.c.	<u>Each end and</u> ≤25'-0" o.c.	<u>Each end and</u> ≤25'-0" o.c.	<u>12'-6"</u>
		<u>35'-0"</u>	<u>NP</u>	<u>Each end and</u> <u>≤25'-0" o.c.</u>	<u>Each end and</u> ≤25'-0" o.c.	<u>12'-6"</u>
<u>C</u>		<u>35'-0"</u>	<u>NP</u>	<u>Each end and</u> <u>≤25'-0" o.c.</u>	<u>Each end and</u> <u>≤25'-0" o.c.</u>	<u>12'-6"</u>
		<u>35'-0"</u>	<u>NP</u>	<u>Each end and</u> <u>≤25'-0" o.c.</u> (min 25% of wall <u>length) [€]</u>	<u>Each end and</u> <u>≤25'-0" o.c.</u> (min 25% of wall <u>length) [€]</u>	<u>12'-6"</u>
<u>D and E</u>		<u>25'-0"</u>	<u>NP</u>	$\frac{Sds < 0.50:}{Each end and}$ $\leq 25'-0" o.c.$ $(min 21\% of wall)$ $length)^{e}$ $0.5 \leq Sds < 0.75:$ Each end and $\leq 25'-0" o.c.$ $(min 32\% of wall)$ $length)^{e}$ $0.75 \leq Sds \leq 1.00:$ Each end and $\leq 25'-0" o.c.$ $(min 37\% of wall)$ $length)^{e}$ $Sds > 1.00:$ Each end and $\leq 25'-0" o.c.$ $(min 37\% of wall)$ $length)^{e}$ $Sds > 1.00:$ Each end and $\leq 25'-0" o.c.$ $(min 48\% of wall)$ $length)^{e}$	$\frac{Sds < 0.50:}{Each end and}$ $\leq 25'-0" o.c.$ $(min 43\% of wall)$ $length)^{e}$ $0.5 \leq Sds < 0.75:$ Each end and $\leq 25'-0" o.c.$ $(min 59\% of wall)$ $length)^{e}$ $0.75 \leq Sds \leq 1.00:$ Each end and $\leq 25'-0" o.c.$ $(min 75\% of wall)$ $length)^{e}$ $Sds > 1.00:$ Each end and $\leq 25'-0" o.c.$ $(min 75\% of wall)$ $length)^{e}$ $Sds > 1.00:$ Each end and $\leq 25'-0" o.c.$ $(min 75\% of wall)$ $length)^{e}$	<u>8'-0"</u>

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

NP = Not Permitted

- a. This table specifies minimum requirements for braced wall panels along interior or exterior braced wall lines.
- b. See Section 2308.6.2 for full description of bracing methods.
- c. Gypsum wallboard applied to framing supports that are spaced at 16 inches on center.
- d. The required lengths shall be doubled for gypsum board applied to only one face of a braced wall panel.
- e. Percentage shown represents the minimum amount of bracing required along the building length (or wall length if the structure has an irregular shape)

2308.6.3 Braced wall panel methods. Construction of braced wall panels shall be by one or a

combination of the methods in Table 2308.6.3(1). *Braced wall panel* length shall be in accordance with Section 2308.6.4 or 2308.6.5.

<u>METHODS,</u> <u>MATERIAL</u>	MINIMUM THICKNESS	FIGURE	CONNECTION CRITERIA ª	
			Fasteners	<u>Spacing</u>
1 IP- ^a	<u>1x4 wood or</u> approved metal straps		Per Fastener Table 2304.9.1, item 20	<u>Wood: per stud</u> plus top and bottom plates
Let-in-bracing	attached at 45° to 60° angles to studs at maximum of <u>16" o.c.</u>		<u>Metal strap:</u> installed per manufacturer's installation recommendations	<u>Metal strap:</u> installed per manufacturer's installation recommendations
DWB Diagonal wood boards	<u>³/₄" thick</u> (<u>1" nominal) x 6"</u> <u>minimum width</u> <u>to studs at</u> <u>maximum of 24"</u> <u>0.C.</u>		Per Fastener Table 2304.9.1, item 21 or 22	<u>Per stud</u>
<u>WSP</u> Wood structural panel	³ / <u>8</u> " Per TABLE 2308.6.3(2) or 2308.6.3(3)		<u>Per Fastener Table</u> 2304.9.1, item 31	<u>6" edges</u> <u>12" field</u>
<u>SFB</u> Structural fiberboard sheathing	<u>لاء"</u> <u>Per TABLE</u> 2308.6.3(4)		<u>Per Fastener Table</u> 2304.9.1, item 33	<u>3" edges</u> <u>6" field</u>
<u>GB</u> <u>Gypsum board</u> (Double sided)	¹ / ₂ " by a minimum of 4 feet wide to studs at maximum of 24" <u>o.c.</u>		Exterior and interior sheathing: with 5d cooler nails (1-5/8" x 0.086") or 11⁄4" screws (type W or S) for 1⁄2" gypsum board or 1 ⁵ / ₈ " screws (type	For all braced wall panel locations: 7" o.c. along panel edges (including top and bottom plates) and

TABLE 2308.6.3(1) BRACING METHODS

<u>METHODS,</u> MATERIAL	MINIMUM THICKNESS	FIGURE	CONNECTION CRITERIA ^ª	
			<u>W or S) for ⁵/₈"</u> gypsum board.	<u>7" o.c.in the field</u>
PBS Particle-board sheathing	$\frac{{}^{3}}{{}^{8}$ " or ${}^{1}\!{}^{2}$ " per Table 2308.9.3(4) to studs at maximum of <u>16</u> " o.c.		$\frac{6d \text{ common } (2" \text{ long})}{x0.113" \text{ dia.}) \text{ nails}}$ $\frac{for \frac{3}{/3"} \text{ thick}}{sheathing \text{ or}}$ $\frac{8d \text{ common } (2'/2")}{long x 0.131" \text{ dia.})}$ $\frac{nails \text{ for } \frac{1}{/2"} \text{ thick}}{sheathing}$	<u>3" edges</u> <u>6" field</u>
PCP Portland cement plaster	<u>See Section</u> <u>2510</u> <u>to studs</u> <u>at maximum of</u> <u>16" o.c.</u>		<u>1½" long, 11 gage,</u> <u>4₁₆" dia. head nails</u> or <u>4₈" long, 16 gage</u> <u>staples</u>	<u>6" o.c. on all</u> framing members
<u>HPS</u> <u>Hardboard panel</u> <u>siding</u>	/_ ₁₆ " <u>TABLE</u> 2308.6.3(5)		<u>Per Fastener Table</u> 2308.9.1	<u>4" edges</u> <u>8" field</u>
ABW Alternate braced wall.	<u>³/<u>8</u>"</u>		<u>See Figure</u> 2308.6.5(1) and Section 2308.6.5.1	<u>See Figure</u> 2308.6.3(1)
PFH Portal frame with hold-downs	<u>³/8</u>		<u>See Figure</u> 2308.6.5(2) and Section 2308.6.5.2	<u>See Figure</u> 2308.6.3(2)

For SI: 1 foot 305 mm

a. Method LIB shall have gypsum board fastened to at least one side with nails or screws.

TABLE 2308.6.3(2) EXPOSED PLYWOOD PANEL SIDING

MINIMUM THICKNESS ^a (inch)	MINIMUM NUMBER OF PLIES	STUD SPACING (inches) Plywood siding applied directly to studs or over sheathing	
3/8	3	16 ^b	
1/2	- 4	24	

For SI: 1 inch = 25.4 mm.

a. Thickness of grooved panels is measured at bottom of grooves.
 b. Spans are permitted to be 24 inches if plywood siding applied with face grain perpendicular to studs or over one of the following: (1) 1-inch board sheathing. (2) ⁷/₁₆-inch wood structural panel sheathing with strength axis (which is the long direction of the panel unless otherwise marked) of sheathing perpendicular to studs.

TABLE 2308.6.3(3)

MINIMUM THICKNESS RATING		STUD SPACING (inches)			
	PANEL SPAN		Nailable sheathing		
	Siding nailed to studs	Sheathing parallel to studs	Sheathing perpendicular to studs		
³ / ₈ , ¹⁵ / ₃₂ , ¹ / ₂	16/0, 20/0, 24/0, 32/16 Wall—24″ o.c.	24	16	24	
⁷ / ₁₆ , ¹⁵ / ₃₂ , ¹ / ₂	24/0, 24/16, 32/16 Wall—24" o.c.	24	24ª	24	

WOOD STRUCTURAL PANEL WALL SHEATHING^b Not Exposed to the Weather, Strength Axis Parallel or Perpendicular to Studs Except as Indicated Below)

For SI: 1 inch = 25.4 mm.

a. Plywood shall consist of four or more pltes.

b. Blocking of horizontal joints shall not be required except as specified in Sections 2306.3 and 2308.12.4.

TABLE 2308.6.3(4) ALLOWABLE SPANS FOR PARTICLEBOARD WALL SHEATHING (Not Exposed to the Weather, Long Dimension of the Panel Parallel or Perpendicular to Studs)

	THERMESE	STUD SPACING (inches)		
GRADE	(inch)	Siding nailed to studs	Sheathing under coverings specified in Section 2308.9.3 parallel or perpendicular to studs	
M-S "Exterior Glue"	3/8	16		
and M-2 "Exterior Glue"	1/2	16	16	

For SI: 1 inch = 25.4 mm.

TABLE 2308.6.3(5) HARDBOARD SIDING

	MINIMUM NOMINAL	2 × 4 FRAMING		NAIL SPACING		
SIDING	ING THICKNESS MAXIMUM SPACING NAIL SIZE		NAIL SIZE ^{4, 0, 0}	General	Bracing panels ^e	
1. Lap siding						
Direct to studs	3/ ₈	16" o.c.	8d	16″ o.c.	Not applicable	
Over sheathing	3/8	16″ o.c.	10d	16″ o.c.	Not applicable	
2. Square edge p	anel siding					
Direct to studs	3/ ₈	24″ o.c.	6d	6" o.c. edges; 12" o.c. at intermediate supports	4" o.c. edges; 8" o.c. at intermediate supports	
Over sheathing	³ / ₈	24″ o.c.	8d	6" o.c. edges; 12" o.c. at intermediate supports	4" o.c. edges; 8" o.c. at intermediate supports	
3. Shiplap edge p	anel siding					
Direct to studs	³ / ₈	16″ o.c.	6d	6" o.c. edges; 12" o.c. at intermediate supports	4" o.c. edges; 8" o.c. at intermediate supports	
Over sheathing	3/ ₈	16″ o.c.	8d	6" o.c. edges; 12" o.c. at intermediate supports	4" o.c. edges; 8" o.c. at intermediate supports	

For SI: 1 Inch = 25.4 mm.

a. Nails shall be corrosion resistant.b. Minimum acceptable nail dimensions:

Panel Siding (inch)Lap Siding (inch)Shank diameter0.0920.099Head diameter0.2250.240

c. Where used to comply with Section 2308.9.3.

d. Nail length must accommodate the sheathing and penetrate framing 11/2 inches.

2308.6.4 Length of braced wall panels. For Methods DWB, WSP, SFB, PBS, PCP and HPS each panel must be at least 48 inches (1219 mm) in length, covering three stud spaces where studs are

spaced 16 inches (406 mm) apart and covering two stud spaces where studs are spaced 24 inches (610 mm) apart. *Braced wall panels* less than the required 48" length shall not contribute towards the amount of bracing required. *Braced wall panels* longer than the required length shall be credited for their actual length. For Method GB, each panel must be at least 96 inches (2438 mm) in length where applied to one side of the studs or 48 inches (1219 mm) where applied to both sides.

All vertical joints of panel sheathing shall occur over studs and adjacent panel joints shall be nailed to common framing members. Horizontal joints shall occur over blocking or other framing equal in size to the studding except where waived by the installation requirements for the specific sheathing materials. Sole plates shall be nailed to the floor framing in accordance with Section 2308.3.2 and top plates shall be connected to the framing above in accordance with Section 2308.5.3. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the *braced wall panels*.

2308.6.5 Alternative bracing. An Alternate Braced Wall (ABW) or a Portal Frame with Hold-downs (PFH) described in this section is permitted to substitute for a 48" braced wall panel of methods DWB, WSP, SFB, PBS, PCP or HPS. For method GB, 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 2308.6.1 is permitted to be replaced by one panel constructed in accordance with method ABW or PFH.

2308.6.5.1. Alternate Braced Wall (ABW). An ABW shall be constructed in accordance with this section and Figure 2308.6.5.1. In one-story buildings, each panel shall have a length of not less than 2 feet 8 inches (813 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with 3/8- inch-minimum-thickness (9.5 mm) wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Table 2304.9.1 and blocked at wood structural panel edges. Two anchor bolts installed in accordance with Section 2308.3.1 shall be provided in each panel. Anchor bolts shall be placed at each panel outside quarter points. Each panel end stud shall have a hold-down device fastened to the foundation, capable of providing an approved uplift capacity of not less than 1,800 pounds (8006 N). The hold-down device shall be installed in accordance with the manufacturer's recommendations. The ABW shall be supported directly on a foundation or on floor framing supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom. Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

When the ABW is installed at the first *story* of two-story buildings, the wood structural panel sheathing shall be provided on both faces, three anchor bolts shall be placed at one-quarter points, and tie-down device uplift capacity shall not be less than 3,000 pounds (13 344 N).

2308.6.5.2 Portal Frame with Hold-downs (PFH). A PFH shall be constructed in accordance with this section and Figure 2308.6.5.2. The adjacent door or window opening shall have a full-length header.

In one-story buildings, each panel shall have a length of not less than 16 inches (406 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with a single layer of 3/8 inch (9.5 mm) minimum thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Figure 2308.6.5.2. The wood structural panel sheathing shall extend up over the solid sawn or glued-laminated header and shall be nailed in accordance with Figure 2308.6.5. A built-up header consisting of at least two 2 × 12s and fastened in accordance with Item 24 of Table 2304.9.1 shall be permitted to be used. A spacer, if used, shall be placed on the side of the built-up beam opposite the wood structural panel sheathing. The header shall extend between the inside faces of the first full-length outer studs of each panel. The clear span of the header between the inner studs of each panel shall be not less than 6 feet (1829 mm) and not more than 18 feet (5486 mm) in length. A strap with an uplift capacity of not less than 1,000 pounds (4,400 N) shall fasten the header to the inner studs opposite the sheathing. One anchor bolt not less than 5/8 inch (15.9 mm) diameter and installed in accordance

with Section 2308.3.1 shall be provided in the center of each sill plate. The studs at each end of the panel shall have a hold-down device fastened to the foundation with an uplift capacity of not less than 4,200 pounds (18 480 N).

Where a panel is located on one side of the opening, the header shall extend between the inside face of the first full-length stud of the panel and the bearing studs at the other end of the opening. A strap with an uplift capacity of not less than 1,000 pounds (4400 N) shall fasten the header to the bearing studs. The bearing studs shall also have a hold-down device fastened to the foundation with an uplift capacity of not less than 1,000 pounds (4400 N). The hold-down devices shall be an embedded strap type, installed in accordance with the manufacturer's recommendations. The PFH panels shall be supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped not less than 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

When a PFH is installed at the first *story* of two-story buildings, each panel shall have a length of not less than 24 inches (610 mm).



FIGURE 2308.6.5.1 ALTERNATE BRACED WALL PANEL (ABW)



Figure 2308.6.5.2 PORTAL FRAME WITH HOLD-DOWNS (PFH)

2308.6.5 Cripple wall bracing. Cripple walls shall be braced in accordance with the following.

2308.6.5.1 Cripple wall bracing in *Seismic Design Category* **A**, **B** and **C**.. For the purposes of this section, cripple walls having a stud height exceeding 14 inches (356 mm) shall be considered a *story* and shall be braced in accordance with Table 2308.6(1). Spacing of edge nailing for required cripple wall bracing shall not exceed 6 inches (152mm) o.c. along the foundation plate and the top plate of the cripple wall. Nail size, nail spacing for field nailing and more restrictive boundary nailing requirements shall be as required elsewhere in the code for the specific bracing material used.

2308.6.5.2 Cripple wall bracing in *Seismic Design Category* **D** and **E** For the purposes of this section, cripple walls having a stud height exceeding 14 inches (356 mm) shall be considered a *story* and shall be braced in accordance with Table 2308.6(1). 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 2308.6(1). Where the cripple wall sheathing type used is method WSP or DWB and this additional length of bracing cannot be provided, the capacity of WSP or DWB 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.

2308.6.6 Connections of braced wall panels. Braced wall panel joints shall occur over studs or blocking. *Braced wall panels* shall be fastened to studs, top and bottom plates and at panel edges. *Braced wall panels* shall be applied to nominal 2-inch-wide [actual 1-1/2 inch (38 mm)] or larger stud framing.

2308.6.6.1 Bottom plate connection. Braced wall line bottom plates shall be connected to joists or fulldepth blocking below in accordance with Table 2304.9.1, Item 6, or to foundations in accordance with Section 2308.3.3.

2308.6.6.2 Top plate connection. Where joists and/or rafters are used, braced wall line top plates shall be fastened over the full length of the braced wall line to joists, rafters, rim boards or blocking above in accordance with Table 2304.9.1, as applicable, based on the orientation of

the joists or rafters to the braced wall line. Blocking at joists with walls above shall be equal to the depth of the joist at the braced wall line. Blocking at rafters need not be full depth but shall extend to within 2 inches (51 mm) from the roof sheathing above. Blocking shall be a minimum of 2 inches (51 mm) nominal thickness and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.8.2 or Section 2308.10.4.2 shall be permitted.

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 over the full length of the braced wall line by blocking of the ends of the trusses or by other *approved* methods providing equivalent lateral force transfer. Blocking shall be minimum 2 inches (51 mm) nominal thickness and shall extend to within 2 inches (51 mm) from the roof sheathing above and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.4.2.4 or Section 2308.7.4 shall be permitted.

2308.6.6.3 Sill anchorage. Where foundations are required by Section 2308.6.7, braced wall line sills shall be anchored to concrete or masonry foundations. Such anchorage shall conform to the requirements of Section 2308.3. The anchors shall be distributed along the length of the braced wall line. Other anchorage devices having equivalent capacity are permitted.

2308.6.6.4 Anchorage to all-wood foundations. Where all-wood foundations are used, the force transfer from the braced wall lines shall be determined based on calculation and shall have a capacity greater than or equal to the connections required by Section 2308.3.

2308.6.7 Braced wall line and diaphragm support. Braced wall lines and floor and roof diaphragms shall be supported in accordance to this section.

2308.6.7.1 Foundation requirements. Braced wall lines shall be supported by continuous foundations.

Exception: For structures with a maximum plan dimension not over 50 feet (15 240 mm), continuous foundations are required at exterior walls only.

For structures in Seismic Design Category D and E, exterior braced wall panels shall be in the same plane vertically with the foundation or the braced wall line shall be designed in accordance with accepted engineering practice according to section 2308.1.1

Exceptions:

- 1. Exterior *braced wall panels* may be located up to 4 feet from the foundation below when supported by a floor constructed in accordance with all the following:
 - 1.1 Cantilevers or setbacks shall not exceed four times the nominal depth of the floor joists
 - 1.2. Floor joists shall be 2 inches by 10 inches (51 mm by 254 mm) or larger and spaced not more than 16 inches (406 mm) o.c.
 - 1.3. The ratio of the back span to the cantilever shall be at least 2:1.
 - 1.4. Floor joists at ends of *braced wall panels* shall be doubled.
 - 1.5.
 A continuous rim joist shall be connected to the ends of cantilevered joists. The rim joist is permitted to be spliced using a metal tie not 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 stress of 33,000 psi (227 MPa).

- 1.6. 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. The end of a required braced wall panel shall be allowed to extend not more than 1 foot (305 mm) over an opening in the wall below. 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 ltem 1 above in this section.

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.

2308.6.7.2 Floor and roof diaphragm support in *Seismic Design Category D and E.* In structures assigned to *Seismic Design Category D or E,* floor and roof diaphragms shall be laterally supported by braced wall lines on all edges and connected in accordance with Section 2308.3.2 [see Figure 2308.6.7.2(1)].

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 2308.6.7.2(2)] provided that the framing members are connected to the braced wall line below in accordance with Section 2308.6.6.

2308.6.7.3 Stepped footings in Seismic Design Category B, C, D and E. Where the height of a required braced wall panel extending from foundation to floor above varies more than 4 feet (1219 mm), the following construction shall be used:

- 1. Where the bottom of the footing is stepped and the lowest floor framing rests directly on a sill bolted to the footings, the sill shall be anchored as required in Section 2308.3.3.
- 2. Where the lowest floor framing rests directly on a sill bolted to a footing not less than 8 feet (2438 mm) in length along a line of bracing, the line shall be considered to be braced. The double plate of the cripple stud wall beyond the segment of footing extending to the lowest framed floor shall be spliced to the sill plate with metal ties, one on each side of the sill and plate. The metal ties shall not be less than 0.058 inch [1.47 mm (16 galvanized gage)] by 11/2 inches (38 mm) wide by 48 inches (1219 mm) with eight 16d common nails on each side of the splice location (see Figure 2308.6.7.3(1). The metal tie shall have a minimum yield stress of 33,000 pounds per square inch (psi) (227 MPa).
- 3. Where cripple walls occur between the top of the footing and the lowest floor framing, the bracing requirements for a *story* shall apply.



ROOF EXTENSION IN SDC D OR E BEYOND BRACED WALL LINE



For SI: 1 inch = 25.4 run, 1 foot = 304.8 run.

FIGURE 2308.6.7.3(1) STEPPED FOOTING CONNECTION DETAILS

2308.6.8 Attachment of sheathing. Fastening of braced wall panel sheathing shall not be less than that prescribed in Tables 2308.6(1) and 2304.9.1. Wall sheathing shall not be attached to framing members by adhesives.

2308.6.9 Limitations of concrete or masonry veneer. Concrete or masonry veneer shall comply with Chapter 14 and this section.

2308.6.9.1 Limitations of concrete or masonry veneer in Seismic Design Categories B or C... Concrete or masonry walls and stone or masonry veneer shall not extend above a basement.

Exceptions:

- In structures assigned to Seismic Design Category B, 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, provided that structural use panel wall bracing is used and the length of bracing provided is one and one-half times the required length as determined in Table 2308.9.3(1).
- 2. Stone and masonry veneer is permitted to be used in the first story above grade plane or the first two stories above grade plane where the lowest story has concrete or masonry walls.
- 3. Stone and masonry veneer is permitted to be used in both stories of buildings with two stories above grade plane, provided the following criteria are met:
 - 3.1. Type of brace per Section 2308.9.3 shall be WSP and the allowable shear capacity in accordance with 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 capacity of 2,000 pounds (8896 N). 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 (17 347 N). In all cases, the hold-down connector force shall be transferred to the foundation.

3.4. Cripple walls shall not be permitted.

2308.6.9.2 Limitations of concrete or masonry in Seismic Design Categories D and E Concrete or masonry walls and stone or masonry veneer shall not extend above a basement.

Exception: In structures assigned to *Seismic Design Category* D, stone and masonry veneer is permitted to be used in the first story above grade plane, provided the following criteria are met:

- 1. Type of brace in accordance with Section 2308.9.3 shall be WSP and the allowable shear capacity in accordance with 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 (9341 N).
- 4. Cripple walls shall not be permitted.

2308.7 Roof and ceiling framing. The framing details required in this section apply to roofs having a minimum slope of three units vertical in 12 units horizontal (25-percent slope) or greater. Where the roof slope is less than three units vertical in 12 units horizontal (25-percent slope), members supporting rafters and ceiling joists such as ridge board, hips and valleys shall be designed as beams.

2308.7.1 Ceiling joist spans. Allowable spans for ceiling joists shall be in accordance with Table 2308.7.1(1) or 2308.7.1(2). For other grades and species, refer to the *AF&PA Span Tables for Joists and Rafters*.

TABLE 2308.10.2(1) TABLE 2308.7.1(1)CEILING JOIST SPANS FOR COMMON LUMBER SPECIES(Uninhabitable Attics Without Storage, Live Load = 10 pounds psf, L/Δ = 240)

(Portions of Table not shown remain unchanged)

TABLE 2308.10.2(2) TABLE 2308.7.1(2)CEILING JOIST SPANS FOR COMMON LUMBER SPECIES(Uninhabitable Attics With Limited Storage, Live Load = 20 pounds per square foot, L/Δ = 240)

(Portions of Table not shown remain unchanged)

2308.7.2 Rafter spans. Allowable spans for rafters shall be in accordance with Table 2308.7.2(1), 2308.7.2(2), 2308.7.2(3), 2308.7.2(4), 2308.7.2(5) or 2308.7.2(6). For other grades and species, refer to the AF&PA Span Tables for Joists and Rafters.

TABLE 2308.10.3(1) TABLE 2308.7.2(1)RAFTER SPANS FOR COMMON LUMBER SPECIES(Roof Live Load = 20 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)

(Portions of Table not shown remain unchanged)

TABLE 2308.10.3(2) TABLE 2308.7.2(2)RAFTER SPANS FOR COMMON LUMBER SPECIES(Roof Live Load = 20 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)

(Portions of Table not shown remain unchanged)

TABLE 2308.10.3(3) TABLE 2308.7.2(3)RAFTER SPANS FOR COMMON LUMBER SPECIES(Ground Snow Load = 30 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)

(Portions of Table not shown remain unchanged)

TABLE 2308.10.3(4) TABLE 2308.7.2(4)RAFTER SPANS FOR COMMON LUMBER SPECIES(Ground Snow Load = 50 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)

(Portions of Table not shown remain unchanged)

TABLE 2308.10.3(5)TABLE 2308.7.2(5)RAFTER SPANS FOR COMMON LUMBER SPECIES(Ground Snow Load = 30 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)

(Portions of Table not shown remain unchanged)

TABLE 2308.10.3(6)TABLE 2308.7.2(6)RAFTER SPANS FOR COMMON LUMBER SPECIES(Ground Snow Load = 50 pounds per square foot, Ceiling Attached to Rafters, L/△ = 240)

(Portions of Table not shown remain unchanged)

2308.7.3 Ceiling joist and rafter framing. Rafters shall be framed directly opposite each other at the ridge. There shall be a ridge board at least 1-inch (25 mm) nominal thickness at ridges and not less in depth than the cut end of the rafter. At valleys and hips, there shall be a single valley or hip rafter not less than 2-inch (51 mm) nominal thickness and not less in depth than the cut end of the rafter.

2308.7.3.1 Ceiling joist and rafter connections. Ceiling joists and rafters shall be nailed to each other and the assembly shall be nailed to the top wall plate in accordance with Tables 2304.9.1 and 2308.7.5. Ceiling joists shall be continuous or securely joined where they meet over interior partitions and be fastened to adjacent rafters in accordance with Tables 2304.9.1 and 2308.7.3.1 to provide a continuous rafter tie across the building where such joists are parallel to the rafters. Ceiling joists shall have a bearing surface of not less than 1-1/2 inches (38 mm) on the top plate at each end.

Where ceiling joists are not parallel to rafters, an equivalent rafter tie shall be installed in a manner to provide a continuous tie across the building, at a spacing of not more than 4 feet (1219 mm) o.c. The connections shall be in accordance with Tables 2308.7.3.1 and 2304.9.1, or connections of equivalent capacities shall be provided. Where ceiling joists or rafter ties are not provided at the top of the rafter support walls, the ridge formed by these rafters shall also be supported by a girder conforming to Section 2308.2.7. Rafter ties shall be spaced not more than 4 feet (1219 mm) o.c.

Rafter tie connections shall be based on the equivalent rafter spacing in Table 2308.7.3.1. Rafter/ceiling joist connections and rafter/tie connections shall be of sufficient size and number to prevent splitting from nailing.

Roof framing member connection to braced wall lines shall be in accordance with 2308.6.6.2.



Note: Where ceiling joints run perpendicular to the rafter, rafter ties shall be installed per section 2308.7.3.1

FIGURE 2308.7 ROOF CEILING FRAMING

TABLE 2308.10.4.1 TABLE 2308.7.3.1 RAFTER TIE CONNECTIONS⁹

(Portions of Table not shown remain unchanged)

2308.7.4 Notches and holes. Notching at the ends of rafters or ceiling joists shall not exceed one-fourth the depth. Notches in the top or bottom of the rafter or ceiling joist shall not exceed one-sixth the depth and shall not be located in the middle one-third of the span, except that a notch not exceeding one-third of the depth is permitted in the top of the rafter or ceiling joist not further from the face of the support than the depth of the member. Holes bored in rafters or ceiling joists shall not be within 2 inches (51 mm) of the top and bottom and their diameter shall not exceed one-third the depth of the member.

2308.7.5 Wind uplift. The roof construction shall have rafter and truss ties to the wall below. Resultant uplift loads shall be transferred to the foundation using a continuous load path. The rafter or truss to wall connection shall comply with Tables 2304.9.1 and 2308.7.5

TABLE 2308.10.1 TABLE 2308.7.5 REQUIRED RATING OF APPROVED UPLIFT CONNECTORS (pounds)^{a, b, c, e, f, g, h}

(Portions of Table not shown remain unchanged)

2308.7.6 Framing around openings. Trimmer and header rafters shall be doubled, or of lumber of equivalent cross section, where the span of the header exceeds 4 feet (1219 mm). The ends of header rafters more than 6 feet (1829 mm) long shall be supported by framing anchors or rafter hangers unless bearing on a beam, partition or wall.

2308.7.6.1 Openings in roof diaphragms in Seismic Design Categories B, C, D and E. Openings in horizontal diaphragms with a dimension perpendicular to the joist that is greater than 4 feet (1219 mm)

shall be constructed with metal ties and blocking in accordance with this section and Figure 2308.4.4.1(1). Metal ties shall not be less than 0.058 inch [1.47 mm (16 galvanized gage)] thick by 1-1/2 inches (38 mm) wide with a minimum yield stress of 33,000 psi (227 Mpa). Blocking shall be provided 2 feet minimum beyond headers. Ties shall be attached to blocking with eight 16d common nails on each side of the header-joist intersection.

2308.7.7 Purlins. Purlins to support roof loads are permitted to be installed to reduce the span of rafters within allowable limits and shall be supported by struts to bearing walls. The maximum span of 2-inch by 4-inch (51 mm by 102 mm) purlins shall be 4 feet (1219 mm). The maximum span of the 2-inch by 6-inch (51 mm by 152 mm) purlin shall be 6 feet (1829 mm), but in no case shall the purlin be smaller than the supported rafter. Struts shall not be smaller than 2-inch by 4-inch (51 mm by 102 mm) members. The unbraced length of struts shall not exceed 8 feet (2438 mm) and the minimum slope of the struts shall not be less than 45 degrees (0.79 rad) from the horizontal.

2308.7.8 Blocking. Roof rafters and ceiling joists shall be supported laterally to prevent rotation and lateral displacement in accordance with the provisions of Section 2308.8.5 and connected to braced wall lines per Section 2308.6.6.2.

2308.7.9 Engineered wood products. Prefabricated wood I-joists, structural glued-laminated timber and structural composite lumber shall not be notched or drilled except where permitted by the manufacturer's recommendations or where the effects of such alterations are specifically considered in the design of the member by a *registered design professional*.

2308.7.10 Roof sheathing. Roof sheathing shall be in accordance with Tables 2304.7(3) and 2304.7(5) for wood structural panels, and Tables 2304.7(1) and 2304.7(2) for lumber and shall comply with Section 2304.7.2.

2308.7.11 Joints. Joints in lumber sheathing shall occur over supports unless *approved* end-matched lumber is used, in which case each piece shall bear on at least two supports.

2308.7.12 Roof planking. Planking shall be designed in accordance with the general provisions of this code.

In lieu of such design, 2-inch (51 mm) tongue-and groove planking is permitted in accordance with Table 2308.10.9. Joints in such planking are permitted to be randomly spaced, provided the system is applied to not less than three continuous spans, planks are center matched and end matched or splined, each plank bears on at least one support, and joints are separated by at least 24 inches (610 mm) in adjacent pieces.

2308.7.13 Wood trusses. Wood trusses shall be designed in accordance with Section 2303.4. Connection to braced wall lines shall be in accordance with Section 2308.6.6.2.

2308.7.14 Attic ventilation. For attic ventilation, see Section 1203.2.

Reason: This proposal is intended to completely replace the existing section 2308 "Conventional Light-Frame Construction" with a re-formatted version. This proposal is not intended to introduce any new requirements into, nor remove any requirements from, the existing section 2308.

As a result of many code cycles, Section 2308 has become fragmented and is not organized in a logical manner and is difficult to use. With this proposal, Section 2308 is formatted to begin with general requirements then proceed to foundations, floor framing, wall framing, wall bracing and roof-ceiling construction in that order. The additional requirements for *Seismic Design Categories* in the 2012 IBC Sections 2308.11 and 2308.12 (SDC B/C and SDC D/E respectively) have been merged into the appropriate new sections based on the type of construction such as floor framing, wall bracing and roof framing.

Terminology has been coordinated throughout the section such as the terms, "conventional light-frame construction", "braced wall line" and "braced wall panel".

This proposal is intended to be non-technical and separate proposals have been submitted to address technical items in section 2308.

In order to make the prescriptive provisions of the IBC more closely resemble the format of the similar provisions in the IRC, much of the wall bracing terminology is replicated from the IRC, namely:

- The requirements for braced wall line spacing were put into a single table format based on Seismic Design Category rather than scattered throughout all of Section 2308.
- The wall bracing methods were compiled into a table similar to the IRC, including abbreviations for the methods, rather

than referring to them by a number. The fasteners specified in this table were cross-referenced to the fastener table 2308.9.3.1 where applicable.

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- For the section, "Alternate bracing" a figure (copied from the IRC) was introduced, but no technical changes were made. Similarly, for Section 2308.9.3.2, "Alternate bracing wall panel adjacent to a door or window opening" was renamed since it aligned perfectly with the Portal Frame with Hold-downs method (PFH) in the IRC. The figure was already in the IBC, so the title was changed to reflect the new name. ٠

Propose	d 2015	2012 IBC
2308 Conventional Light-Frame	Construction	2308 Conventional Light-Frame Construction
2308.1 General. The requirement intended for <i>conventional light-fr</i> <u>construction</u> methods are permit satisfactory design is submitted other provisions of this code. Inter- partitions, ceilings and curtain w <i>frame construction</i> are not subjet section 2308.2.	nts of this section are ame construction. Other ted to be used, provided a showing compliance with erior non-load-bearing alls of conventional light- ct to the limitations of this	2308.1 General. As shown modified to the left
		2308.1.1 Portions exceeding limitations of conventional
2308.2 Limitations		2308.2 Limitations. Included reference to items in 2308.11 (SDC B and C) and 2308.12 (SDC D and E). Those items have been moved here and elsewhere in the section as noted.
2308.2.1 Stories. The height limitations in the table are from: 2308.2.1 Stories. Structures of <i>conventional light-frame construction</i> shall be limited in story height according to the following:		 2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of <i>conventional light-frame construction</i>, 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 <i>stories above grade plane</i>. For the purposes of this section, for
Seismic Design Category	Allowable Story above grade plane	buildings assigned to Seismic Design Category D or E, cripple stud walls shall be considered to be a story.
A and B	Three stories	Exception: Solid blocked cripple walls not exceeding 14 inches (356 mm) in height need not be considered
С	Two Stories	a story.
D and E ^a	One story	2308.11.1 Number of stories. Structures of <i>conventional light-</i> <i>frame construction</i> and assigned to Seismic Design Category
a. For the purposes of assigned to Seismic D unless cripple walls ar exceed 14 inches in h considered to be a sto	this section, for buildings besign Category D or E, e solid blocked and do not eight, cripple walls shall be ry.	C shall not exceed two stories above grade plane. 2308.12.1 Number of stories. Structures of <i>conventional light-frame construction</i> and assigned to <i>Seismic Design Category</i> D or E shall not exceed one story above grade plane.
2308.2.2 Allowable floor-to-floor	height	Moved from 2308.2, item 2
2308.2.3 Allowable Loads		Moved from 2308.2, item 3
2308.2.4 Allowable wind speed		Moved from 2308.2, item 4
2308,2,5 Allowable roof span		Moved from 2308.2, item 5
2308.2.6 Risk Category limitation	n	Moved from 2308.2, item 6. SDC "F" was deleted since the provisions of 2308 are not allowed in SDC F.
2308.2.8 Portions exceeding lim frame construction	itations of conventional light-	Moved from 2308.1.1 and unchanged. The last sentence was moved here from the last sentence of 2308.4.2. The rest of 2308.4.2 was redundant.
2308.3 Foundations and footings	s. Foundations and footings	Moved from 2308.6
2308.3.1 Foundation plates or si	lls	Moved from 2308.12.9
2308.3.2 Sill plate anchorage in and E.	Seismic Design Category D	2308.12.8 Sill plate anchorage
2308.4 Floor framing		
2308.4.1 Girders		Moved from 2308.7
2308.4.2 Floor joists		
2308.4.2.1 Span		Moved from 2308.8

nosed 2015 to the existing 2012 Co arison of the

2308.4.2.2 Bearing	Moved from 2308.8.1. Switched first sentence to end of
2308.4.2.3 Framing details	Moved from 2308.8.2. Notches portion removed and placed in section 2308.4.2.4
2308.4.2.4 Notches and holes	Moved from 2308.8.2
2308.4.3 Engineered wood products	Moved from 2308.8.2.1. First sentence is new.
2308.4.4 Framing around openings	Moved from 2308.8.3
2308.4.4.1 Openings in horizontal diaphragms in SDC B, C, D and E	From 2308.11.3.3 The text of this section has been re-arranged for clarity. The first sentence states that a tie and blocking are required. Then, the tie is described followed by the blocking.
2308.4.5 Joists supporting bearing partitions	Moved from 2308.8.4
2308.4.6 Lateral support	Moved from 2308.8.5. Changed "Floor, attic and roof" to "Floor and ceiling"
2308.4.7 Structural floor sheathing	Moved from 2308.8.6
2308.4.8 Under-floor ventilation	Moved from 2308.8.7
2308.4.9 Floor framing supporting braced wall panels	Reference to existing requirements from 2308.12.6 that have been moved to 2308.6.7
2308.4.10 Anchorage of exterior means of egress components in Seismic Design Category D or E	Moved from 2308.12.7
2308.5 Wall Construction	
2308.5.1 Stud size, height and spacing	Moved from 2308.9.1.
2308.5.2 Framing details	Moved from 2308.9.2 Exception #1 from 2308.9.2.3 Exception #2 from 2308.9.2
Table 2308.5.1	From existing Table 2308.9.1 Footnote "c" is from existing language in section 2308.9.1
2308.5.3 Plates and sills	
2308.5.3.1 Bottom plate or sill	From 2308.9.2.4
2308.5.3.2 Top plates	From 2308.9.2.1
2308.5.4 Nonbearing walls and partitions	From 2308.9.2.3
2308.5.5 Openings in walls and partitions	From 2308.9.5.
2308.5.5.1 Openings in exterior bearing walls	From 2308.9.5.1
"Wall studs shall support"	From 2308.9.5.2
2308.5.5.2 Openings in interior bearing partitions	From 2308.9.6
2308.5.5.2 Openings in interior nonbearing partitions	From 2308.9.7.
2308.5.6 Cripple walls	From 2308.9.4
2308.5.7 Bridging	From 2308.9.9
2308.5.8 Pipes in walls	From 2308.9.8
2308.5.9 Cutting and notching	From 2308.9.10
2308.5.10 Bored holes	From 2308.9.11
2308.6 Wall bracing	
2308.6.1 Braced wall line spacing	
Refers to new Table 2308.6.1 that contains spacing information from:	BWL at 35' o.c. from 2308.3.1 BWL in SDC D/E at 25' o.c. from 2308.12.3
2308.6.2 Location of braced panels	From 2308.9.3. Distance of panel from end of wall line (12 ½ feet) was moved to Table 2308.6.1 along with SDC D and E limitation of 8 feet from 2308.12.4
2308.6.3 Braced wall panel methods	From 2308.9.3. items 1 through 8 are re-located into Table 2308.6.3.(1) and renamed;
INEW TADIE 2000.0.0(1)	

	1 LIB Let In Bracing		
	2 DWB Diagonal Wood Boards		
	3 WSP Wood Structural Panels		
	4 SFB Structural Fiberboard Sheathing		
	5 GB Gypsum Board		
	7 PCP Portland Compart Plactor		
	8 HPS Hardboard Papel Siding		
	The two "Alternative bracing" options from 2308.9.3.1 are incorporated into Table 2308.6.3(1) as items 9 and 10		
	9 Alt bracing from 2308.9.3.1		
	10 Alt bracing wall papel adjacent to a door or		
	window opening		
	PFH (Portal Frame w/ Hold-downs)		
2308.6.4 Length of braced wall panels	From 2308.9.3		
2308.6.5 Alternative bracing	From 2308.9.3.1		
2308.6.5.1 Alternate Braced Wall (ABW)	From 2308.9.3.1		
2308.6.5.2 Portal Frame w/ Hold-downs (PFH)	From 2308.9.3.2 "Alternate bracing wall panel adjacent to a door		
2308.6.6 Cripple wall bracing	From 2308.9.4.1		
	From 2209.0.4.4 and 2209.0.4.0		
Category A, B and C	From 2308.9.4.1 and 2308.9.4.2		
2308.6.6.2 Cripple wall bracing in Seismic Design	From 2308.12.4		
2308.6.7 Connections of braced wall panels	From 2308.12.4		
2308.6.6.1 Bottom plate connection	From 2308.3.2.1		
2308.6.6.2 Top plate connection	From 2308.3.2.2		
2308.6.6.3 Sill anchorage	From first portion of 2308.3.3. The remainder of 2308.3.3 is		
	moved to 2308.3.1 "Foundation Plates and Sills"		
	F10ff1 2308.3.3.1		
2308.6.7 Braced wall line support	-		
2308.6.7.1 Foundation requirements	From 2308.3.4		
Cantilever floor provisions	From 2308.12.6, Item 1 (re-worded)		
2308 6 7 2 Eloor and roof diaphragm support in	From 2308 12.6, item 2		
Seismic Design Category D and E	F1011 2306.12.0, Itel11 2		
2308.6.7.3 Stepped footings in SDC B.C.D and E	From 2308.11.3.2		
2308 6.8 Attachment of sheathing	From 2308 12 5		
2308.6.9 Limitation of concrete or masonry veneer			
	From 0000 44 0		
2308.6.9.1 Concrete or masonry veneer in Seismic Design Category B and C	From 2308.11.2		
2308.6.9.2 Concrete or masonry veneer in Seismic	From 2308.12.2		
Design Category D and E			
2308 7 Roof and ceiling framing	From 2308 10 Figure 2308 7 is new and is similar to the Figure		
	in the IRC		
2308.7.1 Ceiling joist spans	From 2308.10.2		
2308.7.2 Rafter spans	From 2308.10.3		
2308.7.3 Ceiling joist and rafter framing	From 2308.10.4		
2308.7.3 Ceiling joist and rafter connections	From 2308.10.4		
2308.7.4 Notches and holes	From 2308.10.4.2		
2209 7 5 Wind unlift			
	From 2308.10.1		

2308.7.6 Openings in roof diaphragms in SDC B, C, D and E	From 2308.11.3.3 The text of this section has been re-arranged for clarity. The first sentence states that a tie and blocking are required. Then, the tie is described followed by the blocking.		
2308.7.7 Purlins	From 2308.10.5		
2308.7.9 Engineered wood products	From 2308.10.7		
2308.7.10 Roof sheathing	From 2308.10.8		
2308.7.11 Joints	From 2308.10.8.1		
2308.7.12 Roof planking	From 2308.10.9		
2308.7.13 Trusses	From 2308.10.10		
2308.7.14 Attic ventilation	From 2308.10.11		

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

S273-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2308-S-RICE.doc

S274–12 2308.2.1

Proponent: Philip Line, American Wood Council

Revise as follows:

2308.2.1 Nominal design wind speed greater than 100 130 mph (3-second gust). Where V_{asd} as determined in accordance with Section 1609.3.1 V_{ult} exceeds 100 130 mph (3-second gust), the provisions of either AF&PA WFCM, or the ICC 600 are permitted to be used. Wind speeds in Figures 1609A, 1609B, and 1609C shall be converted to V_{asd} wind speed in accordance with Section 1609.3.1 for use with AF&PA WFCM or ICC 600.

Reason: ASD wind speeds, V_{asd} , are converted to V_{ult} wind speeds to work directly with V_{ult} wind speed maps in the IBC (Figure 1609A, Figure 1609B, and Figure 1609C). For 2012 WFCM, the conversion to V_{asd} is not applicable as the updated AWC's 2012 WFCM utilizes V_{ult} wind speeds. Text is added to clarify application of 1609.3.1 for determination of V_{asd} wind speeds for use with ICC 600.

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

S274-12					
Public Hearing: Con	nmittee:	AS	AM	D	
Ass	embly:	ASF	AMF	DF	
					2308.2.1-S-LINE.doc
S275–12 2308.2.1

Proponent: Randall Shackelford, P.E., Simpson Strong-Tie Company, Inc. (rshackelford@strongtie.com)

Revise as follows:

2308.2.1 Nominal design wind speed greater than 100 mph (3-second gust). Where V_{asd} as determined in accordance with Section 1609.3.1 exceeds 100 mph (3-second gust), the provisions of either AF&PA WFCM, or the ICC 600 are permitted to be used. Wind speeds in Figures 1609A, 1609B, and 1609C shall be converted in accordance with Section 1609.3.1 for use with AF&PA WFCM or ICC 600.

Reason: The 2012 WFCM, as referenced in Chapter 35 of the 2012 IBC, is based on Ultimate Wind Speeds, V_{ult} , and therefore does not require conversion of the ultimate wind speed to the nominal wind speed, V_{asd} .

Further, the WFCM is the reference standard for wood framing in the ICC-600, so conversion should not take place when using ICC-600 to design wood framing. A committee has been appointed to revise ICC-600, and this code change is written assuming that the basis of ICC-600 will be changed to V_{ut} wind speeds, with conversion factors in the standard for converting to V_{asd} where needed. If by the Public Comment deadline it is not clear that this will be the case, I will prepare a Public Comment to restore Exception 1 to the list of items where conversion is required.

If this code change is not approved, structures designed using the 2012 WFCM with converted wind speeds will be designed for wind speeds that are only 60% of the pressures they should be designed for.

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

S275-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
					2308.2.1-S-SHACKELFORD.doc

ICC PUBLIC HEARING ::: April - May 2012

S276–12 2308.2

Proponent: Charles S. Bajnai, Chesterfield County (bajnaic@chesterfield.gov), VA, Ed Keith, American Plywood Association, representing Chesterfield County, VA, Robert Rice, OBOA, representing Chesterfield County, VA

Revise as follows:

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

3.1. Average dead loads shall not exceed 15 psf (718 N/m2) for combined roof and ceiling, exterior walls, floors and partitions.

Exceptions:

- 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.
- 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.
- 3.2. Live loads shall not exceed 40 psf (1916 N/m²) for floors <u>of conventional light-frame</u> <u>construction</u>.
- 3.3. Ground snow loads shall not exceed 50 psf (2395 N/m²).

(Portions of text not shown remain unchanged)

Reason: The limitation of 40 psf live load for floors from Table 1607.1 makes Section 2308, Conventional Light- Frame Construction, essentially restricted to residential construction.

This code change proposal is intended to clarify that the 40 psf live load for floors applies to all stories constructed of conventional light-frame construction.

This new exemption would allow Section 2308, Conventional Light-Frame Construction to apply to live/work structures, and one story offices, retail spaces, assembly spaces, schools, etc

S276-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2308.2-BAJNAI-RICE.doc

S277–12 2308.2

Proponent: Philip Line, American Wood Council

Revise as follows:

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

1. Buildings shall be limited to a maximum of three *stories above grade plane*. For the purposes of this section, for buildings assigned to *Seismic Design Category* D or E, 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/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).
- V_{see} as determined in accordance with Section 1609.3.1 V_{ult} shall not exceed 100 130 miles per hour (mph) (44 57.2 m/s) (3- second gust).

Exception: V_{act} as determined in accordance with Section 1609.3.1 V_{ult} shall not exceed 110 140 mph (48.4 61.6 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 *Risk Category* IV buildings assigned to *Seismic Design Category* B, C, D, E or F.
- 7. Conventional light-frame construction is limited in irregular structures assigned to Seismic Design Category D or E, as specified in Section 2308.12.6.

Reason: ASD wind speeds, V_{asd} , are converted to V_{ult} wind speeds to work directly with V_{ult} wind speed maps in the IBC (Figure 1609A, Figure 1609B, and Figure 1609C). This change will allow direct comparison of the wind speed limits in 2308.2 Item 4 with IBC wind speed maps for determination of applicability of provisions in 2308 eliminating potential error due to mathematical conversion of V_{asd} to V_{ult} . Use of V_{ult} also better coordinates with the V_{ult} wind speed of 115 mph defined in Chapter 2 for hurricane prone region. Additionally, this change will allow better coordination with V_{ult} basis of WFCM wind design provisions and strength design basis ASCE 7-10 wind load provisions.

The value of 130 mph comes from the solving Equation 16-33 for V_{ult} and rounding as follows: $V_{ult} = (V_{asd})/(0.6^{0.5})$ $V_{ult} = (100 \text{ mph})/(0.6^{0.5}) = 129.099 \text{ mph}$ $V_{ult} = 130 \text{ mph}$

The value of 140 mph comes from solving Equation 16-33 for V_{ult} and rounding as follows: $V_{ult} = (V_{asd})/(0.6^{0.5})$ $V_{ult} = (110 \text{ mph})/(0.6^{0.5}) = 142.009 \text{ mph}$ $V_{ult} = 140 \text{ mph}$

With the exception of rounding to facilitate use of mapped wind speed contours, this change does not introduce technical change to existing wind speed limitations. Rounding up to 130 mph affects locations with V_{ut} wind speed between 129 mph and 130 mph such that provisions of 2308 are now applicable in those locations. Rounding down to 140 mph affects locations with V_{ut} wind speed between 140 mph and 142 mph such that provisions of 2308 are no longer applicable in those locations.

S277-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
, ,				2308.2-S-LINE.doc

S278–12 2308.2

Proponent: Robert Rice, Josephine County, OR, representing Oregon Building Officials Association (structdesigner@yahoo.com)

Revise as follows:

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

1. Buildings shall be limited to a maximum of three *stories above grade plane*. For the purposes of this section, for buildings assigned to *Seismic Design Category* D or E, 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/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).
- V_{asd} as determined in accordance with Section 1609.3.1 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 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*.

- Roof trusses and <u>Ceiling joist and</u>-rafters framing constructed in accordance with Section 2308.10 and trusses shall not span more than 40 feet (12 192 mm) between points of vertical support. <u>A</u> ridge board in accordance with Section 2308.10 or 2308.10.4.1 shall not be considered a vertical support.
- 6. The use of the provisions for *conventional light-frame construction* in this section shall not be permitted for *Risk Category* IV buildings assigned to *Seismic Design Category* B, C, D, E or F.
- 7. Conventional light-frame construction is limited in irregular structures assigned to Seismic Design Category D or E, as specified in Section 2308.12.6.

Reason: This proposal clarifies the requirements of the existing code language. The provisions of the existing code defining the construction of roof/ceiling assemblies with *conventional light-frame construction* are predicated on the fact that a "ridge-board" does

not provide "vertical support". According to the commentary, the current code limitation of "Roof trusses and rafters shall not span more than 40 feet (12 192 mm) between points of vertical support." is intended to limit the use the rafter/ceiling joist (or rafter tie) provisions of "Conventional light-frame construction".

The commentary states:

"In buildings with roof framing spans in excess of 40 feet (12192 mm), the horizontal thrust of that framing on the top plate on which it rests is greater than can be resisted by the ceiling joist and rafter connections specified in Section 2308.10.4.1. Note that the limitation is on the span of the truss or rafter and not on the width of the building. The building width could exceed 40 feet (12192 mm) as long as the actual span of the roof framing is no more than 40 feet (12192 mm)."

The commentary correctly identifies that there are "horizontal thrust" forces in a rafter/ceiling joist assembly. Those forces are addressed in Section 2308.10.4.1 where it states, "Ceiling joists shall be continuous or securely joined where they meet over interior partitions and fastened to adjacent rafters in accordance with **Tables 2308.10.4.1** and 2304.9.1 to provide a continuous rafter tie across the building where such joists are parallel to the rafters." Table 2308.10.4.1 contains the necessary rafter tie connections based on rafter slope, snow load and roof span. The roof span, per the table, is up to 36 feet. In addition, footnote "c" of Table 2308.10.4.1 further verifies this with the statement that, "Rafter tie heel joint connections are not required where the ridge is supported by a load-bearing wall, header or ridge beam." An error exists in the statement of the commentary in that trusses do not impose the "horizontal thrust" on the top plate of the wall like rafter/ceiling joist framing does. The horizontal forces of a truss at it's bearing points are non-existent, or negligible, due to the fact that the forces are resolved within the chords and web members of the truss and only vertical loads exist at it's bearing points such as on the exterior walls.

However, a second concern exists and is a factor in limiting the roof span to 40 feet. The bearing wall studs in Table 2308.9.1 are limited in their capacity to resist buckling due to the vertical (axial) forces and the unbraced length of the studs. When considering the load limitations of 2308.2 item 3.1, 15 psf dead load, and item 3.3, snow load of 50 psf, the combined roof load could be 65 psf. A 40 foot span would result in a load of 65 x 40/2 = 1300 plf to the top plates. With studs at 16 inch o.c. the load/stud = 1300 x (16/12) = 1733#/stud.

Therefore, the purpose of this proposal is to clarify that a non-vertically-supporting "ridge board" is not to be considered a "vertical support". If it were to be mistakenly considered to be a support, the tributary roof load would far exceed that capacity of the studs as well as the limitations of the values in the rafter tie table. This clarification will not effect the requirements for wall bracing and the location, or spacing, of braced wall lines. Currently, braced wall lines are required at 35 feet o.c. in each direction in *Seismic Design Category* A, B and C and 25 feet o.c. in each direction in *Seismic Design Category* D and E. *For reference, sections 2308.10 and 2308.10.4.1 state:*

2308.10 Roof and ceiling framing. The framing details required in this section apply to roofs having a minimum slope of three units vertical in 12 units horizontal (25-percent slope) or greater. Where the roof slope is less than three units vertical in 12 units horizontal (25-percent slope), members supporting rafters and ceiling joists such as ridge board, hips and valleys shall be designed as beams.

2308.10.4.1 Ceiling joist and rafter connections. Ceiling joists and rafters shall be nailed to each other and the assembly shall be nailed to the top wall plate in accordance with Tables 2304.9.1 and 2308.10.1. Ceiling joists shall be continuous or securely joined where they meet over interior partitions and fastened to adjacent rafters in accordance with Tables 2308.10.4.1 and 2304.9.1 to provide a continuous rafter tie across the building where such joists are parallel to the rafters. Ceiling joists shall have a bearing surface of not less than 11/2 inches (38 mm) on the top plate at each end. Where ceiling joists are not parallel to rafters, an equivalent rafter tie shall be installed in a manner to provide a continuous tie across the building, at a spacing of not more than 4 feet (1219 mm) o.c. The connections shall be in accordance with Tables 2308.10.4.1 and 2304.9.1, or connections of equivalent rafter ties shall be provided. Where ceiling joists or rafter ties are not provided at the top of the rafter support walls, the ridge formed by these rafters shall also be supported by a girder conforming to Section 2308.4.

Cost Impact: The code change proposal will not increase the cost of construction. This proposal does not add any new requirement or limitation to the code. It is intended to clarify the code for consistency in application.

S278-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2308.2-S-RICE.doc

S279–12 2308.3.2.2

Proponent: Robert Rice, C.B.O, Josephine County, OR, representing Oregon Building Officials Association (structdesigner@yahoo.com), R. Terry Malone, P.E., S.E., representing self

Revise as follows:

2308.3.2.2 Top plate connection. Where joists and/or rafters are used, braced wall line top plates shall be fastened over the full length of the braced wall line to joists, rafters, rimboards or <u>full-depth</u> 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 at joists with walls above shall be equal to the depth of the joist at the braced wall line. Blocking at rafters need not be full depth but shall extend to within 2 inches (51 mm) from the roof sheathing above. Blocking shall be a minimum of 2 inches (51 mm) nominal thickness and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.8.2 or Section 2308.10.4.2 shall be permitted.

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 over the full length of the braced wall line 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 shall extend to within 2 inches (51 mm) from the roof sheathing above 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. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.8.2 or Section 2308.10.4.2 shall be permitted.

Reason: In the last code cycle for the development of the 2012 code, section 2308.3.2 was modified. The proposal (S211) rearranged the section into bottom plate connections (2308.3.2.1) and top plate connections (2308.3.2.2). Another proposal was submitted (S212) to make technical changes to this section regarding the blocking between joists, rafters or trusses particularly at high-heel or cantilevered trusses. The 2009 IBC language specifically stated that the blocking was required to be "full-height". As a result of working with other stake-holders and industry representatives, a provision was written into S211 to allow the blocking to stop 2 inches short of the roof sheathing. This provision was intended as a method of allowing for the required venting. Reports and analysis were cited that indicated that the cross-grain bending of the rafter or truss chord was not a significant concern and that the diaphragm forces could be transferred through typical connections and fastening per Table 2304.9.1. However, there has been concern raised since that time that the 2 inch gap at the top causes a disconnect in the lateral load path and is not consistent with referenced standards and other sections of the IBC.

All diaphragm testing and accepted allowable diaphragm shear value tables (past and present) are based on diaphragms having boundary nailing. This nailing is required to transfer diaphragm shears into the boundary elements (shear walls and/or collectors and struts), in accordance with IBC section 1602.1 and ASCE7 section 11.2. If a 2" air gap is allowed between the sheathing and the top of the blocking, this shear transfer cannot happen and the allowable shear values should not be allowed to be used.

The definition of a diaphragm boundary from the 2012 IBC states;

Diaphragm boundary. In light-frame construction, a location where shear is transferred into or out of the diaphragm sheathing. Transfer is either to a boundary element or to another force-resisting element.

IBC section 1604.4, ASCE 7 section 1.3.5, and SDPWS sections 4.1.1 and 4.2.6 all require complete load paths. Since the 2" air gap does not allow a direct load path for the transfer of diaphragm shears to the blocking, then down into the shear walls or collectors, an alternate load path must be provided. With the 2" gap, the diaphragm shears and resulting load path must be transferred through the unsupported diaphragm sheathing, which must act as the initial diaphragm boundary element taking tension and compression (not allowed by IBC section 2305.1.2 and SDPWS section 4.1.4), then into the trusses or joists, then by bearing (assuming full bearing is achieved) into the blocking, and then down into the boundary element. Past and present testing has shown that eliminating blocking, providing partial (skip) blocking or reducing the height of blocking produces failure modes that are undesirable (i.e. trust/joist rotation, loss of gang-nail plates by popping off from cross grain shear forces being applied, or shifting of loads to other members that were not designed to receive those loads). At the very least, the gap should occur at the bottom of the blocking so that the boundary nailing can be installed. However, doing so will not resolve the bad testing failure modes.

Installing blocking only over the shear walls would create shears in the blocking and its connections (transferring the shears into the framing) in excess the connection capacity shown in the prescriptive fastening schedules in the IBC tables, and would also

eliminate the boundary elements connecting the shear walls together. This violates IBC sections 1602.1 (boundary member and chord), 2302, 2305.1.2, SDPWS sections 4.1.4, 4.1.1 and 4.2.6, and ASCE 7 sections 11.2, 12.10.2 and 1.3.5. The shears are not only being applied in the plane of the wall. Loads are applied to the diaphragm in both the transverse and longitudinal directions. When the loads are applied in the transverse direction (perpendicular to the wall), without blocking, or if installed only at the shear walls, the diaphragm sheathing is the only element that can act as the diaphragm chord because the shears cannot be transferred to the blocking and therefore the sheathing must take all of the tension and compression forces, which is in direct violation with the code.

Cost Impact: This change would require that the blocking be 2 inches taller than what is currently required. The additional cost would be negligible. In addition, this change would require boundary nailing (6 inch o.c.) of the roof sheathing to the blocking along the braced wall line. That would be an additional, but undetermined, cost.

S279-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				2308.3.2.2-S-RICE-MALONE.doc

S280–12 2308.2.2

Proponent: Robert Rice, C.B.O., Josephine County, OR, representing Oregon Building Officials Association (structdesigner@yahoo.com)

Revise as follows:

2308.3.2.2 Top plate connection. Where joists and/or rafters are used, braced wall line top plates shall be fastened over the full length of the braced wall line to joists, rafters, rimboards or 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 at joists with walls above shall be equal to the depth of the joist at the braced wall line. Blocking at rafters need not be full depth but shall extend to within 2 inches (51 mm) from the roof sheathing above. Blocking shall be a minimum of 2 inches (51 mm) nominal thickness and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.8.2 or Section 2308.10.4.2 shall be permitted.

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 over the full length of the braced wall line 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 shall extend to within 2 inches (51 mm) from the roof sheathing above and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.8.2 or Section 2308.10.4.2 shall be permitted.

Exception. 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:

- 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.



a. Methods of bracing shall be as described in Section 2308.9.3 method 2, 3, 4, 6, 7 or 8

For SI: 1 inch = 25.4 mm

a. Methods of bracing shall be as described in Section 2308.9.3, method 2,3,4,6,7 or 8

FIGURE 2308.3.2(1) BRACED WALL LINE TOP PLATE CONNECTION



a. Methods of bracing shall be as described in Section 2308.9.3, method 2,3,4,6,7 or 8

FIGURE 2308.3.2 (2) BRACED WALL PANEL TOP PLATE CONNECTION

TABLE 2304.9.1 FASTENING SCHEDULE								
CONNECTION	CONNECTION FASTENING ^a LOCATION							
1. Joist to sill or girder	3 - 8d common (2 1/2" x 0.131") 3 – 3 x 0.131 nails 3 – 3" x 14 gage staples	toenail						
2. Bridging or blocking to joist, rafter or truss	2 - 8d common (2 1/2" 0.131") 2 – 3 x 0.131" nails 2 – 3" x 14 gage staples	toenail each end						
11. Blocking between joists, or rafters <u>or truss</u> to top plate	3 - 8d common (2 1/2" x 0.131") 3 – 3" x 0.131 nails 3 – 3" 14 gage staples	toenail						
Blocking between rafters or truss not at the wall top plate, to rafter or truss	<u>2 - 8d common (2 1/2" x 0.131")</u> <u>2 - 3" x 0.131" nails</u> <u>2 - 3" 14 gage staples</u>	toenail each end						
	<u>2 - 16d common (3 1/2" x 0.162")</u> <u>3 - 3" x 0.131" nails</u> <u>3 - 3" x 14 gage staples</u>	<u>endnail</u>						

(Portions of table not shown remain unchanged)

Reason: The 2012 IBC has fairly clear wording in Section 2308.3.2 that when the Conventional Light-Frame Construction provisions are used that the diaphragms need to be connected to the braced wall line to resist wind and seismic (lateral) forces and states.

The prescriptive provisions of "conventional light-frame construction" as provided for in section 2308 are very limited in scope. In section 2308.2 they are limited to:

1. Three stories max (two stories max in SDC C, one story in SDC D and above)

2. Max floor to floor height of 11'-7"

3. Max dead loads of 15 psf

4. Floor live load of 40 psf max

5. Ground snow of 50 psf max

6. Wind speeds of 100 max

7. Roof truss span of 40 feet max between vertical supports

8. Not allowed to be used for Occupancy Category IV buildings in SDC B,C,D,E

9 More restrictive requirements for SDC B,C, D and E defined in 2308.11.

10 Even more restrictive requirements specifically for SDC D and E

11. Limited by "irregular structures" definitions in 2308.12.6

12. Braced wall line spacing 35 feet max each direction, each floor.

13. In SDC D and E max spacing is 25 feet. (IRC allow exception up to 50 feet)

In other words, due to the limitations listed above as well as the other limitations in the code not listed here, the structures that are built with the provisions of section 2308 are small, light-framed buildings that do not have the significant lateral loading that other buildings do.

The alternate provisions in the exceptions are intended to address the increasingly common occurrence of cantilevered/highheel trusses. This occurs due to insulation requirements and to provide a cantilevered portion of roof to be an exterior covered porch. The current provisions of this section of code do not cover this common condition. The current code language requires that "Blocking shall be a minimum of 2 inches (51 mm) nominal thickness..." This does not work for heights greater than what a 2x 10 or 2x 12 will accommodate.

The current code text (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 in 2006 for the adoption of the 2006 IBC and has worked well for many years and two more code cycles Since then, countless hours have gone into developing proposals for both the IRC and the IBC code development process. The IRC proposal was approved in Minneapolis for the 2009 code. During the process of resolving concerns and developing a consensus changes were made to the proposal. 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.

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, per the current text, is 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 to complete the load path is often not fully understood by plans examiners, inspectors and contractors. The typical requirement that is intended by the code is that full height solid blocking occur at this connection with edge nailing to the blocking and 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.

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 roof diaphragm sheathing to the wall top plates either vertically in the truss bays or horizontally through the soffit. No design 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.

S280-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
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S281–12 2308.7, 2308.9.1, 2308.9.5.1, 2308.9.5.2, 2308.9.6, Table 2308.9.5, Table 2308.9.6

Proponent: Paul Coats, PE, CBO, American Wood Council (pcoats@awc.org)

Revise as follows:

2308.7 Girders. Girders for single-story construction or girders supporting loads from a single floor shall not be less than 4 inches by 6 inches (102 mm by 152 mm) for spans 6 feet (1829 mm) or less, provided that girders are spaced not more than 8 feet (2438 mm) o.c. Spans for built-up 2-inch (51 mm) girders shall be in accordance with Table 2308.9.5 or 2308.9.6. Other girders Girders shall be designed to support the loads specified in this code. Girder end joints shall occur over supports. Where a girder is spliced over a support, an adequate tie shall be provided. The ends of beams or girders supported on masonry or concrete shall not have less than 3 inches (76 mm) of bearing.

2308.9.1 Size, height and spacing. The size, height and spacing of studs shall be in accordance with Table 2308.9.1 except that utility-grade studs shall not be spaced more than 16 inches (406 mm) o.c., or support more than a roof and ceiling, or exceed 8 feet (2438 mm) in height for exterior walls and load-bearing walls or 10 feet (3048 mm) for interior nonload-bearing walls. Studs shall be continuous from a support at the sole plate to a support at the top plate to resist loads perpendicular to the wall. The support shall be a foundation or floor, ceiling or roof diaphragm or shall be designed in accordance with accepted engineering practice.

Exception: Jack studs, trimmer studs and cripple studs at openings in walls that comply with Table 2308.9.5 Section 2308.9.5.2.

2308.9.5.1 Headers. Headers shall be provided over each opening in exterior-bearing walls. The spans in Table 2308.9.5 are permitted to be used for one- and two-family *dwellings*. Headers for other buildings shall be designed in accordance with Section 2301.2, Item 1 or 2. Headers shall be of two <u>or more</u> pieces of nominal 2-inch (51 mm) framing lumber set on edge as permitted by Table 2308.9.5 and nailed together in accordance with Table 2304.9.1 or of solid lumber of equivalent size.

2308.9.5.2 Header support. Wall studs shall <u>be designed to</u> support the ends of the header in accordance with Table 2308.9.5. Each end of a lintel or header shall have a length of bearing of not less than $1_{1/2}$ inches (38 mm) for the full width of the lintel.

2308.9.6 Openings in interior bearing partitions. Headers shall be provided over each opening in interior bearing partitions as required in Section 2308.9.5. The spans in Table 2308.9.6 are permitted to be used. Wall studs shall support the ends of the header in accordance with Table 2308.9.5 or 2308.9.6, as appropriate Section 2308.9.5.2.

TABLE 2308.9.5HEADER AND GIRDER SPANS* FOR EXTERIOR BEARING WALLS(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir* and
Required Number of Jack Studs)

TABLE 2308.9.6

HEADER AND GIRDER SPANS^a FOR INTERIOR BEARING WALLS (Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir^b and Required Number of Jack Studs)

Reason: Deletion of Table 2308.9.5 and Table 2308.9.6 without replacement is proposed because of limited applicability of the tabulated header spans resulting from the exclusion of detached one- and two-family dwellings from the scope of 2308 and the live load limitation of 40 psf per 2308.2. In addition, the species-based header spans are subject to being dated should design values change. Design value-based prescriptive engineered options for header spans are available from other sources. For example,

header spans for conditions covered by Table 2308.9.5 and Table 2308.9.6, as well as support of headers by use of jack studs providing full bearing, can be found in the WFCM.

Specific reference to "one- and two- family dwellings" from 2308.9.5.1 is deleted to coordinate with the exclusion of detached one- and two-family dwellings from the scope of 2308. Other text sections are revised to coordinate with removal of the Tables.

S281-12					
Public Hearing: C	Committee:	AS	AM	D	
A	ssembly:	ASF	AMF	DF	
	•				2308.7-S-COATS.doc

S282-12 2308.8.5

Proponent: Robert Rice, C.B.O., Josephine County, OR, representing Oregon Building Officials Association (structdesigner@yahoo.com)

Revise as follows:

2308.8.5 Lateral support. Floor, *attic* and roof framing with a nominal depth-to-thickness ratio greater than or equal to 5:1 shall have one the compression edge held in line for the entire span. Where the nominal depth-to-thickness ratio of the framing member exceeds 6:1, there shall be one line of bridging for each 8 feet (2438 mm) of span, unless both edges of the member are held in line. The bridging shall consist of not less than 1-inch by 3-inch (25 mm by 76 mm) lumber, double nailed at each end, of equivalent metal bracing of equal rigidity, full-depth solid blocking or other *approved* means. A line of bridging shall also be required at supports where equivalent lateral support is not otherwise provided.

Reason: This proposal clarifies the requirements of the existing code language. The first sentence requires framing with a depth-tothickness ratio greater or equal to 5:1 (e.g. 2x10) to have "...one edge held in line...". The second sentence states that when the depth-to-thickness ratio exceeds 6:1 (e.g. 2x12) "...there shall be one line of bridging for each 8 feet of the span..." in addition to the requirement above unless "both edges of the member are held in line." The remainder of the section describes what the bridging shall be. What is missing from the first sentence is the clarification that it is the compression flange that requires bracing or "support". This is consistent with accepted engineering practice and design standards such as the National Design Specification published by American Forest and Paper Association.

The Commentary states,

When the depth-to-thickness ratio of joists and rafters exceeds 5:1, as would be the case in members larger than 2 inches by 10 inches (51 mm by 254 mm), the lateral support required by Ssection 2308.8.2 is not sufficient to prevent **lateral buckling** between supports. Additional resistance is required. **Sheathing, subflooring, decking and similar materials** attached to each joist or rafter are considered to provide edge restraint. These requirements are cumulative. The support required by Section 2308.8.2 applies to all joists. Additionally, members greater than 2 inches by 10 inches (51 mm by 254 mm) must have one edge held in line, and members greater than 2 inches by 12 inches (51 mm by 305 mm) must have one edge held in line as well as a line of bridging at each 8 feet (2438 mm) of span (which may be omitted if both edges are held in line).

As indicated by the commentary above, the concern is "..lateral buckling between supports." The susceptibility of lateral buckling for floor, attic or roof framing is due to an un-braced compression flange. Section 2308.8.2 requires that "Joists shall be supported laterally at the ends and at each support by solid blocking except where the ends of the joists are nailed to a header, band or rim joist or to an adjoining stud or by other means." The requirements of this section, 2308.8.5 are in addition to 2308.8.2 and are specifically to address out-of-plane buckling of the compression flange.

Cost Impact: The code change proposal will not increase the cost of construction. This proposal is intended to clarify the code and does not add any new requirement to the code.

S282-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly.	ASE	AIVIE	DF	
					2308.8.5-S-RICE.doc

S283-12

2308.8, Table 2308.8(1), Table 2308.8(2), 2308.10.2, Table 2308.10.2(1), Table 2308.10.2(2), 2308.10.3, Table 2308.10.3(1), Table 2308.10.3(2), Table 2308.10.3(3), Table 2308.10.3(4), Table 2308.10.3(5), Table 2308.10.3(6)

Proponent: Paul Coats, P.E. CBO, American Wood Council (pcoats@awc.org)

Revise as follows:

2308.8 Floor joists. Spans for floor joists shall be in accordance with Table 2308.8(1) or 2308.8(2). For other grades and or species, refer to the AF&PA Span Tables for Joists and Rafters.

TABLE 2308.8(1) FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES (Residential Sleeping Areas, Live Load = 30 psf, L/∆ = 360)

TABLE 2308.8(2) FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES (Residential Living Areas, Live Load = 40 psf, L/△ = 360)

2308.10.2 Ceiling joist spans. Allowable spans for ceiling joists shall be in accordance with Table 2308.10.2(1) or 2308.10.2(2). For other grades and species, refer to the AF&PA <u>AWC</u> Span Tables for Joists and Rafters.

TABLE 2308.10.2(1) CEILING JOIST SPANS FOR COMMON LUMBER SPECIES (Uninhabitable Attics Without Storage, Live Load = 10 pounds psf, L/△ = 240)

TABLE 2308.10.2(2) CEILING JOIST SPANS FOR COMMON LUMBER SPECIES (Uninhabitable Attics With Limited Storage, Live Load = 20 pounds per square foot, L/△ = 240)

2308.10.3 Rafter spans. Allowable spans for rafters shall be in accordance with Table 2308.10.3(1), 2308.10.3(2), 2308.10.3(3), 2308.10.3(4), 2308.10.3(5) or 2308.10.3(6). For other grades and species, refer to the *AF&PA* the *AWC* Span Tables for Joists and Rafters.

TABLE 2308.10.3(1) RAFTER SPANS FOR COMMON LUMBER SPECIES (Roof Live Load = 20 pounds per square foot, Ceiling Not Attached to Rafters, L/△ = 180)

TABLE 2308.10.3(2)

RAFTER SPANS FOR COMMON LUMBER SPECIES

(Roof Live Load = 20 pounds per square foot, Ceiling Attached to Rafters, L/A = 240)

TABLE 2308.10.3(3)

RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 30 pounds per square foot, Ceiling Not Attached to Rafters, L/A = 180)

TABLE 2308.10.3(4)

RAFTER SPANS FOR COMMON LUMBER SPECIES

(Ground Snow Load = 50 pounds per square foot, Ceiling Not Attached to Rafters, L/△ = 180)

TABLE 2308.10.3(5) RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 30 pounds per square foot, Ceiling Attached to Rafters, L/△ = 240)

TABLE 2308.10.3(6) RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 50 pounds per square foot, Ceiling Attached to Rafters, L/△ = 240)

Reason: Species- and grade-specific span tables are subject to becoming dated if design values for specific species or grades change, and therefore it is proposed to directly reference the AWC Span Tables for Joists and Rafters. The design value format of the tabulated spans in Span Tables for Joists and Rafters is not sensitive to design value changes for specific species and grades. Span Tables for Joists and Rafters is currently included as a reference in IBC 2306.

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

S283-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	2				2308.8-S-COATS.doc

S284-12 2308.9.2.1

Proponent: Edward L. Keith, APA – The Engineered Wood Association (ed.keith@apawood.org)

Revise as follows:

2308.9.2.1 Top plates. Bearing and *exterior wall* studs shall be capped with double top plates installed to provide overlapping at corners and at intersections with other partitions. End joints in double top plates shall be offset at least 48 inches (1219 mm), and shall be nailed with not less than eight 16d face nails on each side of the joint. Plates shall be a nominal 2 inches (51 mm) in depth and have a width at least equal to the width of the studs.

Exception: A single top plate is permitted, provided the plate is adequately tied at joints, corners and intersecting walls by at least the equivalent of 3-inch by 6-inch (76 mm by 152 mm) by 0.036-inch-thick (0.914 mm) galvanized steel plate that is nailed to each wall or segment of wall by six 8d $2-1/2" \times 0.113"$) nails or equivalent on each side of the joint. For the butt-joint splice between adjacent single top plates at least the equivalent of 3-inch by 12-inch (76 mm by 304 mm) by a 0.036-inch-thick (0.914 mm) galvanized steel plate that is nailed to each wall or segment of wall by twelve 8 d ($2-1/2" \times 0.113"$) nails on each side of the joint shall be required, provided the rafters, joists or trusses are centered over the studs with a tolerance of no more than 1 inch (25 mm). The top plate may be omitted over headers that are adequately tied to adjacent wall sections with steel plates or equivalent as previously described for the butt joint splice between adjacent single top plates

Reason: Item 10 of the 2012 IBC Table 2304.9.1 establishes the minimum capacity required to insure an adequate tension splice in top plates. Aside from simply providing continuity between wall segments, the top-plate splice also acts as a tension tie (often called a collector or drag strut) to distribute the roof and floor shear loads into the bracing elements often spaced as much as 20 feet apart. Assuming spruce-pine-fir top plates, Table 2304.9.1, item 10 requires a top-plate splice with eight 16d box nails on each side of the splice. In accordance with the NDS Table 11N, assuming SPF plates and a duration of load of 1.6 for lateral loads, the design capacity of the item 10 connection is (88 lb/nail x 8 nails x 1.6 dol =) 1,126 lbf.

While sufficient for intersections and corners the 3-*inch by* 6-*inch* (76 mm by 152 mm) by a 0.036-*inch-thick* (0.914 mm) galvanized steel <u>plate</u> that is nailed to each wall or segment of wall by six 8d nails on each side..." only provides about 600 lbf tension capacity (NDS Table 11P, SPF framing, box nails: 60 lbf/nail x 6 nails x 1.6 dol = 576 lbf). This is about ½ of what is required in Table 2304.9.1, item 10. As such, the splice plate requirement for in-line butt joints in single top plate systems should be twice what is currently required:

"... the equivalent of 3-inch by 12-inch (76 mm by 304) by a 0.036-inch-thick (0.914 mm) galvanized steel plate that is nailed to each wall or segment of wall by twelve 8d (2-1.2" x 0.113") nails on each side..."

As a matter of clarification, the type of nail to be used was defined by description as only the penny-weight was specified. This is in keeping with current code style guidelines. I also specified which splice type was appropriate for headers when present. This was taken from the IRC. As these are neither corners nor intersections, it is clear that the butt-joint splice was the appropriate reference.

S284-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				2308.9.2.1-S-KEITH.doc

S285–12 Table 2308.9.1, 2308.9.2.3

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

TABLE 2308.9.1

Revise as follows:

SIZE, HEIGHT, AND SPACING OF WOOD STUDS								
STUD		BEARIN	G WALLS		NONBEARING	G WALLS		
SIZE	Laterally	Supporting	Supporting	Supporting	Laterally	<u>Maximum</u>		
(INCHES)	unsupported	roof and	one floor, roof	two floors,	unsupported	<u>Stud</u>		
	stud height ^a	ceiling	and ceiling	roof and	stud height ^a	<u>S</u> spacing		
	(feet)	only		ceiling	(feet)	(inches)		
	Ma	ximum Stud &	<u>}s</u> pacing (inches))				
2x3 ^b	-	-	-	-	10	16		
2x4	10	24	16	-	14	24		
3x4	10	24	24	16	14	24		
2x5	10	24	24	-	16	24		
2x6	10	24	24	16	20	24		

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

a. Listed heights are distances between points of lateral support placed perpendicular to the plane of the wall. Increases in unsupported height are permitted where justified by an analysis.

b. Shall not be used in exterior walls.

2308.9.2.3 Nonload-bearing walls and partitions. In nonload-bearing walls and partitions, when not part of a braced wall line, studs shall be spaced not more than 28 24 inches (711 610 mm) o.c. and In interior nonload-bearing walls and partitions, are permitted to be set with the long dimension parallel to the wall. Where studs are set with the long dimensions parallel to the wall use of utility grade lumber or studs exceeding 10 feet (3048 mm) is not permitted. 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: Several minor modifications to nonbearing walls and partitions are proposed. Changes include:

- 1. Limit spacing to 24". Studs bending about the strong axis, as shown in Table 2308.9.1, are limited to 24" on center, so the same should also be applied to flat wise (weak axis bending) studs. Also note that the NDS, National Design Specification, the Repetitive Member Factor Cr is limited to framing members spaced not more than 24 inches on center.
- Exclude the use of utility grade flat wise studs and studs over 10 feet in height because the bending stress exceeds the NDS allowable stress limits. For example, 2x4 #3 Spruce-Pine-Fir studs @ 28" o.c. have an allowable maximum span of 7'-6" versus the Table 2308.9.1 limit of 14'-0".
- 3. Limit to exclude braced wall lines, to match the requirements of IRC R602.5 which states the following: "R602.5 Interior nonbearing walls. Interior nonbearing walls shall be permitted to be constructed with 2 inch by 3 inch (51 mm by 76 mm) studs spaced 24 inches (610 mm) on center or, when part of a *braced wall line*, 2 inch by 4 inch (51 mm by 102 mm) flat studs spaced at 16 inches (406 mm) on center. Interior nonbearing walls shall be capped with at least a single top plate. Interior nonbearing walls shall be fireblocked in accordance with Section R602.8."
- 3. Add the words *Maximum Stud* before Spacing to better define the spacing limit. This will also to match the language in the wood stud table in the International Residential Code, IRC Table R602.3(5).
- 4. Change wording of nonbearing to nonload-bearing to match the definition as shown in IBC Section 202 and Section 2308.9.1

S285-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2308.9.2.3-S-HUSTON.doc

S286-12 2308.9.2.3

Proponent: Robert Rice, C.B.O., Josephine County, OR, representing Oregon Building Officials Association (structdesigner@yahoo.com)

Revise as follows:

2308.9.2.3 Nonbearing walls and partitions. In nonbearing walls and partitions <u>that do not serve as</u> <u>braced wall panels</u>, studs shall be spaced not more than 28 inches (711 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 (12.7 mm by 38 mm) metal ties with spliced sections fastened with two 16d nails on each side of the joint.

Reason: This proposal clarifies that studs cannot be installed flat when interior walls serve as wall bracing walls and panels.

S286-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2308.9.2.3-S-RICE.doc

S287–12 202 (NEW), 2302, 2308.9.3 (NEW), 2304.6, Table 2304.6, 2304.6.1, 2304.6.2,

Proponent: Paul Coats, American Wood Council, (pcoats@awc.org)

Add new text as follows:

SECTION 202 DEFINITIONS

GABLE. The triangular portion of the wall beneath a dual-slope, pitched, or mono-slope roof.

Revise as follows:

2302.1 Definitions. For the purposes of this chapter, and as used elsewhere in this code the following terms are defined in Chapter 2:

GABLE

2304.6 <u>Exterior</u> wall sheathing. Except as provided for in Section 1405 for weatherboarding or where stucco construction that complies with Section 2510 is installed, enclosed buildings shall be sheathed with one of the materials of the nominal thickness specified in Table 2304.6 or any other *approved* material of equivalent strength or durability Wall sheathing on the outside of exterior walls, including gables, and the connection of sheathing to framing shall be designed in accordance with the general provisions of this code and shall be capable of resisting wind pressures in accordance with Section 1609.

2304.6.1 Wood structural panel sheathing. Where wood structural panel sheathing is used as the exposed finish on the outside of exterior 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 Wood structural panel sheathing used to resist wind pressures, connections, and framing spacing shall be in accordance with Table 2304.6.1 for the applicable wind speed and exposure category when used with enclosed buildings with a mean roof height not greater than 30 feet (9144 mm) and a topographic factor (K_{zt}) of 1.0.

2304.6.2 <u>2304.7</u> 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 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.

2308.9.3 Exterior wall sheathing. Except where stucco construction that complies with Section 2510 is installed, the outside of exterior walls, including gables, of enclosed buildings shall be sheathed with one of the materials of the nominal thickness specified in Table 2308.9.3. with fasteners in accordance with requirements of 2304.9 or fasteners designed in accordance with accepted engineering practice.

MINIMUM THICKNESS OF WALL SHEATHING						
SHEATING TYPE	MINIMUM THICKNESS	MAXIMUM WALL STUD SPACING				
Wood boards	5/8 inch	24 inches on center				
Fiberboard	1/2 inch	16 inches on center				
Wood structural panel	In accordance with Tables 2308.9.3(2) and 2308.9.3(3)					
M-S "Exterior Glue" and M-2	In accordance with Section					

TABLE 2304.6 2308.9.3 MINIMUM THICKNESS OF WALL SHEATHING

SHEATING TYPE	MINIMUM THICKNESS	MAXIMUM WALL STUD SPACING
"Exterior Glue" Particleboard	2306.3 and Table 2308.9.3(4)	
Gypsum sheathing	½ inch	16 inches on center
Gypsum wallboard	½ inch	24 inches on center
Reinforced cement mortar	1 inch	24 inches on center

For SI: 1 inch = 25.4 mm.

Reason: (2308.9.3) This new section comes from existing Section 2304.6. The content of the current section is moved to 2308.9.3 because it contains prescriptive minimum sheathings more suitable for wind speeds in accordance with limitations of 2308. The section is clarified as being applicable to exterior wall sheathing. The term "gable" is included to clarify that exterior wall sheathing recommendations are equally applicable to the gable.

Table 2304.6 is moved and renumbered as Table 2308.9.3. Gypsum wallboard is removed from the table to make it clear the table applies to exterior wall sheathing, in accordance with the proposed Section 2308.9.3.

Section 2304.6 is rewritten to establish minimum structural performance requirements and clarify that wall sheathing on the outside of exterior walls, as well as connection of sheathing to framing, must be capable of resisting wind pressures in accordance with Section 1609. The term "gable" is included to clarify that exterior wall sheathing recommendations for out of plane wind resistance are equally applicable to the gable.

Revisions to 2304.6.1 coordinate with the minimum structural performance requirements added in the new 2304.6. Prior language covering design for out of plane wind resistance is deleted because it is addressed in new section 2304.6. Reference to Table 2304.6.1 is revised to clarify that several factors are critical for determination of the applicable maximum wind speed including fastener schedule and stud spacing.

This renumbers Section 2304.6.2 to 2304.7 to separate provisions for Interior Paneling from 2306.6 which would contain new provisions applicable to exterior wall sheathing but not to interior paneling.

A definition is added for "gable" used in proposed revisions in Item #1 and #2 to clarify that gables should be sheathed in accordance with provisions for walls.

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

S287-12

Public Hearing: Committe	ee: AS	AM	D	
Assembly	y: ASF	AMF	DF	
				2308.9.3 (NEW)-S-COATS.doc

S288-12 2308.9.3

Proponent: Paul Coats, P.E., CBO, American Wood Council (pcoats@awc.org)

Revise as follows:

2308.9.3 Bracing. Braced wall lines shall consist of braced wall panels that meet the requirements for location, type and amount of bracing as shown in Figure 2308.9.3, specified in Table 2308.9.3(1) and are in line or offset from each other by not more than 4 feet (1219 mm). Braced wall panels shall start not more than 121/2 feet (3810 mm) from each end of a braced wall line. Braced wall panels shall be clearly indicated on the plans. Construction of braced wall panels shall be by one of the following methods:

- 1. Nominal 1-inch by 4-inch (25 mm by 102 mm) continuous diagonal braces let into top and bottom plates and intervening studs, placed at an angle not more than 60 degrees (1.0 rad) or less than 45 degrees (0.79 rad) from the horizontal and attached to the framing in conformance with Table 2304.9.1.
- Wood boards of 5/8 inch (15.9 mm) net minimum thickness applied diagonally on studs spaced not over 24 inches (610 mm) o.c.
- 3. Wood structural panel sheathing with a thickness not less than 3/8 inch (9.5 mm) for 16-inch (406 mm) or 24-inch (610 mm) stud spacing in accordance with Tables 2308.9.3(2) and 2308.9.3(3).
- Fiberboard sheathing panels not less than 1/2 inch (12.7 mm) thick applied vertically or horizontally on studs spaced not over 16 inches (406 mm) o.c. where installed with fasteners in accordance with Section 2306.6 and Table 2306.6 Table 2304.9.1.
- Gypsum board [sheathing 1/2-inch-thick (12.7 mm) by 4-feet-wide (1219 mm) wallboard or veneer base] on studs spaced not over 24 inches (610 mm) o.c. and nailed at 7 inches (178 mm) o.c. with nails as required by Table 2306.7 along panel edges (including top and bottom plates) and 7" o.c. in the field with 5d (0.086 inch diameter) cooler nails.
- 6. Particleboard wall sheathing panels where installed in accordance with Table 2308.9.3(4).
- 7. Portland cement plaster on studs spaced 16 inches (406 mm) o.c.installed in accordance with Section 2510.
- 8. Hardboard panel siding where installed in accordance with Section 2303.1.6 and Table 2308.9.3(5).

For cripple wall bracing, see Section 2308.9.4.1. For Methods 2, 3, 4, 6, 7 and 8, each panel must be at least 48 inches (1219 mm) in length, covering three stud spaces where studs are spaced 16 inches (406 mm) apart and covering two stud spaces where studs are spaced 24 inches (610 mm) apart.

For Method 5, each panel must be at least 96 inches (2438 mm) in length where applied to one face of a panel and 48 inches (1219 mm) where applied to both faces. All vertical joints of panel sheathing shall occur over studs and adjacent panel joints shall be nailed to common framing members. Horizontal joints shall occur over blocking or other framing equal in size to the studding except where waived by the installation requirements for the specific sheathing materials. Sole plates shall be nailed to the floor framing and top plates shall be connected to the framing above in accordance with Section 2308.3.2. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the braced wall panels.

Reason: In the 2012 code, some provisions for fasteners in Chapter 23 were removed and the AF&PA Special Design Provisions for Wind and Seismic was referenced instead. This proposed change cleans up some references to tables that are no longer applicable, while retaining prescriptive guidance in the code for conventional wall bracing methods. For fiberboard sheathing attachment, Section 2306.6 and Table 2306.6 are no longer applicable. In the 2012 IBC, Table 2304.9.1 would be an appropriate reference for fastener size for attachment of fiberboard sheathing. Table 2306.7 is no longer the correct reference in the 2012 IBC for gypsum wallboard attachment. The appropriate fastener, 5d cooler nails, is proposed for consistency with Table 2308.12.4 which addresses nail size for gypsum wallboard bracing used in Seismic Design Category D and E.

S288-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2308.9.3-S-COATS.doc

S289–12 2308.9.3

Proponent: Michael Gardner, Gypsum Association (mgardner@gypsum.org)

Revise as follows:

2308.9.3 Bracing. Braced wall lines shall consist of braced wall panels that meet the requirements for location, type and amount of bracing as shown in Figure 2308.9.3, specified in Table 2308.9.3(1) and are in line or offset from each other by not more than 4 feet (1219 mm). Braced wall panels shall start not more than 121/2 feet (3810 mm) from each end of a braced wall line. Braced wall panels shall be clearly indicated on the plans. Construction of braced wall panels shall be by one of the following methods:

- 1. Nominal 1-inch by 4-inch (25 mm by 102 mm) continuous diagonal braces let into top and bottom plates and intervening studs, placed at an angle not more than 60 degrees (1.0 rad) or less than 45 degrees (0.79 rad) from the horizontal and attached to the framing in conformance with Table 2304.9.1.
- 2. Wood boards of 5/8 inch (15.9 mm) net minimum thickness applied diagonally on studs spaced not over 24 inches (610 mm) o.c.
- 3. Wood structural panel sheathing with a thickness not less than 3/8 inch (9.5 mm) for 16-inch (406 mm) or 24-inch (610 mm) stud spacing in accordance with Tables 2308.9.3(2) and 2308.9.3(3).
- 4. Fiberboard sheathing panels not less than 1/2 inch (12.7 mm) thick applied vertically or horizontally on studs spaced not over 16 inches (406 mm) o.c. where installed with fasteners in accordance with Section 2306.6 and Table 2306.6.
- 5. Gypsum board [sheathing 1/2-inch-thick (12.7 mm) or 5/8-inch-thick (15.9 mm) by 4-feet-wide (1219 mm) wallboard or veneer base] on studs spaced not over 24 inches (610 mm) o.c. and nailed fastened to studs at 7 inches (178 mm) o.c. with nails as required by Table 2306.7. or screws. Nails or screws shall be installed in the field of the board and at board edges. Nails and screws shall comply with Section 2506.2. Nails shall be annular ringed and not less than 1 ½ inches in length. Screws shall be not less than 1 ¼ inches in length.
- 6. Particleboard wall sheathing panels where installed in accordance with Table 2308.9.3(4).
- 7. Portland cement plaster on studs spaced 16 inches (406 mm) o.c.installed in accordance with Section 2510.
- 8. Hardboard panel siding where installed in accordance with Section 2303.1.6 and Table 2308.9.3(5).

For cripple wall bracing, see Section 2308.9.4.1. For Methods 2, 3, 4, 6, 7 and 8, each panel must be at least 48 inches (1219 mm) in length, covering three stud spaces where studs are spaced 16 inches (406 mm) apart and covering two stud spaces where studs are spaced 24 inches (610 mm) apart.

For Method 5, each panel must be at least 96 inches (2438 mm) in length where applied to one face of a panel and 48 inches (1219 mm) where applied to both faces. All vertical joints of panel sheathing shall occur over studs and adjacent panel joints shall be nailed to common framing members. Horizontal joints shall occur over blocking or other framing equal in size to the studding except where waived by the installation requirements for the specific sheathing materials. Sole plates shall be nailed to the floor framing and top plates shall be connected to the framing above in accordance with Section 2308.3.2. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the braced wall panels.

Reason: The proposal adds screws as an acceptable method of panel attachment when gypsum board is used as bracing. It also adds 5/8-inch-thick gypsum board to the list of materials used for bracing in structures constructed to the IBC.

The ability to use screws for the attachment of gypsum board used as bracing was inserted into the International Residential Code by the approval of Public Comment 2 on Proposal RB143 – 07/08. For consistency, similar language should be inserted into the IBC.

The addition of 5/8-inch-thick gypsum board to the text reflects the use of the thicker, when compared to ½ thick gypsum board, material commonly installed in structures constructed to the IBC. The bracing capability of the thicker material is greater than that of the thinner material, so the addition of the reference will not diminish the bracing attributes of the structure.

The reference to Section 2506.2 establishes that the nail or screw must comply with the minimum head size and shank diameter requirements in the appropriate standard in Table 2506.2. The standards referenced in Table 2506.2 are the same standards referenced in the IRC.

The nail and screw length minimum contained in the proposal establishes a fastener length that is no less than the length of the equivalent fastener as prescribed by the IRC for installation of gypsum board used as bracing. Because Table R702.3.5. of the IRC lists four potential nail types, the language requiring the use of an annular ringed nail – a common drywall nail – is inserted. Fastener lengths also reflect the minimum fastener length requirements contained in GA-216, Application and Finishing of Gypsum Board that is referenced in Section 2508 of the IBC.

While the literature presented as substantiation for the IRC modification indicated that a broader spacing for screws, when compared to nails, is justified, it is recommended that, for simplicity and consistency of installation, a one-for-one swap of screws for nails reflecting the current spacing contained in the text is more appropriate. The spacing in this proposal is identical to the spacing presently contained in Table R602.10.4 of the IRC.

S289-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2308.9.3-S-GARDNER.doc

S290–12 2308.9.5.1, Figure 2308.9.5.1.1(1) (NEW), Figure 2308.9.5.1.1(2), 2308.9.5.1.1 (NEW)

Proponent: Paul Coats, American Wood Council (pcoats@awc.org)

Revise as follows:

2308.9.5.1 Headers. Headers shall be provided over each opening in exterior-bearing walls. The spans in Table 2308.9.5 are permitted to be used for one- and two-family *dwellings*. Headers for other buildings shall be designed in accordance with Section 2301.2, Item 1 or 2. Headers shall be of two or more pieces of nominal 2-inch (51 mm) framing lumber shall be set on edge as permitted by Table 2308.9.5 and nailed together in accordance with Table 2304.9.1 or of solid lumber of equivalent size.

2308.9.5.1.1 Single member headers. Single member headers shall be permitted when attached to a single flat 2-inch-nominal (51 mm) member or wall plate not less in width than the wall studs on the top and bottom of the header in accordance with Figures 2308.9.5.1.1(1) and 2308.9.5.1.1(2). Single-ply headers shall be designed in accordance with Section 2301.2, Item 1 or 2.



JACK STUD

FIGURE 2308.9.5.1.1(1) SINGLE MEMBER HEADER IN EXTERIOR BEARING WALL



FIGURE 2308.9.5.1.1(2) ALTERNATIVE SINGLE MEMBER HEADER WITHOUT CRIPPLE

Reason: The single ply header option is added for consistency with similar construction provisions for single ply header in the IRC. The figure illustrates recommended use of 2x material flat-wise as a method to provide resistance to out of plane wind loads as well as to brace the less stable single ply header when compared to a typical 2-ply header. Allowable spans for single ply headers are tabulated in the WFCM.

S290-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					2308 9 5 1-S-COATS doc

S291–12 2308.9.3.2, Figure 2308.9.3.2

Proponent: Edward L. Keith, P.E., APA – The Engineered Wood Association (ed.keith@apawood.org)

Revise as follows:

2308.9.3.2 Alternate bracing wall panel adjacent to a door or window opening. Any bracing required by Section 2308.9.3 is permitted to be replaced by the following when used adjacent to a door or window opening with a full-length header:

1. In one-story buildings, each panel shall have a length of not less than 16 inches (406 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with a single layer of 3/8 inch (9.5 mm) minimum thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Figure 2308.9.3.2. The wood structural panel sheathing shall extend up over the solid sawn or glued-laminated header and shall be nailed in accordance with Figure 2308.9.3.2. A built-up header consisting of at least two 2 × 12s and fastened in accordance with Item 24 of Table 2304.9.1 shall be permitted to be used. A spacer, if used, shall be placed on the side of the built-up beam opposite the wood structural panel sheathing. The header shall extend between the inside faces of the first full-length outer studs of each panel. The clear span of the header between the inner studs of each panel shall be not less than 6 feet (1829 mm) and not more than 18 feet (5486 mm) in length. A strap with an uplift capacity of not less than 1,000 pounds (4,400 N) shall fasten the header to the inner studs opposite the sheathing. One anchor bolt not less than 5/8 inch (15.9 mm) diameter and installed in accordance with Section 2308.6 shall be provided in the center of ach sill plate. The studs at each end of the panel shall have a tie-down device fastened to the foundation with an uplift capacity of not less than 4,200 3,500 pounds (18 480 15 570 N).

Where a panel is located on one side of the opening, the header shall extend between the inside face of the first full-length stud of the panel and the bearing studs at the other end of the opening. A strap with an uplift capacity of not less than 1,000 pounds (4400 N) shall fasten the header to the bearing studs. The bearing studs shall also have a tie-down device fastened to the foundation with an uplift capacity of not less than 1,000 pounds (4400 N).

The tie-down devices shall be an embedded strap type, installed in accordance with the manufacturer's recommendations. The panels shall be supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom.

Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped not less than 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

2. In the first *story* of two-story buildings, each wall panel shall be braced in accordance with Item 1 bove, except that each panel shall have a length of not less than 24 inches (610 mm).



FIGURE 2308.9.3.2

ALTERNATE BRACED WALL PANEL ADJACENT TO A DOOR OR WINDOW OPENING

Reason: 1) There are a couple of types of changes to Figure 2308.9.3.2 proposed. There are both technical changes and editorial changes.

<u>Technical changes:</u> The two technical changes made to the figure are the reduction of the capacity of the portal frame leg tiedown devices from 4200 lbf to 3500 lbf and the removal of the third bottom plate at the portal frame leg. (Note that the third bottom plate we propose to delete is NOT shown in the figure above. The normal strikethrough and underline procedures are difficult to apply to figure changes.)

A. The first technical change is the reduction of the tie-down from 4200 lbf to 3500 lbf. The initial testing was conducted on the portal frames utilizing the 4200 lbf hold down because that was what was readily available and in common use by the construction industry. At the time of initial testing, no attempt was made to determine the sensitivity of the system to such a reduction in tie-down capacity. As the initial prescriptive parameters of the portal frame were based on testing, there was no latitude for determining the impact of the industry wide reduction to such tie-downs in response to the cracked-concrete provisions of ACI 318. As such, retesting of the portal frames with both 4200 lbf and 3500 lbf tie-downs was necessary to determine the impact on the performance of the system, if any. Portals with 16" wide legs x 8 ft height as well as 24" wide x 10 ft high were recently retested by APA. Pairs of each size were tested with 4200 lbf tie-downs and then retested with 3500 lbf tie-downs. The results of these tests showed that the system was relatively insensitive to the reduction in tie-down capacity from 4200 lbf to 3500 lbf. No attempt was made to determine how low the tie-down capacity could be reduced before an impact on the performance of the portal frames could be seen.

These tests were conducted using the CUREe method, as described in ASTM E2126, with a frequency of 0.5 Hz. The following charts show the backbone curves for the Method PFH portal frames tested with 3500 lbf and 4200 lbf tie-downs at both the 16" wide leg portals 8' high as well as the 24" wide portals 10' high.







Free PDF Copies of the full lab report on this testing program entitled APA Report T2011-15, *Bracing Method PFH (Portal Frame with Hold down) – Alternative Attachment,* can be obtained at http://www.apawood.org.

- B. The second technical change is the removal of the third bottom plate. As mentioned above the original testing was conducted with the third plate in place. The third plate causes numerous difficulties in the field, not the least of which is that the normal length threaded anchors are too short to accommodate the third plate and provide the required depth of penetration into the foundation. This results in inadequate anchor depth-of-embedment or the use of threaded sleeves and all-thread to extend the bolt length to accommodate the third plate. When investigating the change to the 3500 lbf hold down, we utilized this opportunity to run the tests with only double bottom plates. All subsequent testing was done without the third bottom plate. The results of this testing indicated that the third bottom plate has negligible impact on the performance of the portal frames.
- Non-technical changes:
 - 1. The intent of the note concerning the location of the portal-leg sheathing-splice, when present, is to place the splice butt joint within the middle 24" of the portal frame height. As currently written "within 24" of mid height" means the splice could be placed within 24 inches either above or below of mid height, or within a band 48" wide. This was never the intent. The proposed language is clearer that the joint must "occur within the middle 24" of portal height", where portal height is illustrated in the figure.
 - 2. At the splice plate, the current wording requires a single row of nailing. The proposed change required this at <u>each panel</u> edge at the splice as was the original intent.
 - 3. In the same annotation, a provision is provided that would permit the splice to be made over a pair of 2x4s as long as they are spliced together. The proposal changes "blocking" to "double blocking" to clarify the intent.

2) The revision to Section 2308.9.3.2 is as explained above.

S291-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				F2308.9.3.2-S-KEITH.doc

S292-12 2308.11.3.3

Proponent: Robert Rice, C.B.O., Josephine County, OR, representing Oregon Building Officials Association (structdesigner@yahoo.com), R. Terry Malone, P.E., S.E.

Revise as follows:

2308.11.3.3 Openings in horizontal diaphragms. Horizontal diaphragms with openings having dimension perpendicular to the joist that is greater than 4 feet (1219 mm) shall be designed in accordance with accepted engineering practice. Openings in horizontal diaphragms with a dimension perpendicular to the joist that is <u>not</u> greater than 4 feet (1219 mm) shall be constructed in accordance with the following: with metal ties and blocking in accordance with this section and Figure 2308.11.3.3.

- 1. Blocking shall be provided beyond headers.
- 2. Metal ties <u>shall</u> not <u>be</u> less than 0.058 inch <u>thick</u> [1.47 mm (16 galvanized gage)] by 1_{1/2} inches (38 mm) wide <u>and shall have a minimum yield strength of 33,000 psi (227 MPa)</u>. <u>Blocking shall extend 2 feet minimum beyond headers</u>. <u>Ties shall be attached to blocking</u> with eight 16d common nails on each side of the header-joist intersection shall be provided (see Figure 2308.11.3.3). The metal ties shall have a minimum yield of 33,000 psi (227 MPa).

Reason: This proposal re-arranges the existing text to read more clearly, corrects an error in the code and clarifies the requirements and limitations of openings in diaphragms in structures assigned to Seismic Design Category B, C, D and E. The text of the current code is intended to provide a prescriptive solution for diaphragm openings, in high seismic design categories, that are 4 feet or less. The current code is missing the word "not" which would make this section correct. The commentary for this code section correctly states,

Horizontal diaphragms are floor and roof assemblies that are usually clad with structural wood sheathing panels, such as plywood or OSB. Though more complicated and difficult to visualize, lateral forces that are applied to a building from wind or seismic events follow a load path that distributes and transfers shear and overturning forces from the lateral loads. When openings are built into the diaphragm, they disrupt the continuity of load across the diaphragm and they must be reinforced to compensate. Another concern is the stiffness of the diaphragm. These provisions are a prescriptive solution for openings not greater than 4 feet (1219 mm) in dimension and provide a general means for a load path in these specific cases in lieu of an engineered design.- 2009 IBC Commentary, International Code Council

S292-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2308.11.3.3-S-RICE.doc

S293–12 Figure 2308.11.3.3

Proponent: Edward L. Keith, P.E., APA – The Engineered Wood Association (ed.keith@apawood.org)

Revise as follows:



Reason: This is one of the last remaining references to "plywood" in the code that should have been converted to the more generic "wood structural panel" (WSP) in the 2000 first printing of the IBC. In terms of structural capacity, the IBC makes no distinction to the type of wood structural panel sheathing used. In addition, the type of floor sheathing is inconsequence to the subject of the figure, which relates to floor framing. We request approval of the code change proposal for the sake of consistency in the IBC.

S293-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	2				F2308.11.3.3-S-KEITH.doc

S294–12 1504.2.1.1 (NEW), 1504.2.1.2 (NEW), 1711.1, 1711.1.1, 1711.1.2, 1711.1.2.1, 1711.1.3, 1711.2, 1711.2.1, 1711.2.2, 2309 (NEW)

Proponent: D. Kirk Harman, P.E., S.E., SECB, FACI, The Harman Group, Inc., National Council of Structural Engineering Association

Revise as follows:

SECTION 1711 MATERIAL AND TEST STANDARDS

SECTION 2409 JOIST HANGERS

1711.1 <u>2309</u> Joist hangers. Testing of joist hangers shall be in accordance with Sections 1711.1.1 through 1711.1.3, as applicable.

1711.1.1 <u>2309.1</u> General. 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).

1711.1.2 <u>2309.2</u> Vertical load capacity for joist hangers. The vertical load-bearing 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-bearing 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).
- 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.

1711.1.2.1 <u>2309.2.1</u> **Design value modifications for joist hangers.** Allowable design values for joist hangers that are determined by Item 4 or 5 in Section 1711.1.2 shall be permitted to be modified by the appropriate load duration factors as specified in AF&PA NDS but shall not exceed the direct loads as determined by Item 1, 2 or 3 in Section 1711.1.2. Allowable design values determined by Item 1, 2 or 3 in Section 1711.1.2. Allowable design values determined by Item 1, 2 or 3 in Section 1711.1.2.

1711.1.3 2309.3 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).

1711.2 Concrete and clay roof tiles **1504.2.1** Testing. Testing of concrete and clay roof tiles shall be in accordance with Sections 1711.2.1 and 1711.2.2, as applicable. this section.

1711.2.1 1504.2.1.1 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.

1711.2.2 <u>**1504.2.1.2**</u> **Wind tunnel testing.** Where concrete and clay 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: Chapter 17 is titled "Special Inspections and Tests" and as such, is intended to be primarily reserved for the special inspection and testing associated with the actual construction work. NCSEA holds the opinion that material compliance testing for joist hangers belongs in Chapter 23 as this testing is not associated with the actual construction work. Similarly, wind tunnel testing to determine overturning resistance of roof tiles belongs in Chapter 15 as these tests are also not associated with the actual construction work and an existing section dealing with wind resistance for concrete and clay roof tiles currently exists as section 1502.1. Current Section 1711 is comprised solely of the two sections proposed to be relocated, and can therefore be deleted subsequent to the relocations.

S294-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2309 (NEW)-S-HARMAN.doc
S295-12 2404.1, 2404.2, 2404.3.1, 2404.3.2, 2404.3.3, 2404.3.4, 2404.3.5, 2405.5.2

Proponent: Julie Ruth, JRuth Code Consulting, representing American Architectural Manufacturers Association (julruth@aol.com)

Revise as follows:

2404.1 Vertical glass. Glass sloped 15 degrees (0.26 rad) or less from vertical in windows, curtain and window walls, doors and other exterior applications shall be designed to resist the wind loads due to ultimate design wind speed V_{ult} in Section 1609 for components and cladding. Glass in glazed curtain walls, glazed storefronts and glazed partitions shall meet the seismic requirements of ASCE 7, Section 13.5.9. The load resistance of glass under uniform load shall be determined in accordance with ASTM E 1300.

The design of vertical glazing shall be based on the following equation:

$$0.6F_{gw} \leq F_{ga}$$

where:

 F_{aw} = Wind load on the glass <u>due to ultimate design wind speed V_{ult}</u> computed in accordance with Section 1609.

 F_{aa} = Short duration load on the glass as determined in accordance with ASTM E 1300.

2404.2 Sloped glass. Glass sloped more than 15 degrees (0.26 rad) from vertical in skylights, sunrooms, sloped roofs and other exterior applications shall be designed to resist the most critical of the following combinations of loads.

F_q	=	<u>0.6</u> Wo – D
F_q	=	$\overline{0.6}W_i + D + 0.5 S$
F_{g}	=	$\frac{0.5}{0.3}W_i + D + S$

where:

D = Glass dead load psf (kN/m2).

For glass sloped 30 degrees (0.52 rad) or less from horizontal,

13 *t_a* (For SI: 0.0245 *tg*).

For glass sloped more than 30 degrees (0.52 rad) from horizontal,

- = 13 $t_a \cos \theta$ (For SI: 0.0245 $tg \cos \theta$).
- F_g = Total load, psf (kN/m2) on glass. S = Snow load, psf (kN/m2) as determined in Section 1608.
- t_g W_i = Total glass thickness, inches (mm) of glass panes and plies.
- = Inward wind force, psf (kN/m2) due to ultimate design wind speed V_{ult} as calculated in Section 1609.
- W_o = Outward wind force, psf (kN/m2) due to ultimate design wind speed V_{ult}, as calculated in Section 1609.
- θ = Angle of slope from horizontal.

Exception: Unit skylights shall be designed in accordance with Section 2405.5.

The design of sloped glazing shall be based on the following equation:

 $F_a \leq F_{aa}$

(Equation 24-5)

(Equation 24-3)

(Equation 24-1)

(Equation 24-2)

(Equation 24-4)

For Equations 24-2 and 24-3:

For Equation 24-4:

 $F_{a} < 0.5 F_{ae}$

where.

 F_{qe} = Nonfactored load from ASTM E 1300.

2404.3.3 Vertical patterned glass. Patterned glass sloped 15 degrees (0.26 rad) or less from vertical in windows, curtain and window walls, doors and other exterior applications shall be designed to resist the wind loads in Section 1609 for components and cladding according to the following equation:

 $F_{qw} < 1.0 F_{qe}$

where:

 F_{gw} = Wind load on the glass <u>due to ultimate design wind speed V_{ult}</u> computed per Section 1609. F_{ge} = Nonfactored load from ASTM E 1300. The value for patterned glass shall be based on the thinnest part of the glass. Interpolation between nonfactored load charts in ASTM E 1300 shall be permitted.

2404.3.4 Sloped patterned glass. Patterned glass sloped more than 15 degrees (0.26 rad) from vertical in skylights, sunspaces, sloped roofs and other exterior applications shall be designed to resist the most critical of the combinations of loads from Section 2404.2.

For Equations 24-2 and 24-3:

 $F_q < 1.0 F_{qe}$

(Equation 24-10)

 $F_{a} < 0.3 F_{ae}$

 F_g = Total load on the glass, as determined by equations 24-2, 24-3 or 24-4.

Short duration load resistance of the glass as determined according to ASTM E 1300 for

windows, curtain and window walls, doors and other exterior applications shall be designed to resist the

where:

 $0.6F_{aw} < 0.5F_{ae}$

where:

 $F_g =$

- F_{gw} = Is the wind load on the glass <u>due to ultimate design wind speed V_{ult_2} computed per Section 1609.</u>
- F_{qe} = Nonfactored load from ASTM E 1300 using a thickness designation for monolithic glass that is not greater than the thickness of wired glass.

2404.3.2 Sloped wired glass. Wired glass sloped more than 15 degrees (0.26 rad) from vertical in skylights, sunspaces, sloped roofs and other exterior applications shall be designed to resist the most

2404.3.1 Vertical wired glass. Wired glass sloped 15 degrees (0.26 rad) or less from vertical in

wind loads in Section 1609 for components and cladding according to the following equation:

Total load on the glass determined from the load combinations above.

(Equation 24-9)

(Equation 24-7)

(Equation 24-6)

(Equation 24-8)

For Equation 24-4:

 $F_{q} < 0.6F_{qe}$

Where

- F_g = Total load on the glass, as determined by equations 24-2, 24-3 or 24-4.
- F_{ae} = Nonfactored load from ASTM E 1300. The value for patterned glass shall be based on the thinnest part of the glass. Interpolation between the nonfactored load charts in ASTM E 1300 shall be permitted.

2404.3.5 Vertical sandblasted glass. Sandblasted glass sloped 15 degrees (0.26 rad) or less from vertical in windows, curtain and window walls, doors, and other exterior applications shall be designed to resist the wind loads in Section 1609 for components and cladding according to the following equation:

where:

- $F_{g} F_{gw}$ = Total Wind load on the glass due to ultimate design wind speed V_{ult} computed per Section 1609.
- = Nonfactored load from ASTM E 1300. The value for sandblasted glass is for moderate levels F_{qe} Of sandblasting.

2405.5.2 Unit skylights rated for separate performance grades for positive and negative design pressure. The design of unit skylights rated for performance grade for both positive and negative design pressures shall be based on the following equations:

 $\begin{array}{l} F_{gi} \leq PG_{Po} \\ F_{go} \leq PG_{Ne} \end{array}$

where:

 PG_{Pos} = Performance grade rating of the skylight under positive design pressure;

 PG_{Neg} = Performance grade rating of the skylight under negative design pressure; and

 F_{gi} and F_{go} are determined in accordance with the following:

For $0.6W_o \ge D$,

where:

- W_o = Outward wind force, psf (kN/m2) <u>due to ultimate design wind speed V_{ult}</u> as calculated in Section 1609.
- D = The dead weight of the glazing, psf (kN/m2) as determined in Section 2404.2 for glass, or by the weight of the plastic, psf (kN/m2) for plastic glazing.
- F_{gi} = Maximum load on the skylight determined from Equations 24-3 and 24-4 in Section 2404.2.
- F_{ao} = Maximum load on the skylight determined from Equation 24-2.

For $\underline{0.6}W_o < D$, where:

- W_{o} = Is the outward wind force, psf (kN/m2) <u>due to ultimate design wind speed V_{ult} as calculated in</u> Section 1609.
- D = The dead weight of the glazing, psf (kN/m2) as determined in Section 2404.2 for glass, or by the weight of the plastic for plastic glazing.

(Equation 24-11)

(Equation 24-14) (Equation 24-15)

(Equation 24-12)

F_{gi} = Maximum load on the skylight determined from Equations 24-2 through 24-4 in Section 2404.2. F_{go} = 0.

Reason: The purpose of this proposal is to coordinate the glass design load equations of Chapter 24 with those of Chapter 16. The design load equations of Chapter 16 of the 2012 IBC were revised as appropriate to respond to the change of design wind load model from Allowable Stress Design to Strength Design in ASCE 7-10. These revisions, however, were not carried back to the

glass design load equations of Chapter 24.

This proposal corrects this previous omission.

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

S295-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	2				2404.1-S-RUTH.doc

S296–12 2406.4.5

Proponent: Jennifer Hatfield, J. Hatfield & Associates, PL, representing Association of Pool & Spa Professions (jen@jhatfieldandassociates.com)

Revise as follows:

2406.4.5 Glazing and wet surfaces. Glazing in walls, enclosures or fences containing or facing hot tubs, spas, whirlpools aquatic vessels, saunas, steam rooms, bathtubs, and 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 or aquatic vessel.

Reason: The new International Swimming Pool & Spa Code (ISPSC) utilizes a new definition to encompass all different types of pools, hot tubs, and spas – aquatic vessel. This proposal utilizes the new terminology found in the ISPSC for consistency between the I-codes.

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

S296-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				2406.4.5-S-HATFIELD.doc

S297–12 2406.4.7

Proponent: Tim Pate, City & County of Broomfield Building Division, representing Colorado Chapter Code Change Committee

Revise as follows:

2406.4.7 Glazing adjacent to the bottom stair landing. Glazing adjacent to the landing at the bottom of a stairway where the glazing is less than 36 inches (914 mm) <u>60 inches (1524 mm)</u> above the landing and within 60 inches (1524 mm) horizontally of the bottom tread shall be considered a hazardous location.

Exception: Glazing that is protected by a guard complying with Sections 1013 and 1607.8 where the plane of the glass is greater than 18 inches (457 mm) from the guard.

Reason: Previous editions of the IBC before the 2012 required glazing that is less than 60" above the landing to be approved safety glazing. It is not clear why this requirement was changed in the 2012. It does not make sense that section 2406.4.6 applies to glazing that is less than 60" above the stairs and intermediate landings but the glazing at bottom landing is treated differently – only when below 36" The potential for falling through the glazing at bottom landing is the same. This change will bring back the 60" height which will then match the requirement at intermediate landings and stairs.

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

S297-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2406.4#1-S-PATE.doc

S298–12 2406.4.7

Proponent: Tim Pate, City & County of Broomfield Building Division, representing self

Revise as follows:

2406.4.7 Glazing adjacent to the bottom stair landing. Glazing adjacent to the landing at the bottom of a stairway where the glazing is less than 36 inches (914 mm) above the landing and within <u>a</u> 60 inches (1524 mm) horizontally of <u>arc less than 180 degrees from</u> the bottom tread shall be considered a hazardous location.

Exception: Glazing that is protected by a guard complying with Sections 1013 and 1607.8 where the plane of the glass is greater than 18 inches (457 mm) from the guard.

Reason: Previous editions of the IBC before the 2012 required glazing that is 60" horizontally in any direction to be approved safety glazing. It is not clear why this requirement was changed in the 2012. The previous editions had the additional wording "in any direction" when applying the 60" horizontal rule. This is due to the "splay" factor for when someone gets to the last tread and falls. The tendency is for someone to flail out in any direction. This added wording will make this section apply to any glazing that is in a wall that is less than 180 degrees from the bottom tread. This will make it very clear what the intent was and still is with this section.

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

S298-12					
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	•				2406.4.7#2-S-PATE.doc

S299–12 2407.1.2

Proponent: Anthony Leto, The Wagner Companies

Revise as follows:

2407.1.2 Support. Each handrail or *guard* section shall be supported by a minimum of three glass balusters or shall be otherwise supported to remain in place should one baluster panel fail. Glass balusters shall not be installed without an attached handrail or *guard* top rail.

Exception: A top rail shall not be required where the glass balusters are laminated glass with two or more glass plies of equal thickness and the same glass type when *approved* by the *building official*. The panels shall be designed to withstand the loads specified in Section 1607.8.

Reason: While the ICC opinion on top railing requirements for monolithic glass baluster guards has remained consistent, we continue to see installations without the required top rail. Where is the disconnect?

The confusion begins with IBC Section 2407.1.1.2 Support. There are three issues:

- The term guard is used improperly at the end of the second sentence. The ICC defines guard as being in place to stop accidental falls and refers to the full assembly not the guard top. The word guard should be replaced with the words top rail as is noted in the Exception.
- 2. In a glass baluster handrail, the handrail and top rail are the same component.

A glass baluster being used only as a handrail (i.e. a stair where there is less than a 30 inch drop from the top step) will require a handrail which must meet the dimensional and clearance requirements for handrail. It should be noted, that under the strict definition of handrail clearance, a handrail placed directly on top of a glass baluster does not meet code as the glass would be considered a 100% obstruction. The handrail would need to be attached to the glass baluster with brackets to provide code compliant clearance. The handrail would be the top most portion of the assembly, therefore the handrail would also serve as the top rail.

3. **Misinterpretation of the phrase**, *Glass balusters shall not be installed without an attached handrail or guard*. Handrail is required on stairs and is located 34 to 38 inches above the stair nosing. A guard is required when there is a 30 inch drop. The IBC minimum for a guard is 42 inches above the walking surace. If a stair has a drop of greater than 30 inches, it would be required to have both a handrail and a guard. However, if the stair height does not exceed 30 inches, only a handrail is required.

There are some who interpret that Section 2407.1.1.2 allows a glass baluster guard to be installed with either a handrail or a guard (top rail).

However, the section's intention is that a glass baluster *handrail* must have an attached *handrail* and that a glass baluster *guard* must have an attached *guard* (top rail). **The presence of a** *handrail* **on a** *guard* **does not eliminate the need for a top railing. This interpretation is supported by:**

A. The ICC

In 2008, Todd Daniel of the National Ornamental and Miscellaneous Metals Association (NOMMA) asked the following question of the International Code Council (ICC):

Can a glass rail system be installed without a guard on top of the glass IF there is a handrail attached to the glass. In other words...no cap, exposed top edge of glass at 42 inch height with a handrail mounted on the side of the glass at handrail heights.

ICC Staff Opinion: No

Reason: The application you describe can only be allowed if the glass can withstand the loads for guards and handrails in Section 1607.7

B. The 2009 IBC Exception

The ICC approved an exception in 2009 that a top railing was not required if laminated glass is used that meets the load requirements and is approved by the building official. If this is the exception to the rule, then it should be understood that a top railing is required in all other situations.

C. The Load Requirements

Section 2407.1.1 requires that glass baluster handrails and guards must meet the load requirements of 1607.7 with a safety factor of four.

In a required guard, the loads must be applied to the *top* of the guard -- not the top of the handrail. Having a 42 inch guard with an attached handrail at between 34 and 38 inches will not meet the load requirement unless it is laminated tempered glass or the monolithic, tempered glass is of significant thickness.

Standard 1/2 inch monolithic, tempered glass edges are highly susceptable to rupture under load. Directing an 800 pound concentrated load (200 lbs multiplied by a safety factor of four) to that bare edge will most likely result in failure.

In 2011, there were numerous cases of glass railing failures across the US and Canada. An article relating these failures was published this past October by US Glass Magazine (http://www.usglassmag.com/digital/2011/Oct2011.pdf). While most cases were likely the result of nickel sulfide inclusions in the glass, the consulting engineering firm brought in to determine the reasons for failure of glass railings at the W Hotel in Austin, TX noted that in one event, the failure was related to debris from above striking a bare edge of a glass panel.



Stair with required guard and attached handrail.



Required handrail for stair when a guard is not required.



Guard with top railing



Guard without top railing. Per ICC staff opinion, permitted only when used with laminated, tempered glass or if the glass meets the structural requirements of 1607.7



Guard with non-required handrail -- handrail is in place in an attempt to meet the requirements of an attached handrail or guard. However, the requirement is that the guard be able to withstand the load at the top of the guard. The handrail is not the top of the guard therefore the load must be met by the top edge of glass -- by a safety factor of four.

Cost Impact: There should be no cost impact since this change is to clarify and eliminate misinterpretation whereby glass railings are being installed without a top rail. In reality there will be long term savings as there are now situations where, as part of due diligence during a building purchase, consulting engineers are pointing out that glass rails without a top rail are not code compliant. Building owners in turn are requiring engineers/architects of record to have the railing redesigned to be code compliant.

S299-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
-				2407.1.2-S-LETO.doc

S300–12 2407.1, 2407.1.1

Proponent: Thomas S. Zaremba, Roetzel & Andress, representing Glazing Industry Code Committee (tzaremba@ralaw.com)

Revise as follows:

2407.1 Materials. Glass used as in a handrail assembly <u>, guardrail</u> or a *guard* section shall be <u>laminated</u> <u>glass</u> constructed of either single fully tempered glass, laminated fully tempered glass or laminated heatstrengthened glass <u>and shall comply with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI</u> <u>Z97.1</u>. 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 Part 1201 or Class A of ANSI Z97.1.

Exception: Single fully tempered glass complying with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1 may be used in handrails and guardrails if there is no walking surface beneath them or the walking surface is permanently protected from the risk of falling glass.

2407.1.1 Loads. The panels and their support system shall be designed to withstand the loads specified in Section 1607.8. A safety design factor of four shall be used for safety.

Reason: Several recent incidents involving spontaneous breakage of fully tempered glass in handrail or guardrail systems on high rise balconies has prompted the Glazing Industry Code Committee to seek this change which, if adopted, will make mandatory the use of the retentive characteristics of laminated glass in these applications unless there is no walking surface below or it is permanently protected from falling glass, in which case, fully tempered glass meeting the safety criteria of Cat. II of CPSC 16 CFR 1201 or Class A of ANSI Z97.1 would be permitted. Additionally, the proposal adds the term "guardrail" to section 2407.1 since that term is also used in various locations throughout the I-codes in connection with these types of systems.

Finally, proposal changes Section 2407.1.1 are intended to make it clear that a "design" factor of four is required "for safety." The intent of this section is to use a "design" factor of four when determining the loads of these panels and their support systems. Using the word "safety" in the way it is currently found in this section is ambiguous and may or may not achieve the section's intended purpose.

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

S300-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
2				2407.1-S-ZAREMBA.doc

S301–12 2409, 2409.1 (NEW), Chapter 35 (NEW)

Proponent: Thomas S. Zaremba, Roetzel & Andress, representing Glazing Industry Code Committee (tzaremba@ralaw.com)

Revise as follows:

SECTION 2409 GLASS IN <u>WALKWAYS,</u> ELEVATOR HOISTWAYS AND ELEVATOR CARS

2409.1 Glass walkways. Glass installed as a part of a floor/ceiling assembly as a walking surface and constructed with laminated glass shall comply with ASTM E 2751-11, otherwise it shall comply with the load requirements specified in Chapter 16. Such assemblies shall also comply with the fire-resistance rating requirements of this code where applicable.

Add new standard to Chapter 35 as follows:

ASTM

E 2751 Standard Practice for Design and Performance of Supported Glass Walkways

Reason:: In the development cycle leading to the 2006 IBC, the Glazing Industry Code Committee ("GICC") asked this body to delete the glass walkway provisions found in Chapter 24 of the 2003 IBC. The reason for its request was that the glass walkway provisions found in the 2003 IBC used load requirements derived from ASTM E1300 and glass walkways are not within the scope of ASTM E1300. As a result, the glass walkway provisions of Chapter 24 were deleted from the 2006 IBC.

Since then, ASTM E2751-11 has been issued and specifically addresses load-bearing glass walkways constructed of laminated glass. If adopted, this new section 2409.1 would apply to glass walkways constructed of laminated glass, otherwise the load requirements of Chapter 16 would apply to glass walkways constructed of non-laminated glass, for example, walkways constructed using glass block.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S301-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					2409.1 (NEW)-S-ZAREMBA.doc

S302–12 202 (NEW), 1710.6, 2410 (NEW), 2410.1 (NEW), 2410.2 (NEW)

Proponent: Timothy Burgos, InterCode Incorporated, representing 3M Company

Add new text as follows:

SECTION 202 DEFINITIONS

SUNLIGHT DELIVERY SYSTEM (SDS). A unit primarily designed to transmit daylight from an exterior surface to an interior space via a reflective duct or conduit. The basic unit consists of an exterior solar collecting device, a daylight-transmitting duct or conduit with a reflective interior surface, and an interior-ceiling device such as a translucent ceiling panel. The unit can be factory assembled, or field-assembled from a manufactured kit.

Revise as follows:

1710.6 Skylights and sloped glazing, and sunlight delivery systems. Unit skylights and tubular daylighting devices (TDDs) shall comply with the requirements of Section 2405. <u>Sunlight delivery systems</u> (SDS's) and tubular daylighting devices (TDDs) shall comply with the requirements of Section 2410. All other skylights and sloped glazing shall comply with the requirements of Chapter 24.

Add new text as follows:

SECTION 2410 SUNLIGHT DELIVERY SYSTEMS AND TUBULAR DAYLIGHTING DEVICES

2410.1 General. Sunlight delivery systems and tubular daylighting devices shall comply with the requirements of this code and be installed per the manufacturer's specifications.

2410.2 Definition. The following terms are defined in Chapter 2:

SUNLIGHT DELIVERY SYSTEM.

TUBULAR DAYLIGHTING DEVICE.

Reason: The purpose of this proposed edit is to create a more expansive definition of the tubular daylighting device. While tubular daylighting devices are a common implementation of the principles of reflective daylighting, new advancements in the field are available worldwide and should be included in the next edition of the International Building Code. Having a more expansive definition in the International Building Code for sunlight delivery systems will open up new technologies that can introduce natural sunlight into the interior areas that do not have windows or natural light entering that room. A sunlight delivery system provides designers with a new method of daylighting that offers significantly greater capabilities than existing alternatives. Traditional daylighting methods, such as skylights or tubular daylighting devices, are limited. These systems can require multiple entry points and are often limited to top floor applications. An example of a sunlight delivery system can be found in the pictures at the end of this reason statement.

The widespread use of electrical lighting in the 20th century changed the design of buildings but often made it impossible to illuminate internal rooms with daylight, thus requiring the use of artificial light in internal spaces. The use of artificial light currently makes up as much as 45% of the energy use in commercial and industrial buildings and up to 35% in residential buildings.

Sunlight delivery systems can significantly reduce energy costs for illumination. In a paper presented to LuxEuropa in 2009 entitled *Hybrid Lighting systems: a feasibility study for Europe* by Mohammed S. Mayhoub and David Carter, energy savings ranging from 28% to 85% (in latitudes ranging from 60° North (Oslo, Norway) to 36° North (Khania, Greece) were reported when a variety of sunlight delivery systems were tested. These locations correlate to locations in the United States as follows: Oslo, Norway is similar in latitude to Juneau and Anchorage, AK and Khania, Greece is similar to Virginia Beach, VA; Las Vegas, NV; and Nashville, TN.

The study showed that the greatest savings were realized in the Southern most latitudes (in the Northern Hemisphere) but still showed the possibility that 50% savings could be realized at 60[°] North with the most advanced systems. Because the study was limited to European Union countries, no analysis was conducted in more southern latitudes similar to the southernmost portion of the United States where cities such as Tampa, San Antonio, and New Orleans are located. In fact, most of the land mass of the

contiguous 48 United States lies well below 50° North indicating that greater savings could be realized in the United States than those projected in Europe.

An abundance of research and knowledge shows not only that the preferred light source in buildings is natural daylight but also that lack of exposure to daylight can lead to biological issues, lack of productivity, higher levels of stress, sleep difficulties and a variety of other human response issues. Studies suggest that creating healthy indoor lighting by providing day-lighting and natural lighting cycles can be a simple form of preventative medicine and can lead to higher production and overall better mental and physical health for the inhabitants. The health benefits that a sunlight delivery device provides is one of the reasons for this code change to be approved.



Roof top solar collecting devices used in a sunlight delivery system.



Sunlight being delivered to the interior space in an open ceiling (on the left) and in a dropped ceiling (on the right) by way of a sunlight duct system.

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

S302-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				202-SDS-S-BURGOS

S303–12 202, 1710.6, 2410 (NEW), 2410.1 (NEW), 2410.2 (NEW)

Proponent: Timothy Burgos, InterCode Incorporated, representing 3M Company

Revise as follows:

SECTION 202 DEFINITIONS

TUBULAR DAYLIGHTING DEVICE (TDD) SUNLIGHT DELIVERY SYSTEM (SDS). A nonoperable fenestration unit primarily designed to transmit daylight from a roof exterior surface to an interior ceiling space via a tubular reflective duct or conduit. The basic unit consists of an exterior glazed weathering surface, solar collecting device a daylight-transmitting tube duct or conduit with a reflective interior surface, and an interior-sealing ceiling device such as a translucent ceiling panel. The unit can be factory assembled, or field assembled from a manufactured kit.

Revise as follows:

1710.6 Skylights and sloped glazing, and sunlight delivery systems. Unit skylights and tubular daylighting devices (TDDs) shall comply with the requirements of Section 2405. Sunlight delivery systems (SDS's) shall comply with the requirements of Section 2410. All other skylights and sloped glazing shall comply with the requirements of Chapter 24.

Add new text as follows:

SECTION 2410 SUNLIGHT DELIVERY SYSTEMS AND TUBULAR DAYLIGHTING DEVICES

2410.1 General. Sunlight delivery systems and tubular daylighting devices shall comply with the requirements of this code and be installed per the manufacturer's specifications.

2410.2 Definition. The following terms are defined in Chapter 2:

SUNLIGHT DELIVERY SYSTEM.

Reason: The purpose of this proposed edit is to create a more expansive definition of the tubular daylighting device. While tubular daylighting devices are a common implementation of the principles of reflective daylighting, new advancements in the field are available worldwide and should be included in the next edition of the International Building Code. Having a more expansive definition in the International Building Code for sunlight delivery systems will open up new technologies that can introduce natural sunlight into the interior areas that do not have windows or natural light entering that room. A sunlight delivery system provides designers with a new method of daylighting that offers significantly greater capabilities than existing alternatives. Traditional daylighting methods, such as skylights or tubular daylighting devices, are limited. These systems can require multiple entry points and are often limited to top floor applications. An example of a sunlight delivery system can be found in the pictures at the end of this reason statement.

The widespread use of electrical lighting in the 20th century changed the design of buildings but often made it impossible to illuminate internal rooms with daylight, thus requiring the use of artificial light in internal spaces. The use of artificial light currently makes up as much as 45% of the energy use in commercial and industrial buildings and up to 35% in residential buildings.

Sunlight delivery systems can significantly reduce energy costs for illumination. In a paper presented to LuxEuropa in 2009 entitled *Hybrid Lighting systems: a feasibility study for Europe* by Mohammed S. Mayhoub and David Carter, energy savings ranging from 28% to 85% (in latitudes ranging from 60^o North (Oslo, Norway) to 36^o North (Khania, Greece) were reported when a variety of sunlight delivery systems were tested. These locations correlate to locations in the United States as follows: Oslo, Norway is similar in latitude to Juneau and Anchorage, AK and Khania, Greece is similar to Virginia Beach, VA; Las Vegas, NV; and Nashville, TN.

The study showed that the greatest savings were realized in the Southern most latitudes (in the Northern Hemisphere) but still showed the possibility that 50% savings could be realized at 60[°] North with the most advanced systems. Because the study was limited to European Union countries, no analysis was conducted in more southern latitudes similar to the southernmost portion of the United States where cities such as Tampa, San Antonio, and New Orleans are located. In fact, most of the land mass of the contiguous 48 United States lies well below 50[°] North indicating that greater savings could be realized in the United States than those projected in Europe.

An abundance of research and knowledge shows not only that the preferred light source in buildings is natural daylight but also that lack of exposure to daylight can lead to biological issues, lack of productivity, higher levels of stress, sleep difficulties and a variety of other human response issues. Studies suggest that creating healthy indoor lighting by providing day-lighting and natural lighting cycles can be a simple form of preventative medicine and can lead to higher production and overall better mental and physical health for the inhabitants. The health benefits that a sunlight delivery device provides is one of the reasons for this code change to be approved.



Roof top solar collecting devices used in a sunlight delivery system.



Sunlight being delivered to the interior space in an open ceiling (on the left) and in a dropped ceiling (on the right) by way of a sunlight duct system.

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

S303-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	2				202-TDD-G-BURGOS

S304–12 [A]110.3.5, 202, 2501.1.1, 2502.1, 2503.1, 2504, 2505, 2506 and 2508

Proponent: Michael Gardner, Gypsum Association (mgardner@gypsum.org)

THIS IS A TWO PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC ADMINISTRATION COMMITTEE. PART II WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE, AS TWO SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC ADMINISTATION

Revise as follows:

[A] 110.3.5 Lath, and gypsum board, and gypsum panel product inspection. Lath, and gypsum board <u>and gypsum panel product</u> inspections shall be made after lathing, and gypsum board, <u>and gypsum panel products</u>, interior and exterior, is <u>are</u> in place, but before any plastering is applied or gypsum board <u>or gypsum panel product</u> joints and fasteners are taped and finished.

Exception: Gypsum board <u>and gypsum panel products</u> that is <u>are</u> not part of a fire resistance- rated assembly or a shear assembly.

PART II – IBC STRUCTURAL

Revise as follows:

GYPSUM BOARD. The generic name for a family of sheet products consisting of a noncombustible core primarily of gypsum with paper surfacing. Gypsum wallboard, gypsum sheathing, gypsum base for gypsum *veneer* plaster, exterior gypsum soffit board, predecorated gypsum board or and water-resistant gypsum backing board complying with the standards listed in Tables 2506.2, 2507.2 and Chapter 35 are types of gypsum board

Add new text as follows:

SECTION 202 DEFINITIONS

<u>GYPSUM PANEL PRODUCT.</u> The general name for a family of sheet products consisting essentially of gypsum.

Revise as follows:

CHAPTER 25 GYPSUM BOARD, GYPSUM PANEL PRODUCTS, AND PLASTER

2501.1.1 General. Provisions of this chapter shall govern the materials, design, construction and quality of gypsum board, <u>gypsum panel products</u>, lath, gypsum plaster and cement plaster.

2501.1.2 Performance. Lathing, plastering, and gypsum board, and gypsum panel product construction shall be done in the manner and with the materials specified in this chapter, and when required for fire protection, shall also comply with the provisions of Chapter 7.

2502.1 Definitions. For the purposes of this chapter and as used elsewhere in this code, the following terms are defined in Chapter 2:

GYPSUM PANEL PRODUCTS

2503.1 Inspection. Lath, and gypsum board, gypsum panel products shall be inspected in accordance with Section 110.3.5.

2504.1 Scope. The following requirements shall be met where construction involves gypsum board, <u>aypsum panel products, or</u> lath and plaster in vertical and horizontal assemblies.

2504.1.1 Wood framing. Wood supports for lath, or gypsum board, <u>or gypsum panel products</u>, as well as wood stripping or furring, shall not be less than 2 inches (51 mm) nominal thickness in the least dimension.

Exception: The minimum nominal dimension of wood furring strips installed over solid backing shall not be less than 1 inch by 2 inches (25 mm by 51 mm).

2504.1.2 Studless partitions. The minimum thickness of vertically erected studless solid plaster partitions of 3/8- inch (9.5 mm) and 3/4-inch (19.1 mm) rib metal lath or 1/2- inch thick (12.7 mm) long-length gypsum lath, and gypsum board, or gypsum panel product partitions shall be 2 inches (51 mm).

2505.1 Resistance to shear (wood framing). Wood-framed shear walls sheathed with gypsum board, <u>gypsum panel products, or</u> lath and plaster shall be designed and constructed in accordance with Section 2306.3 and are permitted to resist wind and seismic loads. Walls resisting seismic loads shall be subject to the limitations in Section 12.2.1 of ASCE 7.

2505.2 Resistance to shear (steel framing). Cold-formed steel-framed shear walls sheathed with gypsum board <u>or gypsum panel products</u>, and constructed in accordance with the materials and provisions of Section 2211.6 are permitted to resist wind and seismic loads. Walls resisting seismic loads shall be subject to the limitations in Section 12.2.1 of ASCE 7.

SECTION 2506 GYPSUM BOARD AND GYPSUM PANEL PRODUCT MATERIALS

2506.1 General. Gypsum board, materials gypsum panel products, and accessories shall be identified by the manufacturer's designation to indicate compliance with the appropriate standards referenced in this section and stored to protect such materials from the weather.

2506.2 Standards. Gypsum board materials and gypsum panel products shall conform to the appropriate standards listed in Table 2506.2 and Chapter 35 and, where required for fire protection, shall conform to the provisions of Chapter 7.

2508.1 General. Gypsum board, <u>gypsum panel product</u>, and gypsum plaster construction shall be of the materials listed in Tables 2506.2 and 2507.2. These materials shall be assembled and installed in compliance with the appropriate standards listed in Tables 2508.1 and 2511.1.1, and Chapter 35.

INSTALLATION OF GYPSUM CONSTRUCTION				
MATERIAL	STANDARD			
Gypsum board and gypsum panel products	GA-216; ASTM C 840			
Gypsum sheathing and gypsum panel products	ASTM C 1280			
Gypsum veneer base	ASTM C 844			
Interior lathing and furring	ASTM C 841			
Steel framing for gypsum boards and gypsum	ASTM C 754; C 1007			
panel products				

TABLE 2508.1 INSTALLATION OF GYPSUM CONSTRUCTION

2508.3 Single-ply application. Edges and ends of gypsum boards <u>and gypsum panel products</u> shall occur on the framing members, except those edges and ends that are perpendicular to the framing

members. Edges and ends of gypsum boards <u>and gypsum panel products</u> shall be in moderate contact except in concealed spaces where fire-resistance rated construction, shear resistance or diaphragm action is not required.

2508.4 Joint treatment. Gypsum board <u>and gypsum panel product</u> fire-resistance-rated assemblies shall have joints and fasteners treated.

Exception: Joint and fastener treatment need not be provided where any of the following conditions occur:

- 1. Where the gypsum board <u>or the gypsum panel product</u> is to receive a decorative finish such as wood paneling, battens, acoustical finishes or any similar application that would be equivalent to joint treatment.
- 2. On single-layer systems where joints occur over wood framing members.
- 3. Square edge or tongue-and-groove edge gypsum board (V-edge), <u>gypsum panel product</u>, gypsum backing board or gypsum sheathing.
- 4. On multilayer systems where the joints of adjacent layers are offset from one to another.
- 5. Assemblies tested without joint treatment.

2508.5 Horizontal gypsum board <u>or gypsum panel product</u> diaphragm ceilings. Gypsum board <u>or gypsum panel products</u> shall be permitted to be used on wood joists to create a horizontal diaphragm ceiling in accordance with Table 2508.5.

2508.5.2 Installation. Gypsum board <u>or gypsum panel products</u> used in a horizontal diaphragm ceiling shall be installed perpendicular to ceiling framing members. End joints of adjacent courses of gypsum board shall not occur on the same joist.

2508.5.3 Blocking of perimeter edges. All perimeter edges shall be blocked using a wood member not less than 2-inch by 6-inch (51 mm by 159 mm) nominal dimension. Blocking material shall be installed flat over the top plate of the wall to provide a nailing surface not less than 2 inches (51 mm) in width for the attachment of the gypsum board <u>or gypsum panel product</u>.

2508.5.4 Fasteners. Fasteners used for the attachment of gypsum board <u>or gypsum panel products</u> to a horizontal diaphragm ceiling shall be as defined in Table 2508.5. Fasteners shall be spaced not more than 7 inches (178 mm) on center (o.c.) at all supports, including perimeter blocking, and not more than 3/8 inch (9.5 mm) from the edges and ends of the gypsum board <u>or gypsum panel product</u>.

2508.5.5 Lateral force restrictions. Gypsum board <u>or gypsum panel products</u> shall not be used in diaphragm ceilings to resist lateral forces imposed by masonry or concrete construction.

MATERIAL	THICKNESS OF MATERIAL	SPACING OF FRAMING MEMBERS	SHEAR VALUE	MINIMUM FASTENER SIZE
Gypsum board <u>or</u> gypsum panel product	No change	No change	No change	No change
Gypsum board <u>or</u> gypsum panel product	No change	No change	No change	No change

TABLE 2508.5 SHEAR CAPACITY FOR HORIZONTAL WOOD FRAMED GYPSUM BOARD DIAPHRAGM CEILING ASSEMBLIES

(Portions of Table not shown remain unchanged)

Reason: This proposal inserts the term gypsum panel product in Chapter 25 where relevant. It also revises Section 110, which is referenced by Section 2503, adds a definition for gypsum panel products to Chapter 2, and revises the existing definition for gypsum board in Chapter 2.

Gypsum panel product is a term that was created by the gypsum manufacturing industry to describe gypsum sheet products that are manufactured unfaced or with a facing other than paper. Glass mat-faced and unfaced gypsum sheet materials are examples of gypsum panel products.

Some gypsum application standards referenced by the code, such as GA 216, ASTM C 840, and ASTM C 1280, are used to define application requirements for both board and panel products, a dual role that is not reflected in current code text. In addition, while the ASTM manufacturing standards for many gypsum panel products (ref. C 1278; C1178; C1658; C1177) were incorporated into Chapter 25 during the past decade, the general text of Chapter 25 was not updated to reflect the incorporation of the new standards. This proposal addresses both issues. It adds text to Table 2508.1 to indicate where the application standards may function as an application reference standard for either a board or a panel product, and it inserts the term gypsum panel product throughout the chapter where appropriate.

The definition for gypsum panel product proposed for Chapter 2 is extracted verbatim from ASTM International Standard C 11, Standard Terminology Relating to Gypsum and Related Building Materials and Systems.

The first sentence of the proposed revision to the current definition for gypsum board is extracted verbatim from the ASTM International Standard C 11 definition for gypsum board. The existing code text has been retained for clarity, notwithstanding a slight modification.

As a part of this proposal it is also suggested that the phrase "long length" should be removed from Section 2504.1.2. It appears to be extraneous text.

Following action on this proposal, other sections of the code requiring parallel modifications will be addressed in subsequent editions of the code.

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

S304-11

PART I – INTER	NATIONAL BUILDING	GODE- ADMIN	NISTRATION		
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
PART II – INTEI		G CODE - STRI	JCTURAL		
Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	-				CH 25-S-GARDNER

S305–12 202, 2102.1 (NEW), 2502.1 (NEW)

Proponent: John Mulder, Intertek Testing Services NA, Inc., representing International Standards Organization Technical Committee 77, *Products in Fibre-reinforced Cement* and self

Revise as follows:

SECTION 202 DEFINITIONS

FIBER-CEMENT SIDING <u>PRODUCTS</u>. A Manufactured, fiber-reinforcing product made with an inorganic hydraulic or calcium silicate binder formed by chemical reaction and reinforced with discrete organic or inorganic nonasbestos fibers, or both. Additives that enhance manufacturing or product performance are permitted. thin section composites of hydraulic cementitious matrices and discrete non-asbestos fibers. Fiber-cement backer board products have either a smooth or textured face and are normally installed to wall or ceiling framing over which paint, wallpaper, resilient flooring, tile, natural stone or dimensioned stone veneer are applied. Fiber-cement underlayment products have either a smooth or textured face and are installed on a wood subfloor over which resilient flooring, tile, natural stone or dimensioned stone veneer are applied. Fiber-cement lap or panel siding, soffit, and trim products have either smooth or textured faces and are intended for *exterior wall* and related applications.

Add new text as follows:

2102.1 General. For the purposes of this chapter and as used elsewhere in this code, the following terms are defined in Chapter 2:

FIBER-CEMENT PRODUCTS

Add new text as follows:

2502.1 Definitions. The following terms are defined in Chapter 2:

FIBER-CEMENT PRODUCTS

Reason: The current definition is limited to fiber-cement siding products. The proposal corrects the definition to that published in ASTM C1154-06, *Standard Terminology for Non-Asbestos Fiber-reinforced Cement Products* (see attached copy of ASTM C1154-06), for "fiber-cement products". Additional text describes types of fiber-cement products to include also fiber-cement backer board, underlayment, soffit and trim products currently recognized in the Code (IBC Sections 1404.10, 1405.16, and 2509.2). The proposed code change eliminates a barrier to trade by including other fiber-cement products currently permitted by the Code.

A revision to Section 2103 (new Section 2103.15) is proposed to include "fiber-cement backer board and underlayment". The term "fiber-cement products" is proposed to be included in the definitions here consistent with the definition published in the Terminology Standard ASTM C1154-06, *Standard Terminology for Non-Asbestos Fiber-Reinforced Cement Products* (see attached Standard).

"Fiber-cement backer board is currently permitted for use in Section 2509.2. A new term is added to reference the permitted backer board material now defined in proposed new TABLE 2509.2, where all 3 permitted products are now listed and the proposed revision to Section 202 to include "fiber-cement products".

Cost Impact: The code change proposal will not increase the cost of construction because the change simply corrects the current definition to be consistent with the National Standard and provides examples of the types of products covered by the definition.

S305-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-			202-FIBER-CEMENT SIDING-S-MULDER	doc

S306–12 Table 2507.2, Chapter 35 (New)

Proponent: James K. Hicks, P.E., CeraTech, Inc., representing self

Revise as follows:

TABLE 2507.2 LATH, PLASTERING MATERIALS AND ACCESSORIES

MATERIAL	STANDARD			
Hydraulic Cement	<u>ASTM C 1157; C1600</u>			
	<u>ASTWC 1157, C1000</u>			

(Portions of table not shown remain unchanged)

Add new standard to Chapter 35 as follows:

ASTM

<u>C 1157-11 Standard Performance Specification for Hydraulic Cement</u> <u>C 1600-11 Standard Specification for Rapid Hardening Hydraulic Cement</u>

Reason: ASTM C 1157 Cements:

ASTM C 1157 and C 1600 cements are "Green Cements" in deference to other cements that take substantial amounts of energy and use primarily virgin materials.

More flexibility is gained by use of any of ASTM C 1157 and C 1600 cements due to their incorporating recovered materials in much of their production. They can be made by using portland cement in combination with ground granulated blast furnace slag, natural pozzolans or up to 95% fly ash in their production. These cements contrast with cements manufactured from mostly virgin materials and require significant amounts of fuel and electrical energy for their production. The above standards allow in excess of the minimum amounts of recycled materials listed in Sections 503.2 and 503.3. Having the specifications listed allows the specifier information to readily access those standards and provides for better flexibility than language allowed in the IBC.

ASTM C 1157 cements with types GU— Hydraulic cement for general construction, Type HE—High Early-Strength, Type MS—Moderate Sulfate Resistance, Type HS—High Sulfate Resistance, Type MH—Moderate Heat of Hydration, Type LH—Low Heat of Hydration can be specified. They are general counterparts for ASTM C 150 Standard Specification for Portland Cement Type I, Type II, Type II, Type V and Type II with the low heat of hydration option.

C 1600 Cements:

In addition to the above characteristics, for those instances wherein rapid hardening is desired, cements conforming to ASTM C 1600 Standard Specification for Rapid hardening Hydraulic Cements should be useable. ASTM C 1600 can be one of four cement types, General Rapid Hardening (GRH), Moderate Rapid Hardening (MRH), Very Rapid Hardening (VRH) and Ultra Rapid Hardening (URH).

C 1600 is a Specification giving numerous performance requirements. Primary characteristics (with inherent increased design flexibility) are:

•Can produce rapid-hardening concrete, precast concrete, block, mortar and grout and is used in rapid hardening stuccos and plasters.

 Depending on the type cement used and the specific mixture, cements meeting ASTM C 1600 can provide either normal, medium or fast time to service (1.5 to 48 h)

•ASTM C 1600 has rigid durability requirements.

ASTM C 1600 cements are used in products such as:

Materials for Concrete Repairs

High Strength Grouts

Precast

•Paving

•Some Cements - Mass Concrete

•Some Cements – Heat Resistant

•Some Cements – Chemical Resistant

Cost Impact: Economic cost of plaster utilizing C 1157 cements may be equal or slightly lower than portland cement concrete due to their sometimes lower process and additive costs. Environmental costs are generally lower with C 1157 cements as fuel use is generally less, costs of components may be less or with the case of activated fly ash based cements, no fuel is used and grinding is not required.

Economic cost of plaster utilizing C 1600 cements, while it may be approximately equal or higher when comparing cementitious to cementitious, is typically negligible for the concrete when considering the costs of other ingredients, transport, placement, finishing and curing.

Environmental costs are generally lower with C 1600 cements as fuel use is generally less, costs of components may be less or with the case of activated fly ash based cements, no fuel is used and grinding is not required.

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S306-12

Public Hearing:	Committee:	AS	AM	D	
-	Assembly:	ASF	AMF	DF	
	5				35-ASTM-S-HICKS.doc

S307–12 2509.2, Table 2509.2 (NEW), Chapter 35 (NEW)

Proponent: John Mulder, Intertek Testing Services NA, Inc. representing International Standards Organization Technical Committee 77, *Products in Fibre-reinforced cement* and self

Revise as follows:

2509.2 Base for tile. Glass mat water-resistant gypsum backing panels, discrete nonasbestos fibercement interior substrate sheets or nonasbestos fiber-mat reinforced cementitious backer units in compliance with ASTM C 1178, C 1288 or C 1325 and installed in accordance with manufacturer recommendations shall be <u>Materials</u> used as a base for wall tile in tub and shower areas and wall and ceiling panels in shower areas <u>shall be of materials listed in Table 2509.2 and installed in accordance</u> <u>with manufacturer recommendations</u>. 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.

BACKERBOARD MATERIALS				
MATERIAL	<u>STANDARD</u>			
Glass mat gypsum backing panel	ASTM C1178			
Nonasbestos fiber-cement backer board	ASTM C1288 or ISO 8336			
Nonasbestos fiber mat reinforced cementitious backer unit	<u>ASTM C1325</u>			

TABLE 2509.2 BACKERBOARD MATERIALS

Add new standard to Chapter 35 as follows:

ISO

8336 Fibre-cement flat sheets -- Product specification and test methods

Reason: GYPSUM BOARD" IN SHOWER AND WATER CLOSETS misrepresents the materials permitted for use in this section, specifically fiber-reinforced cement backer board products. The text is revised to reference permitted backer board materials now defined in new TABLE 2509.2, where all 3 permitted products would now be listed. This revision also makes the addition of future recognized products to the Code easier by simple addition to the table.

Performance requirements of ISO 8336, *Fibre-cement flat sheets – Product specification and test methods*, have been harmonized with the performance requirements of ASTM C1288, *Standard Specification for Discrete Non-Asbestos Fiber-Cement Interior Substrate Sheets*. Fiber-cement producers in Mexico, Central and South America, Europe, Asia, Australia and New Zealand currently manufacture and test their fiber-cement siding products for compliance with ISO 8336. The inclusion of this Standard reference in the IBC will permit manufacturers worldwide to demonstrate product compliance to IBC requirements. The addition of a reference to ISO 8336 in the Code removes a barrier to trade

Cost Impact: The code change proposal will not increase the cost of construction because the proposed code change is editorial in nature to better clarify and present the backer board products currently recognized in the Code.

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S307-12					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	
	-				2509-S-MULDER doc

S308-12 2509.3

Proponent: Michael Gardner, Gypsum Association (mgardner@gypsum.org)

Revise as follows:

2509.3 Limitations. Water-resistant gypsum backing board shall not be used in the following locations:

- 1. Over a vapor retarder in shower or bathtub compartments.
- 2. Where there will be direct exposure to water or in areas subject to continuous high humidity.
- 3. On ceilings where frame spacing exceeds 12 inches (305 mm) o.c. for 1/2-inch thick (12.7 mm) water-resistant gypsum backing board and more than 16 inches (406 mm) o.c. for 5/8-inch thick (15.9 mm) water-resistant gypsum backing board.

Reason: Concurrent language necessitating the addition of supplemental framing members when water-resistant ceiling board is installed on a ceiling has been or is being removed from the code-referenced gypsum board and panel application standards, GA-216 and ASTM C 840.

Testing has shown that water-resistant gypsum board, as presently manufactured, has better sag resistance than regular core board of the same thickness. As a consequence, the supplemental framing limitation is no longer necessary.

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

S308-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2509.3-S-GARDNER

S309–12 202 (NEW), 1404.13 (NEW), 2510.6, Chapter 35 (NEW)

Proponent: John Woestman, Kellen Company, representing Building Enclosure Moisture Management Institute (BEMMI) (jwoestman@kellencompany.com)

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

Add new text as follows:

SECTION 202 DEFINITIONS

POLYMERIC RAINSCREEN PRODUCT. Material in roll or sheet form, installed behind exterior cladding products, that creates a space that allows drainage and ventilation of liquid and vapor moisture that enters an above-grade exterior wall assembly. Rainscreen products are used to reduce / minimize water transfer to the water resistive barrier.

Add new text as follows:

1404.13 Polymeric rainscreen products. Polymeric rainscreen products shall comply with BEMMI 100.

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. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

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 <u>a</u> drainage space, or <u>polymeric</u> rainscreen products complying with BEMMI 100.

Add new standard to Chapter 35 as follows:

BEMMI Building Enclosure Moisture Management Institute <u>355 Lexington Avenue, 15th Floor</u> New York, NY 10017-6603

BEMMI 100-12 Voluntary Test Standard for Evaluation of Polymeric Rainscreen Products

Reason: The Building Enclosure Moisture Management Institute (BEMMI) is developing a new voluntary test standard for evaluation of rainscreen products. This standard will include tests to indicate performance of the rainscreen material behind cladding systems and facilitate drainage and drying of moisture that may get into the wall system behind the cladding.

This standard is currently under development, and is targeted for completion by the Final Action Hearings in 2012. Note: this proposal is not requiring rainscreens to be incorporated in wall systems, only that if rainscreen materials are used, they are to comply with the BEMMI standard.

Cost Impact: This proposal will not increase the cost of construction as it will be an option to current requirements.

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S309-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				1404.13 (NEW)-S-WOESTMAN.doc

S310–12 2510.6, Chapter 35 (NEW)

Proponent: Theresa Weston, DuPont Building Innovation (theresa.a.weston@usa.dupont.com)

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 water-resistive barrier complying with ASTM E 2556 Type 1. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

Exception: Where the *water-resistive barrier* that is applied over wood-based sheathing has a water resistance equal to or greater than that of <u>60-minute Grade D paper</u> <u>a water-resistive barrier</u> <u>complying with ASTM E 2556 Type II</u> and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.

Add new standard to Chapter 35 as follows:

ASTM

<u>E 2556 - Standard Specification for Vapor Permeable Flexible Sheet Water-Resistive Barriers Intended</u> for Mechanical Attachment

Reason: The proposal updates the water-resistive barrier reference to the most recent consensus standard. ASTM E2556 includes house wrap materials, building papers and felt, instead of just building paper and therefore is more representative of the state of the industry. Within ASTM E2556 Grade D paper is a Type I WRB and 60 minute Grade D paper is a Type II WRB. ASTM E2556 is consistent with the current ICC-ES acceptance criteria for water-resistive barriers (AC-38) and therefore should not limit the use of current WRBs.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S310-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
2				2510.6-S-WESTON.doc

S311–12 2510.6

Proponent: John Woestman, Kellen Company, representing Builders Masonry Veneer Manufacturers Association (MVMA) (jwoestman@kellencompany.com)

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 water-resistive barrier with a moisture vapor permeance equal to or greater than that of two layers of Grade D paper. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

Exception: Where the *water-resistive barrier* that is applied over wood-based sheathing has a water resistance <u>and a moisture vapor permeance</u> 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.

Reason: Existing language may be considered ambiguous as to what performance attribute is desired to be at least equivalent to two layers of Grade D paper. Water resistance? Moisture vapor permeance? This proposal clarifies moisture vapor permeability is the performance attribute desired to be at least equivalent to Grade D paper. And in the Exception, states moisture vapor permeance equal to or greater than that of 60-minute Grade D paper.

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

S311-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				2510 6-S-WOESTMAN doc

S312–12 202 (NEW), 2614 (NEW), Chapter 35 (NEW)

Proponent: Betsy Steiner, EPS Molders Association (emsteiner@epscentral.org)

Add new text as follows:

SECTION 202 DEFINITIONS

<u>GEOFOAM</u> – Block or planar rigid cellular foam polymeric material used in geotechnical engineering applications.

Add new text as follows:

SECTION 2614 GEOFOAM

2614.1 General. The provisions of this Section shall govern the quality and methods of application of geofoam for use as a load bearing material in buildings and structures.

2614.2 Material standards. Geofoam shall comply with ASTM D 6817.

2614.3 Load bearing value. The allowable load bearing capacity of geofoam shall be the compressive resistance at 1% deformation in accordance with ASTM D 6817.

2614.4 Labeling and identification. Geofoam delivered to the job site shall bear the label of an approved agency showing the manufacturer's name, product listing, product identification and information sufficient to determine that the end use will comply with the code requirements.

2614.5 Surface-burning characteristics. Geofoam shall have a maximum flame spread index of 75 and a smoke-developed index of 450 when tested at a thickness of 4 inches (102 mm).

2614.6 Protection. Geofoam 4 inches (102 mm) or less in thickness shall be separated from the interior of a building by ½-inch (12.7 mm) gypsum wallboard or a material that is tested in accordance with and meets the acceptance criteria of both the Temperature Transmission Fire Test and the Integrity Fire Test of NFPA 275. Geofoam greater than 4 inches (102 mm) in thickness, shall be separated from the interior of the building by two layers of Type X gypsum wallboard or a minimum of 1-inch (25 mm) thickness of masonry or concrete.

Add new standard to Chapter 35 as follows:

ASTM

D 6817-11 - Standard Specification for Rigid Cellular Polystyrene Geofoam

Reason: Geofoam has been used as a geotechnical material since 1960's providing lightweight, stable solution to engineering challenges. Its many applications include providing stable, insulating sub-surface for building foundations; slope stabilization; road, runway, railway base layer; stadium and theater tiered platform base. Geofoam, in addition to providing excellent insulation also delivers earthquake shock, noise and vibration dampening. Physical properties of geofoam have been established by ASTM Standard D6817. The current version of the standard, D6817-11, is attached to this proposal.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S312-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
2				2614 (NEW)-S-STEINER.doc

S313–12 Chapter 27 (NEW), Chapter 35

Proponent: Paula Baker-Laporte, FAIA, EcoNest Company, representing Natural Building Network (paula@econest.com)

THIS IS A THREE PART CODE CHANGE ALL PARTS WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE AS THREE SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDER FOR THIS COMMITTEE

Add new text as follows:

CHAPTER 27 LIGHT STRAW-CLAY CONSTRUCTION

SECTION 2701 GENERAL

2701.1. Scope. This chapter shall govern the use of *light straw-clay* as a *non-bearing* building material and wall *infill* system.

SECTION 2702 DEFINITIONS

2702.1. General. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein. Refer to Chapter 2 of the *International Building Code* for general definitions.

CLAY. Inorganic soil with particle sizes less than 0.00008 in. (0.002 mm) having the characteristics of high to very high dry strength and medium to high plasticity.

CLAY SLIP. A suspension of clay soil in water.

CLAY SOIL. Inorganic soil containing 50% or more clay by volume.

INFILL. Light straw-clay that is placed between the structural members of a building.

LIGHT STRAW-CLAY. A mixture of straw and clay compacted to form insulation and plaster substrate between or around structural and non-structural members in a wall.

NON-BEARING. Not bearing the weight of the building other than the weight of the light straw-clay itself and its finish.

STRAW. The dry stems of cereal grains after the seed heads have been removed.

VOID. Any space in a light straw-clay wall in which a 2-inch (51 mm) sphere can be inserted.

SECTION 2703 NON-BEARING LIGHT STRAW-CLAY CONSTRUCTION

2703.1 General. Light straw-clay shall not be used to support the weight of the building other than the weight of the light straw-clay material and its finish. Light straw-clay shall be limited to use as infill between or around structural and non-structural wall framing members.

2703.2 Structure. The structure of buildings using light straw-clay shall be designed in accordance with the *International Building Code*. Unfinished light straw-clay shall be deemed to have a design dead load

of 40 pounds per cubic foot (640 kg per cubic meter) unless otherwise demonstrated to the *building* official.

2703.3 Materials. The materials used in light straw-clay construction shall be in accordance with Sections 2703.3.1, 2703.3.2 and 2703.3.3.

2703.3.1 Straw. Straw shall be wheat, rye, oats, rice, or barley, and shall be free of visible decay and insects.

2703.3.2 Clay Soil. Suitability of clay soil shall be determined in accordance with the Figure 2 Ribbon Test or the Figure 3 Ball Test of the Appendix to ASTM E 2392/2392M.

2703.3.3 Clay slip. Clay slip shall be of sufficient viscosity such that a finger dipped in the slip and withdrawn remains coated with an opaque coating.

2703.3.4 Light straw-clay mixture. Light straw-clay shall contain not less than 65 percent and not more than 85 percent straw, by volume of bale-compacted straw to clay soil. Loose straw shall be mixed and coated with clay slip such that there is no more than 5 percent uncoated straw.

2703.4 Wall Construction. Light straw-clay wall construction shall be in accordance with the requirements of Sections 2703.4.1, through 2703.4.8.

2703.4.1 Number of stories. The light straw-clay infill system requirements of this chapter shall be limited to buildings and structures that are not more than 2 stories in height above grade plane. Light straw-clay infill systems for buildings that are greater than 2 stories in height above grade plane shall be in accordance with an approved design by a registered design professional.

2703.4.2 Light straw-clay maximum thickness. Light straw-clay shall be not more than 12 inches (305 mm) thick, to allow adequate drying of the installed material.

2703.4.3 Distance above grade. Light straw-clay shall not be used below grade. Light straw-clay and its exterior finish shall be not less than 8 inches (203 mm) above exterior finished grade.

2703.4.4 Moisture barrier. An approved moisture barrier shall separate the bottom of light straw-clay walls from any masonry or concrete foundation or slab that directly supports the walls. Penetrations and joints in the barrier shall be sealed with an approved sealant.

2703.4.5 Contact with wood members. Light straw clay shall be permitted to be in contact with untreated wood members.

2703.4.6 Contact with non-wood structural members.

Non-wood structural members in contact with light straw-clay shall be resistant to corrosion or shall be coated to prevent corrosion with an approved coating.

2703.4.7 Wall Reinforcing. Light straw-clay shall be reinforced as follows:

- Vertical reinforcing shall be a minimum of nominal 2-inch by 4-inch (51 mm by 102 mm) wood members at a maximum of 32 inches (813 mm) on center where the vertical reinforcing is nonbearing and at 24"(610mm) on center where it is load-bearing. The vertical reinforcing shall be attached at top and bottom in accordance with Table 2304.9.1 and anchored to the foundation in accordance with Section 2308.6 or shall be in accordance with an approved design by a registered design professional. Vertical reinforcing shall not exceed an unrestrained height of 10 feet (3,048 mm) or shall be in accordance with an approved design by a registered design professional.
- 2. <u>Horizontal reinforcing to control settlement of the light straw-clay infill, and to resist out of plane</u> forces shall be installed in the center of the wall at not more than 24 inches (610 mm) on center

and shall be secured to vertical members. Horizontal reinforcing shall be of any of the following: <u>3/4</u> inch (19 mm) bamboo, <u>1/2</u> inch (13 mm) fiberglass rod, <u>1-inch</u> (25 mm) wood dowel or nominal <u>1-inch</u> by <u>2-inch</u> (25 mm by 51 mm) wood.

2703.4.8 Installation. Light straw-clay shall be installed in accordance with the following:

- 1. Formwork shall be sufficiently strong to resist bowing when the light straw-clay is compacted into the forms.
- 2. Light straw-clay shall be uniformly placed into forms and evenly tamped to achieve stable walls free of voids. Light straw-clay shall be placed in lifts of no more than 6 inches (152 mm) and shall be thoroughly tamped before additional material is added.
- 3 Formwork shall be removed from walls within 24 hours after tamping, and walls shall remain exposed until moisture content is in accordance with Section 2703.5.2. Any visible voids shall be patched with light straw-clay prior to plastering.

2703.4.9 Openings in Walls. Openings in walls shall be in accordance with the following:

- 1. Doors and windows. Rough bucks or frames for doors and windows shall be fastened securely to structural members. Windows and doors shall be flashed in accordance with the International Building Code.
- 2. Window sills. An approved moisture barrier shall be installed at window sills in light straw-clay walls prior to installation of windows.

2703.5 Wall Finishes. The interior and exterior surfaces of light straw-clay walls shall be protected with a finish in accordance with Sections 2703.5.1 through 2703.5.4.

2703.5.1 Moisture content of light straw-clay prior to application of finish. Light straw-clay walls shall be dry to a maximum moisture content of 20 percent at a depth of 4 inches (102 mm), as measured from each side of the wall, prior to the application of finish on either side of the wall. Moisture content shall be measured with a moisture meter equipped with a probe that is designed for use with baled straw or hay.

2703.5.2 Plaster finish. Exterior plaster finishes shall be clay plaster and lime plaster. Interior plaster finishes shall be clay plaster, lime plaster, and gypsum plaster. Plasters shall be permitted to be applied directly to the surface of the light straw-clay walls without reinforcement, except that the juncture of dissimilar substrates shall be in accordance with Section 2703.5.3. Exterior clay plaster shall be finished with a lime-based or silicate-mineral coating.

2703.5.3 Bridging across dissimilar substrates. Bridging shall be installed across dissimilar substrates prior to the application of plaster. Acceptable bridging materials shall include: expanded metal lath, woven wire mesh, welded wire mesh, fiberglass mesh, reed matting, or burlap. Bridging shall extend not less than 4 inches (102 mm), on both sides of the juncture.

2703.5.4 Exterior siding. Exterior wood, metal, or composite material siding shall be spaced a minimum of 3/4 inch (19 mm) from the light straw-clay such that a ventilation space is created to allow for moisture diffusion. The siding shall be fastened to wood furring strips in accordance with manufacturer's recommendations. Furring strips shall be spaced not more than 32 inches (813 mm) on center, and shall be securely fastened to the vertical wall reinforcing or structural framing. Insect screening shall be provided at the top and bottom of the ventilation space. An air barrier consisting of clay plaster, lime plaster, or other approved air barrier shall be applied to the light straw-clay prior to application of siding.

PART II – IBC GENERAL

Add new text as follows:

SECTION 2704 TYPE OF CONSTRUCTION

2704.1 Type of construction. Buildings or portions thereof containing light straw-clay in accordance with this chapter shall be classified as Type V-B construction.

PART III - IECC

Add new text as follows:

SECTION 2705 THERMAL INSULATION

2705.1 R-value. Light straw-clay, when installed in accordance with this chapter, shall be deemed to have an R-value of 1.6 per inch.

Add new standard to Chapter 35 as follows:

ASTM

E 2392/2392 M-10 Standard Guide for Design of Earthen Wall Building Systems

Reason: The purpose of the proposed code change is to include the use of Light Straw Clay as a nonload-bearing building material and wall infill ystem into the IBC because no such section currently exists.

Light straw-clay construction has proven to be a viable, ecologically sound, and energy efficient building method. To date, permitting of light straw-clay construction has generally been left to the discretion of individual building officials on a case-by-case basis. Two exceptions are the State of New Mexico and the State of Oregon. Since 1998 the State of New Mexico has successfully permitted straw-clay construction using its standard permitting process when a project complies with its "Clay Straw Guidelines".

The proposed light straw-clay section of the IBC is derived from and builds upon the fourteen years of success of New Mexico's Clay Straw Guidelines. In October of 2011 the Oregon Reach Code (ORC) was amended to include light straw-clay construction. Inclusion in the IBC would make proven provisions accessible to more designers and builders interested in using this environmentally beneficial material and to building officials who will be evaluating and enforcing its proper use.

The proposed mixture of clay and straw as a monolithic non-load bearing building enclosure has been successfully used in the United States since 1990 and since 1950 in Europe. Prior to this a heavier form of clay, straw, and woven wood construction known as wattle and daub was in common use throughout Europe, Africa, Asia, and North and South America. Many thousands of existing structures dating back 300-400 years have been continuously occupied, attesting to the durability of these natural materials. In the United States residential and non-residential structures using straw-clay have been completed in 17 states, and most of those have been constructed with full permits and inspections.

The centuries old European predecessors and light straw clay buildings built to date in North America have all been constructed without the use of a moisture barrier. The proposed light straw clay materials are vapor permeable and do not require a moisture barrier. Code precedents for vapor permeable construction exist for adobe construction, log construction and half-timber construction. In these systems as in light straw clay construction there is sufficient hygric capacity to hold and re-release moisture without damage to structural members or degradation of the wall due to weather related moisture fluctuations. Furthermore for exterior siding applications, with ventilated space and rain screen a water resistive barrier is not necessary.

Through The EcoNest Company, and as a licensed architect for over 25 years, I have been involved in the design and/or construction of over 50 buildings utilizing light straw-clay construction. In 2005 I co-authored, with my husband and business partner Robert Laporte, the book "Econest, Creating Sustainable Sanctuaries of Clay, Straw and Timber".

Official guidelines for straw-clay construction have been in effect in New Mexico since 1998. At least 20 residential structures have been successfully permitted and built since that time in New Mexico following these guidelines. Other building officials in surrounding States have also permitted straw-clay construction in their jurisdictions based on these guidelines.

In 2004 the Canada Mortgage and Housing Corporation (CMHC) funded a study to explore the material characteristics of Straw Light Clay (SLC) construction. The proposed section for the IBC uses this study as well as the many years of experience of our company and other practitioners of light straw-clay construction as its basis. The CMHC study includes issues of thermal performance, fire-resistance, moisture, and vapor permeability. The CMHC study and other supporting documentation is available for viewing and download at: http://www.econesthomes.com/natural-building-resources/technical/. EcoNest's numerous projects utilizing light straw-clay construction can be viewed at www.econesthomes.com.

- 2011 Oregon Reach Code (Section 1307) (Based on 2012 International Green Construction Code)
- Baker-Laporte, Laporte (2005) *Econest, Creating Sustainable Sanctuaries of Clay, Straw and Timber*, Gibbs Smith Publishers (This book is available only by purchase. See http://www.econesthomes.com/bookstore/)

• J. Thornton (2004) Initial Material Characterization of Straw Light Clay Canada Mortgage and Housing Corporation State of New Mexico Construction Industries Division (2001) Clay Straw Guidelines

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2012.

S313-12				
PART I - INTERNATIONAL BU	LDING COD	E - STRUC	TURAL	
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
PART II – INTERNATIONAL BU		DE - GENER	RAL	
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
PART III - INTERNATIONAL EN		ISERVATIO	N CODE	
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
,				2701-(NEW)-S-BAKER-LAPORTE.doc
S314–12 Appendix N (NEW)

Proponent: Paula Baker-Laporte, FAIA, EcoNest Company, representing Natural Building Network (paula@econest.com)

Add new text as follows:

APPENDIX N LIGHT STRAW-CLAY CONSTRUCTION

<u>The provisions contained in this appendix are not mandatory unless specifically referenced in the adopting ordinance.</u>

SECTION N101 GENERAL

N101.1. Scope. This appendix shall govern the use of *light straw-clay* as a *non-bearing* building material and wall infill system.

SECTION N102 DEFINITIONS

N102.1. General. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein. Refer to Chapter 2 for the *International Building Code* for general definitions.

CLAY. Inorganic soil with particle sizes less than 0.00008 in. (0.002 mm) having the characteristics of high to very high dry strength and medium to high plasticity.

CLAY SLIP. A suspension of clay soil in water.

CLAY SOIL. Inorganic soil containing 50% or more clay by volume.

INFILL. Light straw-clay that is placed between the structural members of a building.

LIGHT STRAW-CLAY. A mixture of straw and clay compacted to form insulation and plaster substrate between or around structural and non-structural members in a wall.

NON-BEARING. Not bearing the weight of the building other than the weight of the light straw-clay itself and its finish.

STRAW. The dry stems of cereal grains after the seed heads have been removed.

VOID. Any space in a light straw-clay wall in which a 2-inch (51 mm) sphere can be inserted.

SECTION N103 NON-BEARING LIGHT STRAW-CLAY CONSTRUCTION

N103.1 General. Light straw-clay shall not be used to support the weight of the building other than the weight of the light straw-clay material and its finish. Light straw-clay shall be limited to use as infill between or around structural and non-structural wall framing members.

N103.2 Structure. The structure of buildings using light straw-clay shall be designed in accordance with the *International Building Code*. Unfinished light straw-clay shall be deemed to have a design dead load of 40 pounds per cubic foot (640 kg per cubic meter) unless otherwise demonstrated to the *building*

official.

N103.3 Materials. The materials used in light straw-clay construction shall be in accordance with Sections N103.3.1, N103.3.2, N103.3.3 and N103.3.4.

N103.3.1 Straw. Straw shall be wheat, rye, oats, rice, or barley, and shall be free of visible decay and insects.

N103.3.2 Clay soil. Suitability of clay soil shall be determined in accordance with the Figure 2 Ribbon Test or the Figure 3 Ball Test of the Appendix to ASTM 2392/2392M.

N103.3.3 Clay slip. Clay slip shall be of sufficient viscosity such that a finger dipped in the slip and withdrawn remains coated with an opaque coating.

N103.3.4 Light straw-clay mixture. Light straw-clay shall contain a minimum of 65 percent and a maximum of 85 percent straw, by volume of bale-compacted straw to clay soil. Loose straw shall be mixed and coated with clay slip such that there is no more than 5 percent uncoated straw.

N103.4 Wall Construction. Light straw-clay wall construction shall be in accordance with the requirements of Sections N103.4.1, through N103.4.8.

N103.4.1 Number of stories. The light straw-clay infill system requirements of this chapter shall be limited to buildings and structures that are not more than 2 stories in height above grade plane. Light straw-clay infill systems for buildings that are greater than 2 stories in height above grade plane shall be in accordance with an approved design by a registered design professional.

N103.4.2 Light straw-clay maximum thickness. Light straw-clay shall be not more than12 inches (305 mm) thick, to allow adequate drying of the installed material.

N103.4.3 Distance above grade. Light straw-clay shall not be used below grade. Light straw-clay and its exterior finish shall be not less than 8 inches (203 mm) above exterior finished grade.

N103.4.4 Moisture barrier. An approved moisture barrier shall separate the bottom of light straw-clay walls from any masonry or concrete foundation or slab that directly supports the walls. Penetrations and joints in the barrier, shall be sealed with an approved sealant.

<u>N103.4.5</u> Contact with wood members. Light straw clay shall be permitted to be in contact with untreated wood members.

N103.4.6 Contact with non-wood structural members. Non-wood structural members in contact with light straw-clay shall be resistant to corrosion or shall be coated to prevent corrosion with an approved coating.

N103.4.7 Wall Reinforcing. Light straw-clay shall be reinforced as follows:

- 1. Vertical reinforcing shall be a minimum of nominal 2-inch by 4-inch (51 mm by 102 mm) wood members at a maximum of 32 inches (813 mm) on center where the vertical reinforcing is nonbearing and at 24"(610mm) on center where it is load-bearing. The vertical reinforcing shall be attached at top and bottom in accordance with Table 2304.9.1 and anchored to the foundation in accordance with Section 2308.6 or shall be in accordance with an approved design by a registered design professional. Vertical reinforcing shall not exceed an unrestrained height of 10 feet (3,048 mm) or shall be in accordance with an approved design by a registered design professional.
- 2. Horizontal reinforcing to control settlement of the light straw-clay infill, and to resist out of plane forces shall be installed in the center of the wall at not more than 24 inches (610 mm) on center

and shall be secured to vertical members. Horizontal reinforcing shall be of any of the following: <u>3/4</u> inch (19 mm) bamboo, <u>1/2</u> inch (13 mm) fiberglass rod, <u>1-inch</u> (25 mm) wood dowel or nominal <u>1-inch</u> by <u>2-inch</u> (25 mm by 51 mm) wood.

N103.4.8 Installation. Light straw-clay shall be installed in accordance with the following:

- 1. Formwork shall be sufficiently strong to resist bowing when the light straw-clay is compacted into the forms.
- 2. Light straw-clay shall be uniformly placed into forms and evenly tamped to achieve stable walls free of voids. Light straw-clay shall be placed in lifts of no more than 6 inches (152 mm) and shall be thoroughly tamped before additional material is added.
- 3. Formwork shall be removed from walls within 24 hours after tamping, and walls shall remain exposed until moisture content is in accordance with Section N103.5. Any visible voids shall be patched with light straw-clay prior to plastering.

N103.4.9 Openings in Walls. Openings in walls shall be in accordance with the following:

- 1. Doors and windows. Rough bucks or frames for doors and windows shall be fastened securely to structural members. Windows and doors shall be flashed in accordance with the International Building Code.
- 2. Window sills. An approved moisture barrier shall be installed at window sills in light straw-clay walls prior to installation of windows.

N103.5 Wall Finishes. The interior and exterior surfaces of light straw-clay walls shall be protected with a finish in accordance with Sections N103.5.1 through N103.5.4.

N103.5.1 Moisture content of light straw-clay prior to application of finish. Light straw-clay walls shall be dry to a maximum moisture content of 20 percent at a depth of 4 inches (102 mm), as measured from each side of the wall, prior to the application of finish on either side of the wall. Moisture content shall be measured with a moisture meter equipped with a probe that is designed for use with baled straw or hay.

N103.5.2 Plaster finish. Exterior plaster finishes shall be clay plaster and lime plaster. Interior plaster finishes shall be clay plaster, lime plaster, and gypsum plaster. Plasters shall be permitted to be applied directly to the surface of the light straw-clay walls without reinforcement, except that the juncture of dissimilar substrates shall be in accordance with Section N103.5.3. Exterior clay plaster shall be finished with a lime-based or silicate-mineral coating.

N103.5.3 Bridging across dissimilar substrates. Bridging shall be installed across dissimilar substrates prior to the application of plaster. Acceptable bridging materials shall include: expanded metal lath, woven wire mesh, welded wire mesh, fiberglass mesh, reed matting, or burlap. Bridging shall extend not less than 4 inches (102 mm), on both sides of the juncture.

N103.5.4 Exterior siding. Exterior wood, metal, or composite material siding shall be spaced a minimum of 3/4 inch (19 mm) from the light straw-clay such that a ventilation space is created to allow for moisture diffusion. The siding shall be fastened to wood furring strips in accordance with manufacturer's recommendations. Furring strips shall be spaced not more than 32 inches (813 mm) on center, and shall be securely fastened to the vertical wall reinforcing or structural framing. Insect screening shall be provided at the top and bottom of the ventilation space. An air barrier consisting of clay plaster, lime plaster, or other approved air barrier shall be applied to the light straw-clay prior to application of siding.

SECTION N104 TYPE OF CONSTRUCTION

N104.1 Type of construction. Buildings or portions thereof containing light straw-clay in accordance with this appendix shall be classified as Type V-B construction.

SECTION N105 THERMAL INSULATION

N105.1 R-value. Light straw-clay, when installed in accordance with this chapter, shall be deemed to have an R-value of 1.6 per inch.

SECTION N106 REFERENCE STANDARDS

<u>ASTM</u>

E2392-10 Standard Guide for Design Earthen Wall Building Systems

Reason: The purpose of the proposed code change is to include Light Straw Clay as a nonload-bearing building material and wall infill system into the IBC because no such section currently exists.

Light straw-clay construction has proven to be a viable, ecologically sound, and energy efficient building method. To date, permitting of light straw-clay construction has generally been left to the discretion of individual building officials on a case-by-case basis. Two exceptions are the State of New Mexico and the State of Oregon. Since 1998 the State of New Mexico has successfully permitted straw-clay construction using its standard permitting process when a project complies with its "Clay Straw Guidelines".

The proposed light straw-clay section of the IBC is derived from and builds upon the fourteen years of success of New Mexico's Clay Straw Guidelines. In October of 2011 the Oregon Reach Code (ORC) was amended to include light straw-clay construction. Inclusion in the IBC would make proven provisions accessible to more designers and builders interested in using this environmentally beneficial material and to building officials who will be evaluating and enforcing its proper use.

The proposed mixture of clay and straw as a monolithic non-load bearing building enclosure has been successfully used in the United States since 1990 and since 1950 in Europe. Prior to this a heavier form of clay, straw, and woven wood construction known as wattle and daub was in common use throughout Europe, Africa, Asia, and North and South America. Many thousands of existing structures dating back 300-400 years have been continuously occupied, attesting to the durability of these natural materials. In the United States residential and non-residential structures using straw-clay have been completed in 17 states, and most of those have been constructed with full permits and inspections.

The centuries old European predecessors and light straw clay buildings built to date in North America have all been constructed without the use of a moisture barrier. The proposed light straw clay materials are vapor permeable and do not require a moisture barrier. Code precedents for vapor permeable construction exist for adobe construction, log construction and half-timber construction. In these systems as in light straw clay construction there is sufficient hygric capacity to hold and re-release moisture without damage to structural members or degradation of the wall due to weather related moisture fluctuations. Furthermore for exterior siding applications, with ventilated space and rain screen a water resistive barrier is not necessary.

Through The EcoNest Company, and as a licensed architect for over 25 years, I have been involved in the design and/or construction of over 50 buildings utilizing light straw-clay construction. In 2005 I co-authored, with my husband and business partner Robert Laporte, the book "Econest, Creating Sustainable Sanctuaries of Clay, Straw and Timber".

Official guidelines for straw-clay construction have been in effect in New Mexico since 1998. At least 20 residential structures have been successfully permitted and built since that time in New Mexico following these guidelines. Other building officials in surrounding States have also permitted straw-clay construction in their jurisdictions based on these guidelines.

In 2004 the Canada Mortgage and Housing Corporation (CMHC) funded a study to explore the material characteristics of Straw Light Clay (SLC) construction. The proposed section for the IBC uses this study as well as the many years of experience of our company and other practitioners of light straw-clay construction as its basis. The CMHC study includes issues of thermal performance, fire-resistance, moisture, and vapor permeability. The CMHC study and other supporting documentation is available for viewing and download at: http://www.econesthomes.com/natural-building-resources/technical/. EcoNest's numerous projects utilizing light straw-clay construction can be viewed at www.econesthomes.com.

2011 Oregon Reach Code (Section 1307) (Based on 2012 International Green Construction Code)

Baker-Laporte, Laporte (2005) *Econest, Creating Sustainable Sanctuaries of Clay, Straw and Timber*, Gibbs Smith Publishers (This book is available only by purchase. See http://www.econesthomes.com/bookstore/)

J. Thornton (2004) Initial Material Characterization of Straw Light Clay Canada Mortgage and Housing Corporation

State of New Mexico Construction Industries Division (2001) Clay Straw Guidelines

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

0014-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
	-				N101-(NEW)-S-BAKER-LAPORTE doc

S311-12

S315–12 Appendix N (NEW)

Proponent: Martin Hammer, Architect, representing California Straw Building Association, Colorado Straw Bale Association, Straw Bale Construction Association – New Mexico, Ontario Bale Building Coalition, Development Center for Appropriate Technology, Environmental Building Network (<u>mfhammer@pacbell.net</u>)

THIS IS A TWO PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IBC FIRE SAFETY COMMITTEE. SEE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES

PART I – IBC STRUCTURAL

Add new text as follows:

APPENDIX N STRAWBALE CONSTRUCTION

The provisions contained in this appendix are not mandatory unless specifically referenced in the adopting ordinance.

SECTION N101 GENERAL

<u>N101.1 Scope.</u> This appendix shall govern the use of baled straw as a building material.

SECTION N102 DEFINITIONS

N102.1 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein. Refer to Chapter 2 of the *International Building Code* for general definitions.

BALE. Equivalent to straw bale.

CLAY. Inorganic soil with particle sizes less than 0.00008 in. (0.002 mm) having the characteristics of high to very high dry strength and medium to high plasticity.

CLAY SLIP. A suspension of clay particles in water.

FLAKE. An intact section of compressed straw removed from an untied bale.

LAID FLAT. The orientation of a bale with its largest faces horizontal, its longest dimension parallel with the wall plane, its ties concealed in the unfinished wall and its straw lengths oriented across the thickness of the wall.

LOAD-BEARING WALL. For the purposes of this appendix, any strawbale wall that supports more than 100 lb/linear ft (1,459 N/m) of vertical load in addition to its own weight.

MESH. An openwork fabric of linked strands of metal, plastic, or natural or synthetic fiber, embedded in plaster to provide tensile reinforcement or bonding.

NONLOAD-BEARING WALL. For the purpose of this appendix, any wall that is not a load-bearing wall.

NONSTRUCTURAL WALL. All walls other than load-bearing walls or shear walls.

ON-EDGE. The orientation of a bale with its largest faces vertical, its longest dimension parallel with the wall plane, its ties on the face of the wall, and its straw lengths oriented vertically.

PIN. Metal rod, wood dowel, or bamboo, driven into, or through-tied on the surface of stacked bales for the purpose of connection or stability.

PLASTER. Gypsum, lime, cement-lime, or cement plasters, as defined in Chapter 25 and in Section N106, or clay plaster as defined in Section N106.9, or soil-cement plaster as defined in Section N106.10.

PRE-COMPRESSION. Vertical compression of stacked bales before the application of finish.

REINFORCED PLASTER. A plaster containing mesh reinforcement.

RUNNING BOND. For the purposes of this appendix, the placement of straw bales such that the head joints in successive courses are offset at least one quarter the bale length.

SHEAR WALL. A strawbale wall designed to resist lateral forces parallel to the plane of the wall in accordance with Section N105.15.

SKIN. The compilation of plaster and reinforcing, if any, applied to the surface of stacked bales.

STRUCTURAL WALL. A wall that meets the definition for a load-bearing wall or shear wall.

STACK BOND. For the purposes of this appendix, the placement of straw bales such that head joints in successive courses are vertically aligned.

STRAW. The dry stems of cereal grains after the seed heads have been removed.

STRAW BALE. A rectangular compressed block of straw, bound by ties.

STRAWBALE. The adjective form of straw bale.

STRAW-CLAY. Loose straw mixed and coated with clay slip.

TIE. A synthetic fiber, natural fiber, or metal wire used to confine a straw bale.

TRUTH WINDOW. An area of a strawbale wall left without its finish, to allow view of the straw otherwise concealed by its finish.

SECTION N103 BALES

N103.1 Types of straw. Bales shall be composed of straw from wheat, rice, rye, barley, or oat.

N103.2 Shape. Bales shall be rectangular in shape.

N103.3 Size. Bales shall have a minimum height and thickness of 12 inches (305 mm), except as otherwise permitted or required in this appendix. Bales used within a continuous wall shall be of consistent height and thickness to ensure even distribution of loads within the wall system.

N103.4 Ties. Bales shall be confined with synthetic fiber, natural fiber, or metal ties sufficient to maintain required bale density. Ties shall be at least 3 inches (76 mm) and not more than 6 inches (152 mm) from bale faces and shall be spaced not more than 12 (305 mm) inches apart. Bales with broken ties shall be retied with sufficient tension to maintain required bale density.

N103.5 Moisture content. The moisture content of bales at the time of application of the first coat of plaster or the installation of another finish shall not exceed 20 percent of the weight of the bale. The moisture content of bales shall be determined by use of a moisture meter designed for use with baled straw or hay, equipped with a probe of sufficient length to reach the center of the bale. At least 5 percent and not less than ten bales used shall be randomly selected and tested.

N103.6 Density. Bales shall have a minimum dry density of 6.5 pounds per cubic foot (92 kg/cubic meter). The dry density shall be calculated by subtracting the weight of the moisture in pounds (kg) from the actual bale weight and dividing by the volume of the bale in cubic feet (cubic meters). At least 2 percent and not less than five bales to be used shall be randomly selected and tested on site.

N103.7 Partial bales. Partial bales made after original fabrication shall be retied with ties complying with N103.4.

SECTION N104 MOISTURE CONTROL

N104.1 General. All weather-exposed bale walls and bale walls enclosing showers or steam rooms, shall be protected from water damage and moisture intrusion in accordance with this section.

N104.2 Water-resistive barriers and vapor permeance ratings. Plastered bale walls shall be constructed without any membrane barrier between straw and plaster to facilitate transpiration of moisture from the bales, and to secure a structural bond between straw and plaster, except as permitted or required elsewhere in this appendix. Where a water-resistive barrier is placed behind the exterior finish, it shall have a vapor permeance rating of at least 5 perms, except as permitted or required elsewhere in this appendix. Wall finishes shall be vapor permeable or shall have an equivalent vapor permeance rating of a Class III vapor retarder.

N104.3 Horizontal surfaces. Bale walls and other bale elements shall be provided with a moisture barrier at all weather-exposed horizontal surfaces. The moisture barrier shall be of a material and installation that will prevent water from entering the wall system. Horizontal surfaces shall include, but shall not be limited to, exterior window sills, sills at exterior niches, and buttresses. The finish material at such surfaces shall be sloped not less than 1 unit vertical in 12 units horizontal (8-percent slope) and shall drain away from all bale walls and elements. Where the moisture barrier is below the finish material, it shall be sloped not less than 1 unit vertical in 12 units horizontal (8-percent slope) and shall drain to the outside surface of the bale's vertical finish.

N104.4 Bale and concrete separation. A sheet or liquid applied Class II vapor retarder shall be installed between bales and supporting concrete or masonry. The bales shall be separated from the vapor retarder by not less than 3/4-inch (19 mm), and that space shall be filled with an insulating material such as wood or rigid insulation, or a material that allows vapor dispersion such as gravel, or other approved insulating or vapor dispersion material. Sill plates in structural walls shall comply with Table N105.14 and Table N105.15. Where bales abut a concrete or masonry wall that retains earth, a Class II vapor retarder shall be provided between such wall and the bales.

N104.5 Separation of bales and earth. Bales shall be separated from earth a minimum of 8" (203 mm).

N104.6 Separation of exterior plaster and earth. Exterior plaster applied to straw bales shall be located not less than 4 inches (102 mm) above the earth or 2 inches (51 mm) above paved areas.

N104.7 Showers walls and steam rooms. Bale walls enclosing showers or steam rooms shall be protected by a water-resistive barrier or by a Class I or Class II vapor retarder on the interior face between the finish and the bales.

SECTION N105 STRUCTURAL USE

N105.1 Scope. This section shall apply to structural strawbale walls. Sections N105.11, N105.12, and N105.16 shall also apply to nonstructural strawbale walls.

N105.2 General. An approved engineered design in accordance with Section N105 and the *International Building Code* shall be provided for buildings or portions thereof using structural strawbale walls.

N105.3 Foundations. Foundations for strawbale walls shall be of any type permitted by, and shall be designed in accordance with, the *International Building Code*.

N105.4 Building height and stories. Building height shall not exceed 35 feet and the limits contained in Table N105.13. Structural use of strawbale walls shall be permitted in multi-story buildings where:

1. Complete vertical and lateral load paths are demonstrated by an *approved* engineered design.

2. Strawbale walls interrupted by floor assemblies are designed and detailed by a registered design professional.

N105.5 Configuration of bales. Bales in structural walls shall be laid flat or on-edge and in a running bond or stack bond, except that bales in structural walls with unreinforced plasters shall be laid in a running bond only.

N105.6 Pre-compression of load-bearing strawbale walls. Prior to application of plaster, walls designed to be load-bearing shall be pre-compressed by a uniform load of not less than 100 pounds per linear foot.

N105.7 Voids and stuffing. Voids between bales in structural strawbale walls shall not exceed 4 inches (102 mm) in width, and such voids shall be stuffed with flakes of straw or straw-clay, before application of finish.

N105.8 Plaster skins. Plaster skins on structural walls shall be of any type permitted by Section N106, except gypsum plaster, and shall be in accordance with Tables N105.14 and N105.15.

N105.8.1 Straightness. Plaster skins on structural strawbale walls shall be straight, as a function of the bale wall surfaces they are applied to, as follows:

- 1. As measured across the face of a bale, straw bulges shall not protrude more than 3/4 inch (19 mm) across 2 feet (610 mm) of its height or length.
- 2. As measured across the face of a bale wall, straw bulges shall not protrude from the vertical plane of a bale wall more than 2 inches (51 mm) over 8 feet (2438 mm).
- 3. The vertical face of adjacent bales shall not be offset more than 1/2 inch (13 mm)

N105.8.2 Plaster and membranes. Structural strawbale walls shall not have a membrane between straw and plaster, or shall have attachment through the bale wall from one plaster skin to the other in accordance with an *approved* engineered design.

N105.9 Transfer of loads to and from plaster skins. Where plastered strawbale walls are used to support superimposed vertical loads, such loads shall be transferred to the plaster skins by continuous direct bearing or by an *approved* engineered design. Where plastered strawbale walls are used to resist in-plane lateral loads, such loads shall be transferred via the reinforcing mesh from the structural member or assembly above and to the sill plate in accordance with Table N105.15, or by an *approved* engineered design.

N105.10 Support of plaster skins. Plaster skins for structural strawbale walls shall be continuously supported along their bottom edge to facilitate the transfer of loads to the foundation system. Acceptable supports include, but are not limited to: a concrete or masonry stem wall, a concrete slab on grade, a wood-framed floor adequately blocked, with an *approved* engineered design, or a steel angle adequately anchored, with an *approved* engineered design. A conventional metal or plastic weep screed is not an acceptable support.

N105.11 Unrestrained wall height. Strawbale walls shall not exceed the ratios of stacked bale height to bale thickness between restraints, as stated in Section 2505.12, except where an *approved* engineered design demonstrates the wall will resist buckling from superimposed vertical loads and out-of-plane design loads.

N105.12 Resistance to out-of-plane lateral loads. Structural and non-structural strawbale walls shall be considered capable of resisting out-of-plane loads prescribed in the *International Building Code* with the following limitations and requirements, except where an *approved* engineered design is provided:

- Walls with unreinforced plasters or a non-plaster finish, and without pins in accordance with N105.12.4, or other approved means of out-of-plane bracing, shall not exceed a 5:1 ratio of stacked bale height to bale thickness.
- <u>2.</u> Clay plaster walls with reinforced plasters, or pins in accordance with N105.12 Item 4, or other approved means of out-of-plane bracing, shall not exceed the ratio indicated in Equation 24-1. Plaster reinforcement shall be any type described in Table N105.15 with staples spaced not more than 6 inches (152 mm) on center.

<u>H²/T = 65</u>

Where:

H = stacked bale height

T = bale thickness

<u>H and T are measured in feet. ($H^2/T = 19,800$ when H and T are measured in mm)</u>

3. <u>Cement, cement-lime, lime, or soil cement plaster walls with reinforced plasters, or pins in accordance with N105.12 Item 4, or other approved means of out-of-plane bracing, shall not exceed the ratio indicated in Equation 24-2. Plaster reinforcement shall be any type described in Table N105.15 with staples spaced not more than 6 inches (152 mm) on center.</u>

<u>H²/T = 80</u>

(Equation N-2)

(Equation N-1)

Where:

H = stacked bale height

T = bale thickness

<u>H and T are measured in feet. ($H^2/T = 24,400$ when H and T are measured in mm)</u>

4. Pins shall be in accordance with an *approved* engineered design or shall comply with the following:

- 4.1Pins shall be 3/8 inch (10 mm) diameter steel, 3/4 inch diameter (19 mm) wood, or1/2 inch diameter (13 mm) bamboo. Pins shall be external or internal.
- 4.2 External pins shall be installed on both sides of the wall spaced not more than 24 inches (610 mm) on center.
- 4.3 External pins shall have full lateral bearing on the sill plate and the roof- or floorbearing member, and shall be tightly tied through the wall to an opposing pin with ties spaced not more than 30 inches (762 mm) apart and not more than 15 inches (381 mm) from each end.
- 4.4 Internal pins shall be installed vertically not more than 24 inches (610 mm) on center in the center third of the bales, and shall extend from top course to bottom course.

- <u>4.5</u> The bottom course shall be similarly connected to its support and the top course shall be similarly connected to the roof- or floor-bearing member above with pins or other approved means.
- 4.6 Internal pins shall be continuous or shall overlap through not less than one bale course.

<u>N105.13</u> Design coefficients and factors for seismic design. The values given in Table N105.13 shall apply to seismic design using strawbale shear walls detailed in accordance with Table N105.15.

N105.14 Load-bearing strawbale walls. Load-bearing strawbale walls shall be in accordance with Table N105.14 as part of an *approved* engineered design to support superimposed vertical loads.

N105.15 Strawbale shear walls. Strawbale shear walls shall be in accordance with Table N105.13 as part of an *approved* engineered design to resist in-plane lateral loads. Other *approved* in-plane lateral load resisting systems shall be permitted to be used in combination with strawbale shear walls with apportionment of design loads as prescribed in the *International Building Code*.

N105.16 Connection of light-frame walls to strawbale walls. Light-frame walls perpendicular to, or at an angle to a straw bale wall assembly, shall be fastened to the bottom and top wood members of the strawbale wall in accordance with requirements for wood or cold-formed steel light-frame walls in the *International Building Code*, or the abutting stud shall be connected to alternating straw bale courses with a 1/2 inch (13mm) diameter steel, 3/4" diameter (19 mm) wood, or 5/8" diameter (16 mm) bamboo dowel, with minimum 8 inch (203 mm) penetration.

TABLE N105.13 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC-FORCE-RESISTING SYSTEMS

<u>Seismic-Force-Resisting</u> <u>System</u>		<u>Response</u> <u>Modification</u> <u>Coefficient, R¹</u>	<u>System</u> <u>Overstrength</u> <u>Factor, Omega</u> ²	<u>Deflection</u> <u>Amplificatio</u> <u>n Factor, C</u>	B	Struct Limit uildir	ural S ation ng He Limits	Syster s and ight (1 S	<u>n</u> [<u>f</u>]
						Seismic Design Category B C D E F			
					<u>B</u>	<u>C</u>	D	<u>E</u>	<u>F</u>
A. Bearing Wall Systems									
Strawbale shear walls		<u>3.5</u>	<u>3</u>	<u>3</u>	<u>25</u>	<u>25</u>	<u>15</u>	<u>15</u>	<u>15</u>
B. Building Frame Systems									
Strawbale shear walls		<u>4</u>	<u>3</u>	<u>3.5</u>	35	35	25	25	25
a. R reduces forces to a strength level	, no	t an allowable stress I	evel						

5. The tabulated value of the overstrength factor is permitted to be reduced by subtracting 0.5 for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.

TABLE N105.14

ALLOWABLE GRAVITY LOADS (LBS./FOOT) FOR PLASTERED STRAWBALE WALLS

WALL DESIGNATION	PLASTER (both sides) Thickness each side	<u>SILL</u> <u>PLATES^{b,c}</u>	ANCHOR ^c BOLTS (or other sill fastening)	<u>MESH^a</u>	STAPLES ^{e.r.g}	ALLOWABLE BEARING CAPACITY ^h (plf)
<u>A</u>	<u>Clay^l 1-1/2"</u>	<u>C</u>	<u>c</u>	<u>None</u> required ⁱ	<u>None</u> required ⁱ	<u>400</u>
B	Soil-cement ^k <u>1"</u>	<u>C</u>	<u>C</u>	<u>d</u>	<u>e,f,g</u>	<u>800</u>
<u>C</u>	Lime ¹ 7/8"	C	<u>c</u>	d	<u>e,f,g</u>	500

WALL DESIGNATION	<u>PLASTER</u> (both sides) <u>Thickness</u> each side	<u>SILL</u> PLATES ^{b.c}	ANCHOR ^c BOLTS (or other sill fastening)	<u>MESH^a</u>	STAPLES ^{e.t.g}	ALLOWABLE BEARING CAPACITY ^h (plf)
<u>D</u>	Cement-lime ^k 7/8"	<u>c</u>	<u>C</u>	<u>d</u>	<u>e,f,g</u>	800
<u>E</u>	Cement 7/8"	<u>C</u>	<u>C</u>	<u>d</u>	<u>e,f,g</u>	<u>800</u>

For SI: 1 inch=25.4mm, 1 pound per foot = 14.5939 N/m.

 Plasters shall conform with Sections N106.9 through N106.12 for makeup and thickness, with Section N105.8.1 for straightness, and with Section N105.10 for support of plaster skins.

- b. Sill plates shall support and be flush with each face of the bale wall and shall be preservative-treated where required by the International Building Code..
- c. For walls supporting gravity loads only or for non-structural walls, sill plates and fastening shall be in accordance with the requirements for wood framed walls in the *International Building Code*. See Table N105.15 for requirements for shear walls.
- d. Any metal mesh allowed by this section shall be installed throughout the plaster with minimum 4-inch laps and fastened in accordance with footnote e.
- e. Staples shall be at maximum spacing of 2-inches on center, to roof or floor bearing assembly, or as shown in an approved design in accordance with Section N105.9, and at a maximum spacing of 4-inches on center to sill plates.
- f. Staples shall be gun staples, stainless steel or electro-galvanized, 16 gauge with 1 ¼-inch legs, 7/16-inch crown; or manually driven staples, galvanized 15 gauge with 7/8-inch legs, 3/16-inch inner spread and rounded shoulder. Other staples shall be permitted to be used as designed by a registered design professional. Staples into preservative-treated wood shall be stainless steel.

g. Staples shall be firmly driven diagonally across mesh intersections at the spacing indicated.

- h. For walls with a different plaster on each side, the lower value shall be used.
- Except as necessary to transfer roof or floor loads to the plaster skins in accordance with Section N105.9.

The building official is authorized to require a cube compression test to demonstrate a minimum 100 psi compressive strength.

- K. The building official is authorized to require a compression test to demonstrate a minimum 1000 psi compressive strength.
 Lime plaster shall use hydraulic or natural hydraulic lime. The building official is authorized to require a cube compression test
- to demonstrate a minimum 600 psi compressive strength.
- m. The building official is authorized to require a cube compression test to demonstrate a minimum 1400 psi compressive strength.

ALLOWABL	E SHEAR (POUNDS PE	R FOOT) FO	OR PLASTER	RED STRAWB	ALE WALLS ^a	
DESIGNATION	<u>PLA</u>	<u>STER[≞]</u>	<u>SILL</u> PLATES ^d	ANCHOR ^ª BOLTS	<u>MESH^e</u>	<u>STAPLES^{t, g,} ^h(on center)</u>	ALLOWABLE SHEAR ^{L, <u>i.</u> k}
				(on center)		<u> </u>	(plf)
	<u>TYPE</u>	THICK-					
		(each side)					
A1	Clay ^m	1.5-in.	2 x 4	2 ft. 8 in.	None	None	60
<u>A2</u>	<u>Clay^m</u>	<u>1.5-in.</u>	<u>2 x 4</u>	<u>2 ft. 8 in.</u>	2 in. by 2 in. high-density polypropylen e	2-inches	<u>140</u>
<u>A3</u>	<u>Clay^m</u>	<u>1.5-in,</u>	<u>2 x 4</u>	<u>2 ft. 8 in.</u>	2"x2"x14ga	4-inches	<u>180</u>
<u>B</u>	<u>Soil-</u> cement ^o	<u>1-in.</u>	<u>4 x 4</u>	<u>2 ft. 0 in</u>	<u>2 in. by 2</u> in. by 14ga ^l	2-inches	<u>520</u>
<u>C1</u>	<u>Limeⁿ</u>	<u>7/8-in.</u>	<u>2 x 4</u>	<u>2 ft. 8 in.</u>	<u>17 ga.</u> woven wire	<u>3-inches</u>	<u>330</u>
<u>C2</u>	<u>Limeⁿ</u>	<u>7/8-in.</u>	<u>4 x 4</u>	<u>2 ft. 0 in.</u>	<u>2 in. by 2</u> in. by 14ga ^l	2-inches	<u>450</u>
<u>D1</u>	<u>Cement-</u> lime ^º	<u>7/8-in.</u>	<u>4 x 4</u>	<u>2 ft. 8 in</u>	<u>17 ga.</u> <u>woven wire</u>	2-inches	<u>380</u>
<u>D2</u>	<u>Cement-</u> lime ^o	<u>7/8-in.</u>	<u>4 x 4</u>	<u>2 ft. 0 in.</u>	<u>2 in. by 2</u> in. by 14ga ^l	2-inches	<u>520</u>
<u>E1</u>	<u>Cement^p</u>	<u>7/8-in.</u>	<u>4 x 4</u>	<u>2 ft. 8 in.</u>	<u>2 in. by 2</u> in. by 14 ga	2-inches	<u>540</u>
<u>E2</u>	<u>Cement^p</u>	<u>1.5-in.</u>	<u>4 x 4</u>	<u>2 ft. 0 in.</u>	<u>2 in. by 2</u> in. by 14ga ^l	2-inches	<u>680</u>

TABLE N105.15

SI: 1 inch=25.4 mm, 1 pound per foot = 14.5939 N/m

a. Bales shall be not less than 15 inches thick.

- b. Plasters shall comply with Sections N106.7 through N106.12 for makeup and thickness, with Section N105.8.1 for straightness, and with Section N105.10 for support.
- c. Sill plates shall be Douglas fir-larch or southern pine and shall be preservative-treated where required by the International Building Code. Multiply allowable shear value by .82 for other species with specific gravity of .42 or greater, or by .65 for all other species.
- d. Anchor bolts shall be 5/8-inch diameter with 2-inch by 2-inch by 3/16-inch washers, with not less than 7-inch embedment in concrete or masonry foundation. Anchor bolts or other fasteners into framed floors shall be engineered.
- e. Mesh shall run continuous vertically from sill plate to top plate, roof or floor beam, or roof or floor bearing assembly, or shall lap not less than 12-inches. Horizontal laps shall be a not less than 4-inches. Steel mesh shall be galvanized. Galvanized steel mesh shall be separated from preservative-treated wood by grade D paper, 15# roofing felt, or other approved barrier.
- f. Staples shall be gun staples, stainless steel or electro-galvanized, 16 gauge with 1 ¼-inch legs, 7/16-inch crown; or manually driven staples, galvanized 15 gauge with 7/8-inch legs, 3/16-inch inner spread and rounded shoulder. Other staples shall be permitted to be used as designed by a registered design professional. Staples into preservative-treated wood shall be stainless steel.
- g. Staples at spacing indicated are to boundary conditions, including sill plates, and top plate, roof or floor beam, or roof or floor bearing assembly,
- h. Staples shall be firmly driven diagonally across mesh intersections at spacing indicated.
- i. Values shown are for aspect ratios of 1:1 or less. Reduce values shown to 50 percent for the limit of a 2:1 aspect ratio. Linear interpolation shall be permitted for ratios between 1:1 and 2:1. The full value shown shall be used for aspect ratios greater than 1:1, where an additional layer of mesh is installed at the base of the wall to a height where the remainder of the wall has an aspect ratio of 1:1 or less, and the second layer of mesh is fastened to the sill plate with the required stapling, and the sill bolt spacing is decreased with linear interpolation between1:1 and 2:1.
- j. For walls with a plaster Type A on one side and any other plaster type on the other side, a registered design professional shall show transfer of the design lateral load into the stiffer Type B, C, D, or E plaster only, and 50% of the allowable shear value shown for that wall type shall be used.
- k. These values are permitted to be increased 40 percent for wind design.
- 16 gauge mesh shall be permitted to be used with a reduction to 0.60 of the allowable shear values shown.
- m. The building official is authorized to require a cube compression test demonstrating not less than 600 psi compressive strength.
- n. Lime plaster shall use hydraulic or natural hydraulic lime. The building official is authorized to require a cube compression test demonstrating not less than 600 psi compressive strength.
- The building official is authorized to require a cube compression test demonstrating not less than 1000 psi compressive strength.
- p. The building official is authorized to require a cube compression test demonstrating not less than 1400 psi compressive strength.

SECTION N106 FINISHES

N106.1 General. Finishes applied to strawbale walls shall be any type permitted by the *International Building Code*, and shall comply with this section and with Chapters 14 and 25 unless stated otherwise in this section.

N106.2 Purpose, and where required. Strawbale walls shall be finished so as to provide mechanical protection, fire resistance, restrict the passage of air through the bales, and protect them from weather in accordance with this appendix and the *International Building Code*.

Exception: Truth windows shall be permitted where a fire-resistive rating is not required. Weatherexposed truth windows shall be fitted with a weather-tight cover.

N106.3 Vapor retarders. Class I and Class II vapor retarders shall not be used on a strawbale walls, nor shall any other material be used that has a vapor permeance rating of less than 5 perms, except as permitted or required elsewhere in this appendix, or as *approved* and demonstrated to be necessary by a *registered design professional*.

N106.4 Plaster. Plaster applied to bales shall be of any type described in Section N106, and as required or limited in this appendix.

N106.5 Plaster and membranes. Plaster shall be applied directly to strawbale walls to facilitate transpiration of moisture from the bales, and to secure a mechanical bond between the skin and the bales, except where a membrane is allowed or required elsewhere in this appendix. Structural bale walls

shall have no membrane between straw and plaster, or shall have attachment through the bale wall from one plaster skin to the other in accordance with an approved engineered design.

N106.6 Lath and mesh for plaster. The surface of the straw bales functions as lath, and no other lath or mesh shall be required, except as required for tensile or shear strength in structural applications as required in Table N105.14, Table N105.15, or by an *approved* engineered design.

N106.7 Plaster on non-structural walls. Plaster on non-structural walls shall be in accordance with Section N106.9, N106.10, N106.11, N106.12, N106.13 or N106.14.

N106.8 Plaster on structural walls. Plaster on structural walls shall comply with Section N106.9, N106.10, N106.11, N106.12, N106.13 or N106.14. Plaster on load-bearing walls shall also comply with Table N105.14. Plaster on shear walls shall also comply with Table N105.15.

N106.9 Clay plaster. Clay plaster shall comply with Sections N106.9.1 through N106.9.6.

N106.9.1 General. Clay plaster shall be any plaster having a clay or clay soil binder. Such plaster shall contain sufficient clay to fully bind the plaster, sand or other inert granular material, and shall be permitted to contain reinforcing fibers. Reinforcing fibers shall include, but shall not be limited to, chopped straw, sisal, and animal hair.

N106.9.2 Mesh. Clay plaster shall not be required to contain reinforcing mesh except as required in Table N105.15. Where provided, mesh shall be natural fiber, corrosion-resistant metal, nylon mesh, or high-density polypropylene.

N106.9.3 Thickness and coats. Clay plaster shall be a minimum 1 inch (25 mm) thick, unless required to be thicker for structure or fire-resistance, as described elsewhere in this appendix, and shall be applied with in not less than two coats.

N106.9.4 Rain-exposed. Clay plaster, where exposed to rain, shall be finished with lime wash, linseed oil, or other *approved* erosion resistant finish.

<u>N106.9.5</u> Prohibited finish coat. Cement plaster shall not be permitted as a finish coat over clay plasters.

N106.9.6 Additives. Additives shall be permitted to increase the plaster's workability, durability, strength, or water resistance.

N106.10 Soil-cement plaster. Soil-cement plaster shall comply with Sections N106.10.1 through N106.10.3.

N106.10.1 General. Soil-cement plaster shall be comprised of soil (free of organic matter), sand, and not less than10 percent Portland cement by volume, and shall be permitted to contain reinforcing fibers.

N106.10.2 Mesh. Soil-cement plaster shall use any corrosion-resistant metal mesh permitted by the *International Building Code*, or as required in Section N105 where used on a structural wall.

N106.10.3 Thickness. Soil-cement plaster shall be not less than 1 inch (25 mm) thick.

N106.11 Gypsum plaster. Gypsum plaster shall comply with Section 2511 of the *International Building Code*. Gypsum plaster shall be limited to use on interior surfaces, and on non-structural walls, except as an interior finish coat over a structural plaster that complies with this appendix.

N106.12 Lime plaster. Lime plaster shall comply with Sections N106.12.1 and N106.12.2.

N106.12.1 General. Lime plaster is any plaster whose binder is comprised of calcium hydroxide (CaOH) including Type N or Type S hydrated lime, hydraulic lime, natural hydraulic lime, or quicklime. Hydrated lime plasters shall comply with ASTM C 206. Quicklime plasters shall comply with ASTM C 5. Lime plaster shall be permitted to be applied in 2 coats, provided that the combined thickness is at least 7/8 inch (22 mm), and each coat is not greater than 1/2 inch (13 mm) thick.

N106.12.2 On structural walls. Lime plaster on structural strawbale walls in accordance with Table N105.14 or Table N105.15 shall use hydraulic or natural hydraulic lime.

N106.13 Cement-lime plaster. Cement-lime plaster shall be plaster mixes CL or FL as described in ASTM C 926. Cement-lime plaster shall be permitted to be applied in 2 coats, provided the combined thickness is at least 7/8 inch (22 mm) thick, and each coat is not greater than 1/2 inch (13 mm) thick.

N106.14 Cement plaster. Cement plaster shall comply with Section 2512 of the *International Building Code*, except that the amount of lime in all plaster coats shall be not less than 1 part lime to 6 parts cement to allow a minimum acceptable vapor permeability. The plaster shall be permitted to be applied in 2 coats, provided the combined thickness is at least 7/8 inch (22 mm), and each coat is not greater than 1/2 inch (13 mm) thick. The combined thickness of all plaster coats shall be not more than 1 1/2 inch (38 mm) thick.

N106.15 Finishes over plaster. Other finishes, as permitted elsewhere in this section and the *International Building Code*, shall be permitted to be applied over the plaster, except as prohibited in Section N106.16.

N106.16 Prohibited plasters and finishes. Any plaster or finish with a singular or cumulative perm rating less than 5 perms shall be prohibited on straw bale walls, except where approved and demonstrated to be necessary by a *registered design professional*, or as required elsewhere in this appendix.

N106.17 Separation of wood and plaster. Where wood framing or wood sheathing occurs in strawbale walls, such wood surfaces shall be separated from exterior plaster with No. 15 asphalt felt, grade D paper, or other *approved* material in accordance with Section 1404.2 of the *International Building Code*, except where the wood is preservative-treated or naturally durable.

Exception: Exterior clay plasters shall not be required to be separated from wood.

SECTION N108 THERMAL INSULATION

N108.1 R-value. The unit R-value of a strawbale wall with bales laid flat is R-1.3 per inch, and with bales on-edge is R-2 per inch.

PART II – IBC FIRE SAFETY

SECTION N107 FIRE RESISTANCE

N107.1 Fire-resistance rating. Fire-resistance ratings for strawbale walls shall be established in accordance with Section N107.1.1 or N107.1.2, or shall be determined in accordance with Section 703.2 or 703.3 of the *International Building Code*.

N107.1.1 1-hour rated clay plastered wall. 1-hour fire-resistance-rated nonload-bearing clay plastered strawbale walls shall comply with all of the following:

1. Bales shall be laid flat or on-edge in a running bond. Gaps shall be fire-stopped with straw-clay,

2. Bales shall maintain thickness of not less than 18 inches (457 mm),

- 3. Clay plaster on each side of the wall shall be not less than 1 inch (25 mm) thick and shall be comprised of a mixture of 3 parts clay, 2 parts chopped straw, and 6 parts sand, or an alternative approved clay plaster.
- Plaster application shall be in accordance with Section N106.9 for the number and thickness of coats.

N107.1.2 2-hour rated cement plastered wall. 2-hour fire-resistance-rated nonload-bearing cement plastered strawbale walls shall comply with all of the following:

- 1. Bales shall be laid flat or on-edge in a running bond. Gaps shall be fire-stopped with straw-clay.
- Bales shall maintain a thickness of not less than 14 inches (356 mm).
- 3. 1 1/2 inch (38 mm) by 17 gauge galvanized woven wire mesh shall be attached to wood members with 1 1/2 inch (38 mm) staples at 6 inches (406 mm) on center. 9 gauge U-pins with minimum 8 inch (203 mm) legs shall be installed in the field at 18 inches (457 mm) on center.
- 4. Cement plaster on each side of the wall shall be not less than 1 inch (25 mm) thick.
- Plaster application shall be in accordance with Section N106.14 for the number and thickness of coats.

N107.2 Openings in rated walls. Openings and penetrations in bale walls required to have a fireresistance rating shall satisfy the same requirements for openings and penetrations as prescribed in the *International Building Code*.

N107.3 Clearance to fireplaces and chimneys. Strawbale surfaces adjacent to fireplaces or chimneys shall have a minimum 3/8 inch (10 mm) thick plaster coat of any type permitted by this section, and shall maintain the specified clearances to the plaster finish as required to combustibles in *International Building Code* Chapter 21, Sections 2111, 2112, and 2113, or as required by manufacturer's installation instructions, whichever is more restrictive.

N107.4 Type of construction. Buildings or portions thereof utilizing strawbale walls in accordance with this appendix shall be classified as Type V-B construction. Strawbale walls constructed in compliance with Section N107.1.1 or N107.1.2 shall be permitted wherever combustible walls of the same fire-resistance are allowed by Chapter 6 of the *International Building Code*. Strawbale walls are allowed by the *International Building Code*.

Reason: Strawbale construction has proven to be a safe, durable, resource efficient, and fully viable method of construction. However, the International Building Code does not contain a section on strawbale construction, which has been an impediment to this construction system's proper and broader use.

First practiced in Nebraska in the late 1800's, with buildings over 100 years old still in service, strawbale construction was rediscovered in the 1980's in the American southwest. Since then it has been further developed and explored, including considerable testing and research regarding structural performance (under vertical and lateral loads), moisture, fire, and its thermal and acoustic properties.

Currently only Oregon and New Mexico have adopted statewide strawbale building codes. California has legislated strawbale construction guidelines that are voluntarily adopted at the local level. In addition, nine U.S. cities or counties have adopted strawbale building codes. Three countries outside the United States – Germany, France, and Belarus - have limited strawbale building codes.

Most of the strawbale building codes that do exist are derived from the first such code, created for and adopted by Tucson / Pima County, Arizona in 1996. Much experience, testing, and research since then have proven these codes to be deficient. They are often either too restrictive, or not restrictive enough, and in some cases don't address important issues at all.

Although strawbale codes are both few and flawed, strawbale buildings are now found in 49 of the 50 United States, and strawbale construction is practiced in over 45 countries throughout the world and in every climate. There are an estimated 600-1000 strawbale buildings in California alone. The strawbale buildings in the U.S. include residences, schools, office buildings, wineries, multi-story buildings, buildings over 10,000 sq.ft in floor area, load-bearing strawbale structures, and structures in areas of high seismic risk (plastered strawbale walls are particularly resistant to earthquakes). The practice of, and the desire to utilize strawbale construction, continues to increase and promises to accelerate as we face increased pressure on our environment and natural resources.

There is great need for a comprehensive strawbale code, with full benefit of the experience and knowledge that has been gained to date about this method of construction. The following proposed Strawbale Construction appendix for the IBC was created to fulfill this need. It is based on the collective experience of the design, construction, and testing of strawbale buildings over 20 years by architects, engineers, builders, and academics throughout the U.S., Canada, and other countries throughout the world. The testing, research, and comprehensive understanding of the performance of strawbale buildings are summarized in the book

Design of Straw Bale Buildings (B.King, et al, 2006, Green Building Press). Testing, research reports, and other supporting documentation are available for viewing and download at: <u>http://www.ecobuildnetwork.org/strawbale-construction-code-supporting-documentation</u>

As lead author of the proposed appendix, and as a licensed architect for 25 years, I have been involved in the design, construction, testing, and research of strawbale buildings since 1995. In 2001 I spearheaded legislation and revisions to the current California Guidelines for Straw-Bale Structures. The proposed Strawbale Construction appendix for the IBC has benefited from numerous peer reviews by experienced, licensed design and building professionals over the course of more than five years. It would serve designers, builders, owners, inhabitants, and building officials alike in the construction and utilization of strawbale buildings.

Supporting Documentation: List of selected documents available via the above link

Load-Bearing Straw Bale Construction – A summary of worldwide testing and experience, B.King, PE Testing of Straw Bale Walls with Out-of-Plane Loads – K.Donahue, SE In-Plane Cyclic Tests of Plastered Straw Bale Wall Assemblies – C.Ash, M.Aschheim, PE, D.Mar, SE Structural Testing of Plastered Straw Bale Wall Assemblies – K.Lerner, Architect, K.Donahue, SE Seismic Design Factors and Allowable Shears for Strawbale Wall Assemblies – S. Jalali, M. Aschheim, PE Shake Table Test Video of Full Scale Straw Bale Building Specimen – D.Donovan, PE Moisture Properties of Plaster and Stucco for Strawbale Buildings – J.Straube, PE Monitoring of Hygrothermal Performance of Strawbale Walls – J.Sraube, PE, C.Schumacher ASTM E119 1-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Clay Plaster ASTM E119 2-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Cement Plaster ASTM E119 Fire Tests - Video Thermal Performance of Straw Bale Wall Systems (incl. Oak Ridge Lab test results) – N.Stone Support Letters from Licensed Practitioners: Letters from 2 Structural Engineers, 4 Civil Engineers, 1 Professor of Civil Engineering, 7 Architects

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

S315-12

PART I - INTERNATIONAL BU	ILDING COD	E - STRUC	TURAL	
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
PART II – INTERNATIONAL BU	ILDING CO	DE – FIRE S	AFETY	
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•				APPENDIX N (NEW)-S-HAMMER-AB2-15-12.doc

S316–12 Chapter 24 (NEW), 202

Proponent: Proponent: Martin Hammer, Architect, representing California Straw Building Association, Colorado Straw Bale Association, Straw Bale Construction Association-New Mexico, Ontario Straw Bale Building Coalition, Development Center for Appropriate Technology, Environmental Building Network (mfhammer@pacbell.net)

THIS IS A TWO PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IBC FIRE SAFETY COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THESE COMMITTEES

PART I – IBC STRUCTURAL

Add new text as follows:

CHAPTER 24 STRAWBALE CONSTRUCTION

SECTION 2401 GENERAL

2401.1 Scope. This Chapter shall govern the use of baled straw as a building material.

SECTION 2402 DEFINITIONS

2402.1 Definitions. The following terms are defined in Chapter 2.

BALE. CLAY. CLAY SLIP. FLAKE. LAID FLAT. MESH. ON-EDGE. PIN. **PRE-COMPRESSION. REINFORCED PLASTER.** RUNNING BOND. SHEAR WALL, STRAWBALE. SKIN. STACK BOND. STRAW. STRAW BALE. STRAWBALE. STRAW-CLAY. TIE. TRUTH WINDOW. WALL. LOAD-BEARING. WALL, NONLOAD-BEARING WALL, NONSTRUCTURAL. WALL, STRUCTURAL.

SECTION 2403 BALES

2403.1 Types of straw. Bales shall be composed of straw from wheat, rice, rye, barley, or oat.

2403.2 Shape. Bales shall be rectangular in shape.

2403.3 Size. Bales shall have a minimum height and thickness of 12 inches (305 mm), except as otherwise permitted or required in this chapter. Bales used within a continuous wall shall be of consistent height and thickness to ensure even distribution of loads within the wall system.

2403.4 Ties. Bales shall be confined with synthetic fiber, natural fiber, or metal ties sufficient to maintain required bale density. Ties shall be at least 3 inches (76 mm) and not more than 6 inches (152 mm) from bale faces and shall be spaced not more than 12 (305 mm) inches apart. Bales with broken ties shall be retied with sufficient tension to maintain required bale density.

2403.5 Moisture content. The moisture content of bales at the time of application of the first coat of plaster or the installation of another finish shall not exceed 20 percent of the weight of the bale. The moisture content of bales shall be determined by use of a moisture meter designed for use with baled straw or hay, equipped with a probe of sufficient length to reach the center of the bale. At least 5 percent and not less than ten bales used shall be randomly selected and tested.

2403.6 Density. Bales shall have a minimum dry density of 6.5 pounds per cubic foot (92 kg/cubic meter). The dry density shall be calculated by subtracting the weight of the moisture in pounds (kg) from the actual bale weight and dividing by the volume of the bale in cubic feet (cubic meters). At least 2 percent and not less than five bales to be used shall be randomly selected and tested on site.

2403.7 Partial bales. Partial bales made after original fabrication shall be retied with ties complying with 2403.4.

SECTION 2404 MOISTURE CONTROL

2404.1 General. All weather-exposed bale walls and bale walls enclosing showers or steam rooms, shall be protected from water damage and moisture intrusion in accordance with this section.

2404.2 Water-resistant barriers and vapor permeance ratings. Plastered bale walls shall be permitted to be constructed without any membrane barrier between straw and plaster to facilitate transpiration of moisture from the bales, or to secure a structural bond between straw and plaster, except as allowed or required elsewhere in this chapter. Where a water-resistant barrier is placed behind the exterior finish, it shall have a minimum vapor permeance rating of 5 perms, except as permitted or required elsewhere in this chapter to be necessary by a *registered design professional*. Wall finishes shall be vapor permeable or have an equivalent vapor permeance rating of a Class III vapor retarder

2404.3 Horizontal surfaces. Bale walls and other bale elements shall have a moisture barrier at all horizontal surfaces exposed to weather. This moisture barrier shall be of a material and installation that will prevent water from entering the wall system. Horizontal surfaces include, but are not limited to, exterior window sills, sills at exterior niches, and buttresses. The finish material at all such surfaces shall be sloped not less than 1 unit vertical in 12 units horizontal (8-percent slope) and shall drain away from all bale walls and elements. Where the moisture barrier is below the finish material, it shall be sloped not less than 1 unit vertical in 12 units horizontal (8-percent slope) and shall drain to the outside surface of the bale's vertical finish.

2404.4 Bale and concrete separation. A sheet or liquid applied Class II vapor retarder shall be installed between bales and supporting concrete or masonry. The bales shall be separated from the vapor retarder a minimum of 3/4" (19 mm), and that space shall be filled with an insulating material such

as wood or rigid insulation, or a material allowing vapor dispersion, such as gravel. Sill plates in structural walls shall comply with Table 2405.14 and Table 2405.15. Where bales abut a concrete or masonry wall that retains earth, a Class II vapor retarder shall be provided between such wall and the bales.

2404.5 Separation of bales and earth. Bales shall be separated from earth a minimum of 8" (203 mm).

2404.6 Separation of exterior plaster and earth. Exterior plaster applied to straw bales shall be a minimum of 4 inches (102 mm) above the earth or 2 inches (51 mm) above paved areas.

2404.7 Shower walls, steam rooms. Bale walls enclosing showers, tub shower combinations, or steam rooms shall be protected by a water-resistive barrier or by a Class I or Class II vapor retarder.

SECTION 2405 STRUCTURAL USE

2405.1 Scope. This section shall apply to structural strawbale walls. Sections 2405.11, 2405.12, and 2405.16 shall also apply to nonstructural strawbale walls.

2405.2 General. An approved engineered design in accordance with Section 2405 and the *International Building Code* shall be provided for buildings or portions thereof using structural strawbale walls.

2405.3 Foundations. Foundations for strawbale walls shall be any type permitted by, and shall be designed in accordance with, the *International Building Code*.

2405.4 Building height and stories. Building height shall not exceed 35 feet and the limits contained in Table 2405.13. Structural use of strawbale walls shall be permitted in multi-story buildings where:

- 1. Complete vertical and lateral load paths are demonstrated by an *approved* engineered design.
- 2. Strawbale walls interrupted by floor assemblies are designed and detailed by a registered design professional.

2405.5 Configuration of bales. Bales in structural walls shall be laid flat or on-edge and in a running bond or stack bond, except that bales in structural walls with unreinforced plasters shall be laid in a running bond only.

2405.6 Pre-compression of load-bearing strawbale walls. Prior to application of plaster, walls designed to be load-bearing shall be pre-compressed by a uniform load of not less than 100 pounds per linear foot.

2405.7 Voids and stuffing. Voids between bales in structural strawbale walls shall not exceed 4 inches (102 mm) in width, and such voids shall be stuffed with flakes of straw or straw-clay, before application of finish.

2405.8 Plaster skins. Plaster skins on structural walls shall be of any type permitted by Section 2406, except gypsum plaster, and shall be in accordance with Tables 2405.14 and 2405.15.

2405.8.1 Straightness. Plaster skins on structural strawbale walls shall be straight, as a function of the bale wall surfaces they are applied to, as follows:

- 1. As measured across the face of a bale, straw bulges shall not protrude more than 3/4 inch (19 mm) across 2 feet (610 mm) of its height or length.
- 2. As measured across the face of a bale wall, straw bulges shall not protrude from the vertical plane of a bale wall more than 2 inches (51 mm) over 8 feet (2438 mm).
- 3. The vertical face of adjacent bales shall not be offset more than 1/2 inch (13 mm)

Where:

 $H^{2}/T = 80$

accordance with 2405.12 Item 4, or other approved means of out-of-plane bracing, shall not exceed the ratio indicated in Equation 24-2. Plaster reinforcement shall be any type described in Table 2405.15 with staples spaced not more than 6 inches (152 mm) on center.

2405.11 Unrestrained wall height. Strawbale walls shall not exceed the ratios of stacked bale height to bale thickness between restraints, as stated in Section 2505.12, except where an approved engineered design demonstrates the wall will resist buckling from superimposed vertical loads and out-of-plane design loads.

2405.8.2 Plaster and membranes. Structural strawbale walls shall not have a membrane between straw

and plaster, or shall have attachment through the bale wall from one plaster skin to the other in

2405.9 Transfer of loads to and from plaster skins. Where plastered strawbale walls are used to support superimposed vertical loads, such loads shall be transferred to the plaster skins by continuous direct bearing or by an approved engineered design. Where plastered strawbale walls are used to resist in-plane lateral loads, such loads shall be transferred via the reinforcing mesh from the structural member or assembly above and to the sill plate in accordance with Table 2405.15, or in accordance with an

2405.10 Support of plaster skins. Plaster skins for structural strawbale walls shall be continuously supported along their bottom edge to facilitate the transfer of loads to the foundation system. Supports shall include, but shall not be limited to: a concrete or masonry stem wall, a concrete slab on grade, a wood-framed floor adequately blocked, with an *approved* engineered design, or a steel angle adequately anchored, with an *approved* engineered design. A conventional metal or plastic weep screed is not an

2405.12 Resistance to out-of-plane lateral loads. Structural and non-structural strawbale walls shall be considered capable of resisting the out-of-plane loads prescribed in the International Building Code with the following limitations and requirements, except where an *approved* engineered design is provided:

- 1. Walls with unreinforced plasters or a non-plaster finish, and without pins in accordance with 2405.12.4, or other approved means of out-of-plane bracing, shall not exceed a 5:1 ratio of stacked bale height to bale thickness.
- 2. Clay plaster walls with reinforced plasters, or pins in accordance with 2405.12 Item 4, or other approved means of out-of-plane bracing, shall not exceed the ratio indicated in Equation 24-1. Plaster reinforcement shall be any type described in Table 2405.15 with staples spaced not more than 6 inches (152 mm) on center.

H = stacked bale height

accordance with an approved engineered design.

approved engineered design.

acceptable support.

= bale thickness

H and T are measured in feet. ($H^2/T = 19,800$ when H and T are measured in mm)

3. Cement, cement-lime, lime, or soil cement plaster walls with reinforced plasters, or pins in

(Equation 24-2)

(Equation 24-1)

S632

= stacked bale height = bale thickness

ICC PUBLIC HEARING ::: April - May 2012

<u>H and T are measured in feet. ($H^2/T = 24,400$ when H and T are measured in mm)</u>

$H^{2}/T = 65$

Where:

4. Pins shall be in accordance with an approved engineered design, or shall comply with the following: Pins shall be 3/8 inch (10 mm) diameter steel, 3/4 inch diameter (19 mm) wood, or 1/2 inch diameter (13 mm) bamboo. Pins shall be external or internal. External pins shall be installed on both sides of the wall spaced not more than 24 inches (610 mm) on center. External pins shall have full lateral bearing on the sill plate and the roof- or floor-bearing member, and shall be tightly tied through the wall to an opposing pin with ties spaced not more than 30 inches (762 mm) apart and not more than 15 inches (381 mm) from each end. Internal pins shall be installed vertically not more than 24 inches (610 mm) on center in the center third of the bales, and shall extend from top course to bottom course. The bottom course shall be similarly connected to its support and the top course shall be similarly connected to the roof- or floor-bearing member above with pins or other approved means. Internal pins shall be continuous or shall overlap through not less than one bale course.

2405.13 Design coefficients and factors for seismic design. The values given in Table 2405.13 shall apply to seismic design using strawbale shear walls detailed in accordance with Table 2405.15.

2405.14 Load-bearing strawbale walls. Load-bearing strawbale walls shall be in accordance with Table 2405.14 as part of an *approved* engineered design to support superimposed vertical loads.

2405.15 Strawbale shear walls. Strawbale shear walls shall be in accordance with Table 2405.13 as part of an *approved* engineered design to resist in-plane lateral loads. Other *approved* in-plane lateral load resisting systems shall be permitted to be used in combination with strawbale shear walls with apportionment of design loads as prescribed in the *International Building Code*.

2405.16 Connection of light-frame walls to strawbale walls. Light-frame walls perpendicular to, or at an angle to a straw bale wall assembly, shall be fastened to the bottom and top wood members of the strawbale wall in accordance with requirements for wood or cold-formed steel light-frame walls in the *International Building Code*, or the abutting stud shall be connected to alternating straw bale courses with a 1/2 inch (13mm) diameter steel, 3/4" diameter (19 mm) wood, or 5/8" diameter (16 mm) bamboo dowel, with minimum 8 inch (203 mm) penetration.

TABLE 2405.13 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC-FORCE-RESISTING SYSTEMS

<u>Seismic-Force-Resisting</u> <u>System</u>		<u>Response</u> <u>Modification</u> <u>Coefficient, R¹</u>	<u>System</u> <u>Overstrength</u> Factor, Omega ²	<u>Deflection</u> <u>Amplificatio</u> <u>n Factor, C</u>	B	Struct Limit Suildir	ural S ation ng He Limits	Syster s and ight (1	<u>n</u> [<u>ft]</u>
						<u>Seisı</u> C	<u>nic D</u> atego	<u>esign</u> ry	l
					B	<u>C</u>	D	<u>E</u>	<u>F</u>
A. Bearing Wall Systems									
Strawbale shear walls		<u>3.5</u>	<u>3</u>	<u>3</u>	<u>25</u>	<u>25</u>	<u>15</u>	<u>15</u>	<u>15</u>
B. Building Frame Systems									
Strawbale shear walls		4	<u>3</u>	<u>3.5</u>	<u>35</u>	<u>35</u>	25	25	25
^a R reduces forces to a strength level, no	ot an	allowable stress leve	1						

^b The tabulated value of the overstrength factor is permitted to be reduced by subtracting 0.5 for structures with flexible diaphragms, but shall not be taken s less than 2.0 for any structure.

TABLE 2405.14 ALLOWABLE GRAVITY LOADS (LBS./FOOT) FOR PLASTERED STRAWBALE WALLS

WALL DESIGNATION	PLASTER (both sides) Thickness each side	<u>SILL</u> <u>PLATES^{b.c}</u>	ANCHOR [©] BOLTS (or other sill fastening)	<u>MESH^a</u>	STAPLES ^{e.t.g}	ALLOWABLE BEARING CAPACITY ^h (plf)
A	<u>Clay^l 1-1/2"</u>	<u>C</u>	<u>C</u>	<u>None</u> required ⁱ	<u>None</u> required ⁱ	<u>400</u>
<u>B</u>	Soil-cement ^k <u>1"</u>	<u>C</u>	<u>C</u>	<u>d</u>	<u>e,f,g</u>	<u>800</u>
C	Lime ^l 7/8"	<u>c</u>	<u>c</u>	d	<u>e,f,g</u>	500
<u>D</u>	Cement-lime ^k <u>7/8"</u>	<u>C</u>	<u>C</u>	<u>d</u>	<u>e,f,g</u>	<u>800</u>
<u>E</u>	Cement 7/8"	<u>c</u>	<u>C</u>	<u>d</u>	<u>e,f,g</u>	<u>800</u>

For SI: 1 inch=25.4mm, 1 pound per foot = 14.5939 N/m.

a. Plasters shall conform with Sections 2406.9 through 2406.12 for makeup and thickness, with Section 2405.8.1 for straightness, and with Section 2405.10 for support of plaster skins. Specified minimum plaster thicknesses are applied on each face of the wall.

b. Sill plates shall support and be flush with each face of the bale wall and shall be preservative-treated where required by the International Building Code..

<u>c.</u> For walls supporting gravity loads only or for non-structural walls, sill plates and fastening shall be in accordance with the requirements for wood framed walls in the *International Building Code*. See Table 2405.15 for requirements for shear walls.
 <u>d.</u> Any metal mesh allowed by this section shall be installed throughout the plaster with minimum 4-inch laps and fastened per

loads into the plaster skins in accordance with Section 2405.9, and at a maximum spacing of 4-inches o.c. to sill plates. f. Staples shall be gun staples, stainless steel or electro-galvanized, 16 gauge with 1 ¼-inch legs, 7/16-inch crown; or manually

driven staples, galvanized 15 gauge with 7/8-inch legs, 3/16-inch inner spread and rounded shoulder. Other staples shall be permitted to be used as designed by a registered design professional. Staples into preservative-treated wood shall be stainless steel.

g. Staples shall be firmly driven diagonally across mesh intersections at the spacing indicated.

h. For walls with a different plaster on each side, the lower value shall be used.

Except as necessary to transfer roof or floor loads to the plaster skins in accordance with Section 2405.9.

j. The building official is authorized to require a cube compression test to demonstrate a minimum 100 psi compressive strength.

k. The building official is authorized to require a compression test to demonstrate a minimum 1000 psi compressive strength.

Lime plaster shall use hydraulic or natural hydraulic lime. The building official is authorized to require a cube compression test m. to demonstrate a minimum 600 psi compressive strength.

 The building official is authorized to require a cube compression test to demonstrate a minimum 1400 psi compressive strength.

TABLE 2405.15 ALLOWABLE SHEAR (POUNDS PER FOOT) FOR PLASTERED STRAWBALE WALLS^a

DESIGNATION	<u>PLA</u>	<u>STER[®]</u>	<u>SILL</u> PLATES [₫]	ANCHOR ^d BOLTS	<u>MESH^e</u>	<u>STAPLES^{r. g.} ^h(on center)</u>	ALLOWABLE SHEAR ^{I, J, K}
	<u>TYPE</u>	<u>THICK-</u> <u>NESS</u> (each side)		(on center)			())))
<u>A1</u>	<u>Clay^m</u>	<u>1.5-in.</u>	<u>2 x 4</u>	<u>2 ft. 8 in.</u>	None	None	<u>60</u>
<u>A2</u>	<u>Clay^m</u>	<u>1.5-in."</u>	<u>2 x 4</u>	<u>2 ft. 8 in.</u>	2 in. by 2 in. high-density polypropylen <u>e</u>	2-inches	<u>140</u>
<u>A3</u>	<u>Clay^m</u>	1.5-in.	<u>2 x 4</u>	<u>2 ft. 8 in.</u>	2"x2"x14ga	4-inches	<u>180</u>
B	<u>Soil-</u> <u>cement^o</u>	<u>1-in.</u>	<u>4 x 4</u>	<u>2 ft. 0 in</u>	<u>2 in. by 2</u> in. by 14ga ^l	2-inches	<u>520</u>
<u>C1</u>	<u>Limeⁿ</u>	<u>7/8-in.</u>	<u>2 x 4</u>	<u>2 ft. 8 in.</u>	<u>17 ga.</u> woven wire	<u>3-inches</u>	<u>260</u>
<u>C2</u>	<u>Limeⁿ</u>	<u>7/8-in.</u>	<u>4 x 4</u>	<u>2 ft. 0 in.</u>	<u>2 in. by 2</u> in. by 14ga ^l	2-inches	<u>450</u>
<u>D1</u>	Cement-	<u>7/8-in.</u>	<u>4 x 4</u>	<u>2 ft. 8 in</u>	<u>17 ga.</u>	2-inches	<u>380</u>

<u>footnote e.</u> <u>e.</u> Staples shall be at maximum spacing of 2-inches o.c., to roof or floor bearing assembly, or as shown necessary to transfer

DESIGNATION	<u>PLA</u>	<u>STER^b</u>	<u>SILL</u> PLATES [₫]	ANCHOR ^ª BOLTS (on center)	<u>MESH</u> ⁰	<u>STAPLES^{r. g.} ^h(on center)</u>	ALLOWABLE SHEAR ^{I, J, k} (plf)
	<u>TYPE</u>	<u>THICK-</u> <u>NESS</u> (each side)					
	<u>lime^o</u>				woven wire		
<u>D2</u>	<u>Cement-</u> lime ^o	<u>7/8-in.</u>	<u>4 x 4</u>	<u>2 ft. 0 in.</u>	<u>2 in. by 2</u> in. by 14ga ^l	2-inches	<u>520</u>
<u>E1</u>	<u>Cement^p</u>	<u>7/8-in.</u>	<u>4 x 4</u>	<u>2 ft. 8 in.</u>	<u>2 in. by 2</u> in. by 14 ga ^l	2-inches	<u>540</u>
<u>E2</u>	<u>Cement^p</u>	<u>1.5-in.</u>	<u>4 x 4</u>	<u>2 ft. 0 in.</u>	<u>2 in. by 2</u> in. by 14ga ^l	2-inches	<u>680</u>

SI: 1 inch=25.4 mm, 1 pound per foot = 14.5939 N/m

a. Bales shall be a minimum of 15 inches thick.

b. Plasters shall comply with Sections 2406.7 through 2406.12 for makeup and thickness, with Section 2405.8.1 for straightness, and with Section 2405.10 for support.

c. Sill plates shall be Douglas fir-larch or southern pine and shall be preservative-treated where required by the International Building Code. Multiply allowable shear value by .82 for other species with specific gravity of .42 or greater, or by .65 for all other species.

d. Anchor bolts shall be 5/8-inch diameter with 2-inch by 2-inch by 3/16-inch washers, with not less than 7-inch embedment in concrete or masonry foundation. Anchor bolts or other fasteners into framed floors shall be engineered.

e. Mesh shall run continuous vertically from sill plate to top plate, roof or floor beam, or roof or floor bearing assembly, or shall lap a not less than 12-inches. Horizontal laps shall be not less than 4-inches. Steel mesh shall be galvanized. Galvanized steel mesh shall be separated from preservative-treated wood by grade D paper, 15# roofing felt, or other approved barrier.

f. Staples shall be gun staples, stainless steel or electro-galvanized, 16 gauge with 1 ¼-inch legs, 7/16-inch crown; or manually driven staples, galvanized 15 gauge with 7/8-inch legs, 3/16-inch inner spread and rounded shoulder. Other staples shall be permitted to be used as designed by a registered design professional. Staples into preservative-treated wood shall be stainless steel.

g. Staples at spacing indicated are to boundary conditions, including sill plates, and top plate, roof or floor beam, or roof or floor bearing assembly,

h. Staples shall be firmly driven diagonally across mesh intersections at spacing indicated.

i. Values shown are for aspect ratios of 1:1 or less. Reduce values shown to 50% for the limit of a 2:1 aspect ratio. Linear interpolation shall be permitted for ratios between 1:1 and 2:1. The full value shown shall be used for aspect ratios greater than 1:1, where an additional layer of mesh is installed at the base of the wall to a height where the remainder of the wall has an aspect ratio of 1:1 or less, and the second layer of mesh is fastened to the sill plate with the required stapling, and the sill bolt spacing is decreased with linear interpolation between1:1 and 2:1.

j. For walls with a plaster Type A on one side and any other plaster type on the other side, a registered design professional shall show transfer of the design lateral load into the stiffer Type B, C, D, or E plaster only, and 50% of the allowable shear value shown for that wall type shall be used.

k. These values are permitted to be increased 40 percent for wind design.

I. 16 gauge mesh shall be permitted to be used with a reduction to 0.60 of the allowable shear values shown.

m. The building official is authorized to require a cube compression test demonstrating not less than 600 psi compressive strength.

n. Lime plaster shall use hydraulic or natural hydraulic lime. The building official is authorized to require a cube compression test demonstrating not less than 600 psi compressive strength.

 <u>o.</u> The building official is authorized to require a cube compression test demonstrating not less than 1000 psi compressive strength.

p. The building official is authorized to require a cube compression test demonstrating not less than 1400 psi compressive strength.

SECTION 2406 FINISHES

2406.1 General. Finishes applied to strawbale walls shall be any type permitted by the *International Building Code*, and shall comply with this section and with Chapters 14 and 25 unless stated otherwise in this section.

2406.2 Purpose, and where required. Strawbale walls shall be finished so as to provide mechanical protection, fire resistance, restrict the passage of air through the bales, and protect them from weather in accordance with this chapter and the *International Building Code*.

Exception: Truth windows are permitted where a fire-resistive rating is not required. Weatherexposed truth windows shall be fitted with a weather-tight cover.

2406.3 Vapor retarders. Class I and Class II vapor retarders shall not be used on a strawbale walls, nor shall any other material be used that has a vapor permeance rating less than 5 perms, except as permitted or required elsewhere in this chapter, or as *approved* and demonstrated to be necessary by a registered design professional.

2406.4 Plaster. Plaster applied to bales shall be of any type described in Section 2406, and as required or limited in this chapter.

2406.5 Plaster and membranes. Plaster shall be applied directly to strawbale walls to facilitate transpiration of moisture from the bales, and to secure a mechanical bond between the skin and the bales, except where a membrane is allowed or required elsewhere in this chapter. Structural bale walls shall have no membrane between straw and plaster, or shall have attachment through the bale wall from one plaster skin to the other in accordance with an approved engineered design.

2406.6 Lath and mesh for plaster. The surface of the straw bales functions as lath, and no other lath or mesh shall be required, except as required for tensile or shear strength in structural applications as required in Table 2405.14, Table 2405.15, or by an *approved* engineered design.

2406.7 Plaster on non-structural walls. Plaster on non-structural walls shall be in accordance with Section 2406.9, 2406.10, 2406.11, 2406.12, 2406.13 or 2406.14.

2406.8 Plaster on structural walls. Plaster on structural walls shall comply with Section 2406.9, 2406.10, 2406.11, 2406.12, 2406.13 or 2406.14. Plaster on load-bearing walls shall also comply with Table 2405.14. Plaster on shear walls shall also comply with Table 2405.15.

2406.9 Clay plaster. Clay plaster shall comply with Sections 2406.9.1 through 2406.9.6.

2406.9.1 General. Clay plaster shall be any plaster having a clay or clay soil binder. Such plaster shall contain sufficient clay to fully bind the plaster, sand or other inert granular material, and shall be permitted to contain reinforcing fibers. Reinforcing fibers shall include, but shall not be limited to, chopped straw, sisal, and animal hair.

2406.9.2 Mesh. Clay plaster shall not be required to contain reinforcing mesh except as required in Table 2405.15. Clay plaster shall be permitted to contain natural fiber mesh, corrosion-resistant metal mesh, nylon mesh, or high-density polypropylene mesh.

2406.9.3 Thickness and coats. Clay plaster shall be not less than 1 inch (25 mm) thick, unless required to be thicker for structure or fire-resistance, as described elsewhere in this chapter, and shall be applied with a minimum of two coats.

2406.9.4 Rain-exposed. Clay plaster, where exposed to rain, shall be finished with lime wash, linseed oil, or other *approved* erosion resistant finish.

2406.9.5 Prohibited finish coat. Cement plaster is prohibited as a finish coat over clay plasters.

2406.9.6 Additives. Additives shall be permitted to increase the plaster's workability, durability, strength, or water resistance.

2406.10 Soil-cement plaster. Soil-cement plaster shall comply with Sections 2406.10.1 through 2406.10.3.

2406.10.1 General. Soil-cement plaster shall be comprised of soil (free of organic matter), sand, and not less than 10 percent Portland cement by volume, and shall be permitted to contain reinforcing fibers.

2406.10.2 Mesh. Soil-cement plaster shall use any corrosion-resistant metal mesh permitted by the *International Building Code*, or as required in Section 2405 where used on a structural wall.

2406.10.3 Thickness. Soil-cement plaster shall be a minimum of 1 inch (25 mm) thick.

2406.11 Gypsum plaster. Gypsum plaster shall comply with Section 2511 of the *International Building Code*. Gypsum plaster shall be limited to use on interior surfaces, and on non-structural walls, except as an interior finish coat over a structural plaster that complies with this chapter.

2406.12 Lime plaster. Lime plaster shall comply with Sections 2406.12.1 and 2406.12.2.

2406.12.1 General. Lime plaster is any plaster whose binder is comprised of calcium hydroxide (CaOH) including Type N or Type S hydrated lime, hydraulic lime, natural hydraulic lime, or quicklime. Hydrated lime plasters shall comply with ASTM C 206. Quicklime plasters shall comply with ASTM C 5. Lime plaster shall be permitted to be applied in 2 coats, provided that the combined thickness is at least 7/8 inch (22 mm), and each coat is no greater than 1/2 inch (13 mm).

2406.12.2 On structural walls. Lime plaster on structural strawbale walls in accordance with Table 2405.14 or Table 2405.15 shall use hydraulic or natural hydraulic lime.

2406.13 Cement-lime plaster. Cement-lime plaster shall be plaster mixes CL or FL as described in ASTM C 926. Cement-lime plaster shall be permitted to be applied in 2 coats, provided the combined thickness is at least 7/8 inch (22 mm) thick, and each coat is not greater than 1/2 inch (13 mm) thick.

2406.14 Cement plaster. Cement plaster shall comply with Section 2512 of the *International Building Code*, except that the amount of lime in all plaster coats shall be not less than 1 part lime to 6 parts cement to allow a minimum acceptable vapor permeability. The plaster shall be permitted to be applied in 2 coats, provided the combined thickness is at least 7/8 inch (22 mm), and each coat is not greater than 1/2 inch (13 mm) thick. The combined thickness of all plaster coats shall be not more than 1 1/2 inch (38 mm) thick.

2406.15 Finishes over plaster. Other finishes, as permitted elsewhere in this section and the *International Building Code*, shall be permitted to be applied over the plaster, except as prohibited in Section 2406.16.

2406.16 Prohibited plasters and finishes. Any plaster or finish with a singular or cumulative perm rating less than 5 perms shall be prohibited on straw bale walls, except where approved and demonstrated to be necessary by a *registered design professional*, or as required elsewhere in this chapter.

2406.17 Separation of wood and plaster. Where wood framing or wood sheathing occurs in strawbale walls, such wood surfaces shall be separated from exterior plaster with No. 15 asphalt felt, grade D paper, or other *approved* material in accordance with Section 1404.2 of the *International Building Code*, except where the wood is preservative-treated or naturally durable.

Exception: Exterior clay plasters shall not be required to be separated from wood.

SECTION 2408 THERMAL INSULATION

2408.1 R-value. The unit R-value of a strawbale wall with bales laid flat is R-1.3 per inch, and with bales on-edge is R-2 per inch.

Revise as follows:

SECTION 202 DEFINITIONS

BALE. Equivalent to straw bale.

CLAY. Inorganic soil with particle sizes less than 0.00008 in. (0.002 mm) having the characteristics of high to very high dry strength and medium to high plasticity.

CLAY SLIP. A suspension of clay particles in water.

FLAKE. An intact section of compressed straw removed from an untied bale.

LAID FLAT. The orientation of a bale with its largest faces horizontal, its longest dimension parallel with the wall plane, its ties concealed in the unfinished wall and its straw lengths oriented across the thickness of the wall.

MESH. An openwork fabric of linked strands of metal, plastic, or natural or synthetic fiber, embedded in plaster to provide tensile reinforcement or bonding.

ON-EDGE. The orientation of a bale with its largest faces vertical, its longest dimension parallel with the wall plane, its ties on the face of the wall, and its straw lengths oriented vertically.

PIN. Metal rod, wood dowel, or bamboo, driven into or through-tied on the surface of stacked bales for purpose of connection or stability.

PRE-COMPRESSION. Vertical compression of stacked bales before application of finish.

REINFORCED PLASTER. A plaster containing mesh reinforcement.

RUNNING BOND. The placement of *masonry units* <u>or straw bales</u> such that the *head joints* in successive courses are offset at least one-quarter the unit <u>or bale</u> length.

STRAWBALE SHEAR WALL. A strawbale wall designed to resist lateral forces parallel to the plane of the wall in accordance with Section 2405.15.

SKIN. The compilation of plaster and reinforcing, if any, applied to the surface of stacked straw bales.

STACK BOND. The placement of *masonry units* <u>or straw bales</u> such that *head joints* in successive courses are vertically aligned. For the purposes of this code, requirements of stack bond shall apply to *masonry* <u>or straw bales</u> laid in other than a *running bond*.

STRAW. The dry stems of cereal grains after the seed heads have been removed.

STRAW BALE. A rectangular compressed block of straw, bound by ties.

STRAWBALE. The adjective form of straw bale.

STRAW-CLAY. Loose straw mixed and coated with clay slip.

TIE. A synthetic fiber, natural fiber, or metal wire used to confine a straw bale.

TRUTH WINDOW. An area of a strawbale wall left without its finish, to allow view of the straw otherwise concealed by its finish.

WALL, LOAD-BEARING. Any wall meeting either one of the following classifications:

- 1. Any metal or wood stud wall that supports more than 100 pounds per linear foo (1459 N/m) of vertical load in addition to its own weight.
- 2. Any masonry or concrete wall that supports more than 200 pound per linear foot (2919 N/m) of vertical load in addition to its own weight.
- 3. Any strawbale wall that supports more than 100 lb/linear ft (1,459 N/m) of vertical load in addition to its own weight.

WALL, NONSTRUCTURAL. All walls other than load-bearing walls or shear walls. WALL, STRUCTURAL. A wall that meets the definition for a load-bearing wall or shear wall.

PART II – IBC FIRE SAFETY

SECTION 2407 FIRE RESISTANCE

2407.1 Fire-resistance rating. Fire-resistance ratings for strawbale walls shall be established in accordance with Section 2407.1.1 or 2407.1.2, or shall be determined in accordance with Section 703.2 or 703.3 of the *International Building Code*.

2407.1.1 1-hour rated clay plastered wall. 1-hour fire-resistance-rated nonload-bearing clay plastered strawbale walls shall comply with all of the following:

- 1. Bales shall be laid flat or on-edge in a running bond. Gaps shall be fire-stopped with straw-clay,
- 2. Bales shall maintain a thickness of not less than 18 inches (457 mm),
- 3. Clay plaster on each side of the wall shall be not less than 1 inch (25 mm) thick and shall be comprised of a mixture of 3 parts clay, 2 parts chopped straw, and 6 parts sand, or an alternative approved clay plaster.
- 4. Plaster application shall be in accordance with Section 2406.9 for the number and thickness of coats.

2407.1.2 2-hour rated cement plastered wall. 2-hour fire-resistance-rated nonload-bearing cement plastered strawbale walls shall comply with all of the following:

- 1. Bales shall be laid flat or on-edge in a running bond. Gaps shall be fire-stopped with straw-clay.
- 2. Bales shall maintain a minimum thickness of 14 inches (356 mm).
- 3. 1 1/2 inch (38 mm) by 17 gauge galvanized woven wire mesh shall be attached to wood members with 1 1/2 inch (38 mm) staples at 6 inches (406 mm) on center. 9 gauge U-pins with minimum 8 inch (203 mm) legs shall be installed in the field at 18 inches (457 mm) on center.
- 4. Cement plaster on each side of the wall shall be not less than 1 inch (25 mm) thick.
- 5. Plaster application shall be in accordance with Section 2406.14 for the number and thickness of coats.

2407.2 Openings in rated walls. Openings and penetrations in bale walls required to have a fireresistance rating, shall satisfy the same requirements for openings and penetrations as prescribed in the *International Building Code*.

2407.3 Clearance to fireplaces and chimneys. Strawbale surfaces adjacent to fireplaces or chimneys shall have a minimum 3/8 inch (10 mm) thick plaster coat of any type permitted by this section, and shall maintain the specified clearances to the plaster finish as required to combustibles in *International Building Code* Chapter 21, Sections 2111, 2112, and 2113, or as required by manufacturer's installation instructions, whichever is more restrictive.

2407.4 Type of construction. Buildings or portions thereof utilizing strawbale walls in accordance with this chapter shall be classified as Type V-B construction. Strawbale walls constructed in compliance with Section 2407.1.1 or 2407.1.2 shall be permitted wherever combustible walls of the same fire-resistance are allowed by Chapter 6 of the *International Building Code*. Strawbale walls are allowed by the *International Building Code*. Strawbale walls are allowed by the *International Building Code*.

Reason: Strawbale construction has proven to be a safe, durable, resource efficient, and fully viable method of construction. However, the International Building Code does not contain a section on strawbale construction, which has been an impediment to this construction system's proper and broader use.

First practiced in Nebraska in the late 1800's, with buildings over 100 years old still in service, strawbale construction was rediscovered in the 1980's in the American southwest. Since then it has been further developed and explored, including considerable testing and research regarding structural performance (under vertical and lateral loads), moisture, fire, and its thermal and acoustic properties.

Currently only Oregon and New Mexico have adopted statewide strawbale building codes. California has legislated strawbale construction guidelines that are voluntarily adopted at the local level. In addition, nine U.S. cities or counties have adopted

strawbale building codes. Three countries outside the United States – Germany, France, and Belarus - have limited strawbale building codes.

Most of the strawbale building codes that do exist are derived from the first such code, created for and adopted by Tucson / Pima County, Arizona in 1996. Much experience, testing, and research since then have proven these codes to be deficient. They are often either too restrictive, or not restrictive enough, and in some cases don't address important issues at all.

Although strawbale codes are both few and flawed, strawbale buildings are now found in 49 of the 50 United States, and strawbale construction is practiced in over 45 countries throughout the world and in every climate. There are an estimated 600-1000 strawbale buildings in California alone. The strawbale buildings in the U.S. include residences, schools, office buildings, wineries, multi-story buildings, buildings over 10,000 sq.ft in floor area, load-bearing strawbale structures, and structures in areas of high seismic risk (plastered strawbale walls are particularly resistant to earthquakes). The practice of, and the desire to utilize strawbale construction, continues to increase and promises to accelerate as we face increased pressure on our environment and natural resources

There is great need for a comprehensive strawbale code, with full benefit of the experience and knowledge that has been gained to date about this method of construction. The following proposed Strawbale Construction chapter for the IBC was created to fulfill this need. It is based on the collective experience of the design, construction, and testing of strawbale buildings over 20 years by architects, engineers, builders, and academics throughout the U.S., Canada, and other countries throughout the world. The testing, research, and comprehensive understanding of the performance of strawbale buildings are summarized in the book *Design of Straw Bale Buildings* (B.King, et al, 2006, Green Building Press). Testing, research reports, and other supporting documentation are available for viewing and download at: <u>http://www.ecobuildnetwork.org/strawbale-construction-code-supporting-</u>documentation

As lead author of the proposed chapter, and as a licensed architect for 25 years, I have been involved in the design, construction, testing, and research of strawbale buildings since 1995. In 2001 I spearheaded legislation and revisions to the current California Guidelines for Straw-Bale Structures. The proposed Strawbale Construction chapter for the IBC has benefited from numerous peer reviews by experienced, licensed design and building professionals over the course of more than five years. It would serve designers, builders, owners, inhabitants, and building officials alike in the construction and utilization of strawbale buildings.

Supporting Documentation: List of selected documents available via the above link

Load-Bearing Straw Bale Construction – A summary of worldwide testing and experience, B.King, PE Testing of Straw Bale Walls with Out-of-Plane Loads – K.Donahue, SE In-Plane Cyclic Tests of Plastered Straw Bale Wall Assemblies – C.Ash, M.Aschheim, PE, D.Mar, SE Structural Testing of Plastered Straw Bale Wall Assemblies – K.Lerner, Architect, K.Donahue, SE Seismic Design Factors and Allowable Shears for Strawbale Wall Assemblies – S. Jalali, M. Aschheim, PE Shake Table Test Video of Full Scale Straw Bale Building Specimen – D.Donovan, PE Moisture Properties of Plaster and Stucco for Strawbale Buildings – J.Straube, PE Monitoring of Hygrothermal Performance of Strawbale Walls – J.Sraube, PE, C.Schumacher ASTM E119 1-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Clay Plaster ASTM E119 2-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Cement Plaster ASTM E119 Fire Tests - Video Thermal Performance of Strawbale (incl. Oak Ridge Lab test results) – N.Stone Support Letters from Licensed Practitioners: Letters from 2 Structural Engineers, 4 Civil Engineers, 1 Professor of Civil Engineering, 7 Architects

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

PARII-INI	ERNATIONAL BU	ILDING COL	DE - STRUC	TURAL
Public Hearin	g: Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

Public Hearing: Committee: Assembly:	AS ASF	AM AMF	D DF		
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CH 24 (NEW)-S-HAMMER.doc

S317–12 Chapter 35 (NEW)

Proponent: John Woestman, Kellen Company, representing Composite Lumber Manufacturers Association (CLMA) (jwoestman@kellencompany.com)

Add new text as follows:

CHAPTER 35 COMPOSITES

SECTION 3501 GENERAL

3501.1 Scope. These provisions shall govern the materials, design, application, construction and installation of composite materials and products.

SECTION 3502 DEFINITIONS

3502.1 General. The following words and terms shall, for the purposes of this chapter have the meanings shown herein.

WOOD PLASTIC COMPOSITE. A composite material made primarily from wood or cellulose-based materials, and plastic.

SECTION 3503 WOOD PLASTIC COMPOSITE EXTERIOR MATERIALS AND PRODUCTS

3503.1 General. The provisions of this section shall govern the requirements and uses of wood plastic composite materials and products for exterior decks, balconies, and porches of buildings and structures.

<u>3503.1.1 Wood plastic composite exterior deck boards, stair treads, handrails, and guardrail</u> <u>systems.</u> Exterior deck boards, stair treads, handrails, and guardrail systems of wood plastic composite shall comply with this section.

3503.1.1.1 Minimum standards and quality. Exterior wood plastic composite deck boards, stair treads, and handrails and guardrail systems shall comply with ASTM D 7032.

3503.1.1.2 Structural. The allowable load and maximum allowable span for exterior wood plastic composite deck boards and stair treads shall be determined in accordance with ASTM D7032. Testing of handrails and guardrail systems to demonstrate compliance to the structural performance requirements of this code shall be in accordance with ASTM D7032.

3503.1.1.3 Labeling. Deck boards and stair treads shall bear a label that indicates compliance to ASTM D7032 and includes the allowable load and maximum allowable span. Handrails and guardrail systems or their packaging shall bear a label that indicates compliance to ASTM D7032 and includes the maximum allowable span.

3503.1.1.4 Installation. Wood plastic composite deck components shall be installed in accordance with the manufacturer's instructions.

Add new standard to Chapter 35 as follows:

ASTM

<u>D 7032-10a</u> Standard Specification for Establishing Performance Ratings For Wood-Plastic Composite Deck Boards and Guardrail Systems (Guards or Handrails)

Reason: Currently, the IBC is silent regarding specific requirements for wood plastic composite decking materials.

Composite materials may not neatly fit into the wood chapter of the IBC (Chapter 23) or in the plastics chapter (Chapter 26). A logical location for this material is in a new chapter titled "Composites." Looking to the future, this new chapter creates a logical location in the IBC for other composites that may be utilized in building construction but fall outside the scopes of Chapter 23 and Chapter 26. If Chapter 35, at the end of the IBC, is not the best location for this proposed new chapter for composites, ICC staff can editorially move this proposed new chapter to a more appropriate location in the IBC.

This proposal introduces a definition of wood plastic composite (limited to the scope of this chapter) and creates a section for exterior materials and products made from this specific material. Then the proposal limits the scope of the requirements to materials and products for exterior decks, balconies, and porches. Finally, the proposal introduces specific requirements for exterior wood plastic composite deck boards, stair treads, handrails, and guardrail systems.

With this proposal, CLMA seeks to introduce mandatory requirements in the IBC for exterior wood plastic composite deck components while making it easier for builders to comply with the code and for building officials to enforce the code.

Including the labeling requirement in this proposal brings WPCs within the requirements of the definition of "label" in Chapter 2 of the IBC, thus requiring 3rd party certification of these WPCs and ongoing quality assurance. This requirement helps to assure building officials that wood plastic composite decking and guards will meet the performance requirements of the IBC.

As with most engineered building components, wood plastic composite deck components should be required to be installed per the manufacturer's instructions. The manufacturer's instructions and this proposed language limits the use of these wood plastics to only those uses they were designed for.

This proposal requires wood plastic composite deck boards, stair treads, handrails and guardrail systems to meet the requirements of ASTM D7032, a standard developed specifically for demonstrating code compliance of WPC exterior deck components. Meeting the requirements of ASTM D7032 verifies the engineered WPC products are appropriate for use as exterior deck components. ASTM D7032 includes deck-related performance evaluations and performance requirements such as flexural tests, bio-degradation 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, moisture, concentrated loads, freeze-thaw resistance tests, UV resistance, and duration of load on WPC deck boards, stair treads, and handrail and guardrail systems.

The design capacity of each WPC deck board, stair tread, handrail, and guardrail system is tested and evaluated according to product specification ASTM D7032. The testing required in D7032 addresses IBC requirements for deck boards, stair treads, handrails, and guardrail systems.

The result of these tests determines an allowable load and span rating for deck boards and a stringer spacing for stair treads. Product labels will show verification of compliance with ASTM D 7032 and provide the appropriate performance information. For example, deck board labels would identify the allowable load and span (e.g., 100 psf load on a 16" span would be expressed as "16/100"). For stair treads, ASTM D7032 requires load and span testing at higher loads (300 psf and 750 lb concentrated load). This concentrated load test for WPC stair treads is 2.5 times what's required in the IBC in Table 1607.1, Footnote f.

Guardrail systems, per ASTM D7032, are required to be subjected to and pass the in-fill load test, the uniform load test, and the concentrated load test at 2.5 times the loads required by the IBC (in Section 1607.8) with the guardrail system constructed according to the manufacturer's instructions. These tests evaluate the strength and stiffness of all components and their connections.

For designers, specifiers, builders, and for code enforcement, the maximum post spacing (span) for guardrail systems is required to be on the label, as is verifying compliance to ASTM D7032. And, of course, guardrail systems for projects constructed under the IBC must meet the requirements of Section 1012 and 1013.

Assuming WPC deck boards, stair treads, and guardrail systems are selected, specified, and installed according to the manufacturer's instructions – and the manufacturer confirms compliance to ASTM D7032 in their literature and on the product label – designing exterior deck projects which use WPC components is quite straightforward: 1) Select WPC deck boards that meet or exceed the required load (per IBC Table 1607.1) at the desired span of the deck's joists. 2) Plan for stair stringers no farther apart than the maximum allowable span for the desired WPC stair treads. 3) Select a WPC guardrail system that meets the minimum height requirements for the project (i.e. 42" for the IBC) and plan for guardrail supports (posts) no further apart than the maximum spacing (span) allowed by the guardrail system's manufacturer.

Cost Impact: Zero to a cost reduction because of easier code compliance with specific requirements included in the IBC.

5317-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				CH 35 (NEW)-S-WOESTMAN.doc

S318–12 G1001.4

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net).

Revise as follows:

G1001.4 Enclosures below design flood elevation. Fully enclosed areas below the design flood elevation shall be at or above grade on all sides and conform to the following: <u>constructed in accordance</u> with ASCE 24.

- 1. In *flood hazard areas* not subject to high-velocity wave action, enclosed areas shall have flood openings to allow for the automatic inflow and outflow of floodwaters.
- 2. In flood hazard areas subject to high-velocity wave action, enclosed areas shall have walls below the design flood elevation that are designed to break away or collapse from a water load less than that which would occur during the design flood, without causing collapse, displacement or other structural damage to the building or structure.

Reason: ASCE 24 includes requirements for enclosures below elevated buildings that vary based on flood zone. Referencing ASCE 24 eliminates the need to make coordinating changes if ASCE 24 changes in the future.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: The code change proposal will not increase the cost of construction. Utility and miscellaneous group U buildings with enclosures should already be required to meet the requirements for enclosures.

S318-12				
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF
				G1001.4-S-INGARGIOLA-WILSON-QUINN.doc

S319–12 G102.1

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G102.1 General. This appendix, in conjunction with the *International Building Code*, provides minimum requirements for development located in flood hazard areas, including the subdivision of land; <u>site</u> <u>improvements and</u> installation of utilities; placement and replacement of manufactured homes; <u>placement</u> <u>of recreational vehicles</u>; new construction and repair, reconstruction, rehabilitation or additions to new construction; substantial improvement of existing buildings and structures, including restoration after damage, <u>installation of tanks</u>; temporary structures, and temporary or permanent storage, utility and miscellaneous Group U buildings and structures, and certain building work exempt from permit under Section 105.2 and other buildings and development activities.

Reason: The purpose of this section is to identify the development activities for which minimum requirements are listed in Appendix G. The proposed changes are consistent with the subsections in Appendix G (including some proposed new subsections).

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

S319-12				
Public Hearing:	Committee:	AS	AM	D
Assembly:	ASF	AMF	DF	
				G102.1-S-INGARGIOLA-WILSON-QUINN.doc

S320–12 G103.1

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G103.1 Permit applications. All applications for permits must comply with the following

- 1. The *building official* shall review all *permit* applications to determine whether proposed development sites will be reasonably safe from flooding are located in flood hazard areas established in Section G102.2.
- 2. If a proposed development site is in a flood hazard area, all site development activities (including grading, filling, utility installation and drainage modification), all new construction and substantial improvements (including the placement of prefabricated buildings and manufactured homes) and certain building work exempt from *permit* under Section 105.2 all development to which this appendix is applicable as specified in Section G102.1 shall be designed and constructed with methods, practices and materials that minimize flood damage and that are in accordance with this code and ASCE 24.

Reason: This proposal clarifies that the first step is to determine whether proposed development activities are locate in (or out) of the mapped flood hazard area. The second item is simplified; rather than restate the long list of development activities, it is clearer to refer to the list that is already present in G102.1.

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

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S321–12 G103.4, G103.5, G103.6.1, G401.1

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net).

Revise as follows:

G103.4 Activities in riverine flood hazard areas. In riverine *flood hazard areas* where *design flood elevations* are specified but *floodways* have not been designated, the *building official* shall not permit any new construction, substantial improvement or other development, including fill, unless the applicant <u>submits an engineering analysis prepared and sealed by a registered design professional, that</u> demonstrates that the cumulative effect of the proposed development, when combined with all other existing and anticipated flood hazard area encroachment, will not increase the design flood elevation more than 1 foot (305 mm) at any point within the community.

G103.5 Floodway encroachment. Prior to issuing a *permit* for any *floodway* encroachment, including fill, new construction, substantial improvements and other development or land-disturbing activity, the *building official* shall require submission of a certification, <u>sealed by a registered design professional</u>, along with supporting technical data, that demonstrates that such development will not cause any increase of the level of the base *flood*.

G103.6.1 Engineering analysis. The *building official* shall require submission of an engineering analysis, <u>prepared and sealed by a registered professional</u>, which demonstrates that the flood-carrying capacity of the altered or relocated portion of the watercourse will not be decreased. Such watercourses shall be maintained in a manner which preserves the channel's flood-carrying capacity.

G103.7 Alterations in coastal areas. Prior to issuing a permit for any alteration of sand dunes and mangrove stands in flood hazard areas subject to high velocity wave action, the *building official* shall require submission of an engineering analysis, <u>prepared and sealed by a registered design professional</u>, which demonstrates that the proposed alteration will not increase the potential for flood damage.

G401.1 Development in floodways. Development or land disturbing activity shall not be authorized in the *floodway* unless it has been demonstrated through hydrologic and hydraulic analyses performed in accordance with standard engineering practice, and prepared and sealed by a registered design professional, that the proposed encroachment will not result in any increase in the level of the base *flood*.

Reason: The analyses referred to in these sections are prepared by engineers. The building official is not expected to have the experience or qualifications to determine whether such analyses were properly prepared. Specifying that the work has to be prepared and sealed by an RDP puts the burden on the RDP to meet standards of practice for these analyses. This requirement is consistent with the NFIP and the same requirement should already appear in local floodplain management regulations.

Cost Impact: The code change proposal will not increase the cost of construction. This requirement is consistent with the NFIP and the same requirement should already appear in local floodplain management regulations.

S321-12			
Public Hearing: Committee:	AS	AM	D
Assembly:	ASF	AMF	DF
•			G103.4-S-INGARGIOLA-WILSON-QUINN.doc

S322-12 G103.8 (NEW)

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Add new text as follows:

G103.8 Inspections. Development for which a permit under this appendix is required shall be subject to inspection. The building official or the building official's designee shall make or cause to be made, inspections of all development in flood hazard areas authorized by issuance of a permit under this appendix.

Reason: Just as the code requires inspection of permitted buildings, this appendix should require inspection of all other development in flood hazard areas for which permits are issued.

Cost Impact: The code change proposal will not increase the cost of construction. Inspection of non-building development that is permitted in flood hazard areas should already be performed by communities that participate in the NFIP.

S322-12				
Public Hearing:	Committee:	AS	AM	D
-	Assembly:	ASF	AMF	DF
				G103.8 #1-S-INGARGIOLA-WILSON-QUINN.doc

S323–12 G103.8 (NEW), G104.2

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G103.8 Substantial improvement and substantial damage determinations. For permit applications to improve or repair buildings and structures, including additions, repairs, rehabilitations, renovations, alterations, relocations, reconstructions, or other work, the building official, shall:

- Estimate the market value, or require the applicant to obtain a professional appraisal of the market value, of the building or structure before the proposed work is performed; the market value of the building or structure shall be the market value before the damage occurred or before any improvement is made;
- Compare the cost to perform the improvement, the cost to repair the damaged building to its predamaged condition, or the combined costs of improvements and repairs, if applicable, to the market value of the building or structure;
- 3. Determine and document whether the proposed work constitutes substantial improvement or repair of substantial damage; and
- 4. If the determination finds that the proposed work constitutes substantial improvement or repair of substantial damage, notify the applicant of the results of the determination and whether compliance with the requirements of the building code is required.

G103.8 <u>G103.9</u> **Records.** The *building official* shall maintain a permanent record of all *permits* issued in *flood hazard areas*, including copies of inspection reports and certifications required in Section 1612.

G104.2 Application for permit. The applicant shall file an application in writing on a form furnished by the *building official*. Such application shall:

- 1. Identify and describe the development to be covered by the permit.
- 2. Describe the land on which the proposed development is to be conducted by legal description, street address or similar description that will readily identify and definitely locate the site.
- 3. Include a site plan showing the delineation of flood hazard areas, floodway boundaries, flood zones, design flood elevations, ground elevations, proposed fill and excavation and drainage patterns and facilities.
- 4. Indicate the use and occupancy for which the proposed development is intended.
- 5. Be accompanied by construction documents, grading and filling plans and other information deemed appropriate by the building official.
- 6. State the valuation of the proposed work.
- 7. Include a market value appraisal of the building (excluding land), for applications for work on existing buildings, unless otherwise advised by the building official.
- 78. Be signed by the applicant or the applicant's authorized agent.

Reason: Communities that participate in the NFIP agree to regulate all development in flood hazard areas. FEMA states that the flood provisions in the I-Codes are consistent with the NFIP requirements for the design and construction of buildings. To fully meet the requirements of the NFIP local jurisdictions must adopt a local ordinance or Appendix G in order to have the necessary administrative provisions and requirements for development other than buildings.

Section 105.3 of the code requires the applicant to describe the work to be covered by the permit and to state the valuation of the proposed work. The building code defines and uses the terms "substantial improvement" and "substantial damage." This proposal clarifies how the building official is to use the information to determine whether proposed work meets the definitions.

FEMA recently published FEMA P-758, Substantial Improvement/Substantial Damage Desk Reference, that includes guidance for local officials on estimating market value as well as estimating costs. This proposal states that the applicant shall submit a
market value appraisal unless otherwise advised; FEMA guidance now states that local officials may use "adjusted assessed value" or "actual cash value" (replacement minus depreciation).

Cost Impact: The code change proposal will not increase the cost of construction. Determining whether work proposed on an existing building is substantial improvement or repair of substantial damage is implicit in the definitions of those terms. This proposal does not change the fact that determining whether proposed work meets those definitions has to be done. It simply clarifies how it is to be done.

S323-12

			G103.8 #2-S-INGARGIOLA-WILSON-QUINN.do
Assembly:	ASF	AMF	DF
Public Hearing: Committee:	AS	AM	D

S324–12 G104.2

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G104.2 Application for permit. The applicant shall file an application in writing on a form furnished by the *building official*. Such application shall:

- 1. Identify and describe the development to be covered by the permit.
- 2. Describe the land on which the proposed development is to be conducted by legal description, street address or similar description that will readily identify and definitely locate the site.
- Include a site plan showing the delineation of flood hazard areas, floodway boundaries, flood zones, design flood elevations, ground elevations, proposed fill and excavation and drainage patterns and facilities.
- 4. Include in subdivision proposals and other proposed developments with more than 50 lots or larger than 5 acres, base flood elevation data in accordance with to Section 1612.3.1 if such data are not identified for the flood hazard areas established in Section G102.2.
- 45. Indicate the use and occupancy for which the proposed development is intended.
- 56. Be accompanied by construction documents, grading and filling plans and other information deemed appropriate by the building official.
- 67. State the valuation of the proposed work.
- $7\overline{8}$. Be signed by the applicant or the applicant's authorized agent.

Reason: Appendix G includes requirements for subdivisions which is consistent with the NFIP requirement un federal regulation (44 CFR 60.3(b)(3)). If proposals for larger developments and subdivisions are affected by flood hazard areas shown on FIRMs, but the areas do not have base flood elevations, the requirement is that elevations have to be developed. Section 1612.3.1 allows use of data available from other sources, or authorizes the building official to require such information be developed by the applicant.

Cost Impact: The code change proposal will not increase the cost of construction. This should already be required by communities that participate in the NFIP

S324-12				
Public Hearing: Comr	mittee:	AS	AM	D
Asser	mbly:	ASF	AMF	DF
				G104.2-S-INGARGIOLA-WILSON-QUINN.doc

S325–12 G501 (NEW)

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Add new text as follows:

G501.4 Protection of mechanical equipment and outside appliances. Mechanical equipment and outside appliances shall be elevated to or above the design flood elevation.

Exception. Where such equipment and appliances 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 up to the elevation required by Section 1612, the systems and equipment shall be permitted to be located below the elevation required by Section 1612. Electrical wiring systems shall be permitted below the design flood elevation provided they conform to the provisions of NFPA 70.

Reason: This language comes from G1001.6. Adding this does not create a new requirement because the NFIP requires that the same code requirements for equipment and appliances associated with buildings in flood hazard areas also apply to equipment and appliances associated with manufactured homes.. FEMA guidance is found in *Protecting Manufactured Homes from Floods and Other Hazards* (FEMA P-85, issued November 2009).

Cost Impact: The code change proposal will not increase the cost of construction. Elevation or protection of equipment and appliances is already a requirement for communities that participate in the NFIP

S325-12				
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF
				GEN1 (NEW) S INGARCIOLA WILSON OLINN doc

S326–12 G501.4 (NEW)

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Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Add new text as follows:

G501.4 Enclosures. Fully enclosed areas below elevated manufactured homes shall comply with the requirements of Section 1612.

Reason: Adding this does not create a new requirement because the NFIP and local floodplain management ordinances require that the same requirements for enclosed areas below elevated buildings also apply to enclosures under elevated manufactured homes (Section 1612 refers to ASCE 24 for specific requirements, which vary based on flood zone). FEMA guidance is found in *Protecting Manufactured Homes from Floods and Other Hazards* (FEMA P-85, issued November 2009).

Cost Impact: The code change proposal will not increase the cost of construction. Already a requirement for communities that participate in the NFIP.

S326-12					
Public Hearing:	Committee:	AS	AM	D	
	Assembly:	ASF	AMF	DF	
					G501-S-INGARGIOLA-WILSON-QUINN.doc

S327–12 G701.1 (NEW)

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net).

Delete and substitute as follows:

G701.1 Underground tanks. Underground tanks in *flood hazard areas* shall be anchored to prevent flotation, collapse or lateral movement resulting from hydrostatic loads, including the effects of buoyancy, during conditions of the design *flood*.

G701.2 Above-ground tanks. Above-ground tanks in flood hazard areas shall be elevated to or above the design *flood* elevation or shall be anchored or otherwise designed and constructed to prevent flotation, collapse or lateral movement resulting from hydrodynamic and hydrostatic loads, including the effects of buoyancy, during conditions of the design *flood*.

G701.3 Tank inlets and vents. In flood hazard areas, tank inlets, fill openings, outlets and vents shall be:

- 1. At or above the design flood elevation or fitted with covers designed to prevent the inflow of floodwater or outflow of the contents of the tanks during conditions of the design *flood*.
- Anchored to prevent lateral movement resulting from hydrodynamic and hydrostatic loads, including the effects of buoyancy, during conditions of the design flood.

G701.1 Tanks. Underground and above-ground tanks shall be designed, constructed, installed and anchored in accordance with ASCE 24.

Reason: ASCE 24 contains both performance requirements for tanks and the limitations based on flood zone. This proposal references ASCE 24, rather than replicate those requirements in Appendix G, thus eliminating the need to make coordinating changes if ASCE 24 changes in the future.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: The code change proposal will not increase the cost of construction. Tanks in flood hazard areas are already regulated.

S327-12			
Public Hearing: Committee:	AS	AM	D
Assembly:	ASF	AMF	DF
-			G701-1-S-INGARGIOLA-WILSON-QUINN.doc

S328–12 G801.1

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G801.1 Detached <u>Garages and</u> accessory structures. Detached accessory structures shall be anchored to prevent flotation, collapse or lateral movement resulting from hydrostatic loads, including the effects of bouyancy, during conditions of the design *flood*. Fully enclosed accessory structures shall have flood openings to allow for the automatic entry and exit of *flood* waters. <u>Garages and accessory structures shall be designed and constructed in accordance with ASCE 24.</u>

Reason: ASCE 24 contains requirements garages and accessory structures that allow them to be constructed without meeting the elevation requirements, provided certain other requirements are met. Those requirements are, in part, based on flood zone. This proposal references ASCE 24, rather than replicate those requirements in Appendix G, thus eliminating the need to make coordinating changes if ASCE 24 changes in the future.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: The code change proposal will not increase the cost of construction. Garaged and accessory structures in flood hazard areas are development and thus are already regulated.

S328-12				
Public Hearing:	Committee:	AS	AM	D
-	Assembly:	ASF	AMF	DF
	•			G801.1-S-INGARGIOLA-WILSON-QUINN.doc

S329–12 G801.5

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G801.5 Swimming pools. Prefabricated swimming pools Swimming pools shall be designed and constructed in accordance with ASCE 24. Above-ground swimming pools, on-ground swimming pools, and in-ground swimming pools that involve placement of fill in *floodways* shall also meet the requirements of Section G103.5.

Reason: ASCE 24-05 includes requirements for pools which vary by flood zone. The next edition of ASCE 24 will more distinctly clarify requirements for pools in different flood zones. Referencing ASCE 24 eliminates the need to make coordinating changes in the future.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: The code change proposal will not increase the cost of construction. Pools in flood hazard areas are development and thus are already regulated.

S329-12			
Public Hearing: Committee:	AS	AM	D
Assembly:	ASF	AMF	DF
			G801.5-S-INGARGIOLA-WILSON-QUINN.doc

S330–12 G801.6 (NEW)

Proponent:

Revise as follows: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Add new text as follow:

<u>G801.6</u> Decks, porches, and patios. Decks, porches and patios shall be designed and constructed in accordance with ASCE 24.

Reason: ASCE 24 includes requirements for decks, porches, and patios which vary by flood zone. Referencing ASCE 24 eliminates the need to make coordinating changes if ASCE 24 changes in the future.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: The code change proposal will not increase the cost of construction. Decks, porches, and patios in flood hazard areas are development and thus are already regulated.

S330-12				
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

G801.6#1-S-INGARGIOLA-WILSON-QUINN.doc

S331–12 G801.6 (NEW)

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Add new text as follows:

G801.6 Non-structural concrete slabs in coastal high hazard areas and coastal A zones. In coastal high hazard areas and coastal A zones, non-structural concrete slabs used as parking pads, enclosure floors, landings, decks, walkways, patios and similar nonstructural uses are permitted beneath or adjacent to buildings and structures provided the concrete slabs shall be constructed in accordance with ASCE 24

Reason: ASCE 24 includes requirements for nonstructural slabs, which vary by flood zone. Referencing ASCE 24 eliminates the need to make coordinating changes if ASCE 24 changes in the future.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: The code change proposal will not increase the cost of construction. Non-structural concrete slabs in flood hazard areas are development and thus are already regulated.

S331-12					
Public Hearing:	Committee: Assembly:	AS ASF	AM AMF	D DF	

G801.6#2-S-INGARGIOLA-WILSON-QUINN.doc

S332-12 G801.6 (NEW)

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Add new text as follows:

<u>G801.6 Roads and watercourse crossings in regulated floodways</u>. Roads and watercourse crossings that encroach into regulated floodways, including roads, bridges, culverts, low-water crossings and similar means for vehicles or pedestrians to travel from one side of a watercourse to the other side, shall meet the requirement of Section G103.5.

Reason: The NFIP requires communities to regulate all development. The concern with roads and other crossings is whether they encroach into floodways. Floodway encroachments may cause increases in flood elevations which can increase flooding on other properties and increase the extend of mapped special flood hazard areas.

Cost Impact: The code change proposal will not increase the cost of construction. Waterway crossings are development and thus are already regulated.

S332-12				
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF
				G801.6-S-INGARGIOLA-WILSON-QUINN.doc

S333–12 G901.1

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G901.1 Temporary structures. Temporary structures shall be erected for a period of less than 180 days. Temporary structures shall be anchored to prevent flotation, collapse or lateral movement resulting from hydrostatic loads, including the effects of buoyancy, during conditions of the design *flood*. Fully enclosed temporary structures shall have flood openings <u>that are in accordance with ASCE 24</u> to allow for the automatic entry and exit of floodwaters.

Reason: Without the reference to ASCE 24, neither the applicant nor the building official has enough specificity to determine whether flood openings are compliant.

Cost Impact: The code change proposal will not increase the cost of construction. Consistent with FEMA guidance for temporary structures that are walled and roofed.

S333-12			
Public Hearing: Committee:	AS	AM	D
Assembly:	ASF	AMF	DF G901.1-S-INGARGIOLA-WILSON-QUINN.doc

S334–12 J101.2

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

J101.2 Flood hazard areas. The provisions of this chapter shall not apply to Unless the applicant has submitted an engineering analysis, prepared in accordance with standard engineering practice, and sealed by a registered design professional, that demonstrates the proposed work will not result in any increase in the level of the base flood, grading, excavation and earthwork construction, including fills and embankments, shall not be permitted in *floodways* within *flood hazard areas* established in Section 1612.3 or in *flood hazard areas* where design *flood* elevations are specified but floodways have not been designated, unless it has been demonstrated through hydrologic and hydraulic analyses performed in accordance with standard engineering practice that the proposed work will not result in any increase in the level of the base flood.

Reason: This proposal is editorial only. It is intended to make the provision clearer. The only new text is that the engineering analysis is to be prepared and sealed by a registered design professional, which takes the burden off the building official.

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

S334-12				
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF
				J101.2-S-INGARGIOLA-WILSON-QUINN.doc

S335–12 L101.1

Proponent: James Bela, Oregon Earthquake Awareness, representing self

Revise as follows:

L101.1 General. Every structure building located where the 1-second spectral response acceleration, S1, in accordance with Section 1613.3 is greater than 0.40 within 15 miles distance of an active fault with a maximum potential earthquake M 6 or above, or lies within 25 miles distance of an active fault wit a maximum potential earthquake M 7 or avove; that either 1) exceeds six stories in height with an aggregate floor area of 60,000 square feet (5574 m2) or more, or 2) exceeds ten stories in height regardless of floor area, shall be equipped with not less than three approved recording accelerographs. The accelerographs shall be interconnected for common start and common timing.

Reason: The 1-second spectral response acceleration contours are interesting, but their locations are yo-yoing around with each new addition of the maps; such that they are not reliable over time. See discussion per Code Change: **IBC-12.13 FIGURE 1613.3.3.1 (1)(2)(3)(4)(5)(6)_.**

An earthquake will occur on a fault, and it is the proximity of a building to an earthquake source that determines its actual experience to ground shaking in a real earthquake. This additional charging language fills this hole in locations, particularly in the western U.S. where there are active faults; but the sum total (of probabilities of exceedence) of all contributing faults is not enough to give 1-second contours of 0.40g.

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

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

S335-12 Public Hearing: Committee: AS AM D Assembly: ASF AMF DF L101.1-S-BELA.doc

S336–12 M101

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

Revise as follows:

APPENDIX M TSUNAMI-GENERATED FLOOD HAZARD The provisions contained in this appendix are not mandatory unless specifically referenced in the adopting ordinance

SECTION M101 TSUNAMI-GENERATED FLOOD HAZARD

SECTION M101 GENERAL

M101.1 General Scope. The purpose of this appendix is to provide tsunami regulatory criteria for those communities that have a tsunami hazard and have elected to develop and adopt a map of their tsunami hazard inundation zone. This appendix applies to structures located within an identified Tsunami Hazard Zone, as defined by the Authority Having Jurisdiction.

M101.2 Performance objectives. All structures that are considered either essential to the community and its disaster response or structures that represent a substantial hazard to human life in the event of failure, as defined by Risk Category III and IV as specified under Section 1604.5 of the International Building Code, must be protected from tsunamis by either being located outside of the Tsunami Hazard Zone or be designed and constructed to withstand without collapse the specified loads and effects associated with the Maximum Considered Tsunami. For structures in other Risk Categories, life safety protection is to be provided by a community Tsunami Warning and Evacuation Procedure.

M101.3 Tsunami Design Hazard Level. The regulatory criteria contained in this appendix is based on the Maximum Considered Tsunami and its associated flow elevation and velocity, which shall be determined by the Authority Having Jurisdiction. The Maximum Considered Tsunami shall be permitted to be derived either deterministically or probabilistically by the Authority Having Jurisdiction. The Maximum Considered Tsunami shall be represented using a Tsunami Hazard Zone Map adopted by the Authority Having Jurisdiction.

M101.2 M101.4 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein.

MAXIMUM CONSIDERED TSUNAMI. A tsunami that is determined and adopted by the Authority Having Jurisdiction for design purposes and represented using a Tsunami Hazard Zone Map. The Maximum Considered Tsunami shall be taken as having a collapse prevention design equivalent of a 2% probability of being exceeded in a 50-year period or a 2500 year average return period.

TSUNAMI HAZARD ZONE MAP. A map adopted by the <u>community</u> <u>authority having jurisdiction</u> that designates the extent of inundation by a design event the maximum considered tsunami. This map shall be based on the take into consideration any available tsunami inundation map which is developed and provided to a community by either the applicable State agency or the National <u>Oceanic and</u> Atmospheric and <u>Oceanic</u> Administration (NOAA) under the National Tsunami Hazard Mitigation program, but shall be permitted to utilize a different probability or hazard level.

TSUNAMI HAZARD ZONE. The area vulnerable to being flooded or inundated by a design event the maximum considered tsunami as identified on a community's Tsunami Hazard Zone Map.

TSUNAMI VERTICAL EVACUATION REFUGE. A Tsunami Vertical Evacuation Refuge is a structure designated to serve as a point of refuge to which a community's population can evacuate above a tsunami when high ground is not available. It is designed and constructed so as to comply with the applicable provisions of the latest edition of *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*, published by the Federal Emergency Management Agency (FEMA P-646).

TSUNAMI WARNING AND EVACUATION PROCEDURE. A Tsunami Warning and Evacuation Procedure is a plan and procedure developed and adopted by a community that would receive a tsunami warning from the National Oceanic and Atmospheric Administration (NOAA) at all hours and transmit that warning to its citizens and establishes and designates evacuation routes for its citizens to either high ground or to a designated Tsunami Vertical Evacuation Refuge. Tsunami evacuation procedures may use evacuation maps that are significantly greater in extent than the tsunami hazard zone and are not developed for design purposes. Tsunami evacuation maps are based on the tsunami inundation map which is developed and provided to a community by either the applicable State agency or NOAA under the National Tsunami Hazard Mitigation Program.

SECTION M102 TSUNAMI REGULATORY CRITERIA

M101.3 M102.1 Establishment of Tsunami Hazard Zone. Where applicable, if a community has adopted a Tsunami Hazard Zone Map, that map shall be used to establish a community's Tsunami Hazard Zone.

M101.4 <u>M102.2</u> Construction within the Tsunami Hazard Zone. Construction of structures designated Risk Category III and IV as specified under Section 1604.5 shall be prohibited within a Tsunami Hazard Zone.

Exceptions:

- 1. A vertical evacuation tsunami refuge shall be permitted to be located in a Tsunami Hazard Zone provided it is constructed in accordance with FEMA P646.
- 2. Community <u>Risk Category III and IV structures and other</u> critical facilities shall be permitted to be located within the Tsunami Hazard Zone when such a location is necessary to fulfill their function, providing suitable structural and emergency evacuation the following measures have been incorporated.
- 1. The structure and its foundation shall be designed to resist without collapse all tsunami loads associated with the Maximum Considered Tsunami, including hydrostatic, hydrodynamic, waterborne debris accumulation and impact loads, and scour.
- 2. A Tsunami Warning and Evacuation Procedure has been incorporated for the facilities.

M102.3 Tsunami Vertical Evacuation Refuge. A structure designated as a Tsunami Vertical Evacuation Refuge shall be permitted to be located in a Tsunami Hazard Zone provided it meets the following criteria:

- 1. The structure shall be designated as a Tsunami Vertical Evacuation Refuge Structure and shall be capable of being operational within the community's tsunami warning time.
- 2. The structure shall be designed and constructed so as to comply with the applicable provisions of the latest edition of *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*, published by the Federal Emergency Management Agency (FEMA P-646).
- 3. All operational components of the refuge structure necessary for life safety shall be located above the elevation of the Maximum Considered Tsunami.

The structure and its foundation shall be designed and constructed to resist seismic loads as defined in Chapter 16 of the International Building Code for Risk Category IV structures.

M102.4 Tsunami Warning and Evacuation Procedure. The jurisdiction shall have a Tsunami Warning and Evacuation Procedure adopted and enforced by a community that shall be capable of receiving a tsunami warning from the National Oceanic and Atmospheric Administration (NOAA) at all hours and transmit that warning to its citizens and shall establish and designate evacuation routes for its citizens to either high ground or to a designated Tsunami Vertical Evacuation Refuge.

SECTION M102 M103 REFERENCED STANDARDS

FEMA P646—08 Guidelines for Design of Structures for Vertical Evacuation from Tsunamis

Reason: On March 11, 2011, a magnitude 9.0 earthquake struck off the coast of Japan. Although Japan is the most advanced country in the world when it comes to tsunami protection measures, 20,000 people perished from the resulting tsunami. While the damage was utterly devastating with over 250,000 structures collapsed, there were many examples of engineered buildings of multistory construction that survived the earthquake and subsequent tsunami as well as many partially inundated vertical evacuation refuge buildings that successfully saved many lives.

This same type of subduction fault lies off the coastline of Washington, Oregon and northern California, and Alaska and is capable of unleashing a similar magnitude earthquake and resulting tsunami. Furthermore, tsunamis can and have struck the entire Pacific coast, Hawaii, the Caribbean, portions of the Atlantic coast and even within the Gulf of Mexico. While the probability of a damaging tsunami may be low, the consequences would be enormous.

Prior to the 2011 Japan tsunami, the American Society of Civil Engineers/Structural Engineering Institute Standard ASCE/SEI 7 *Minimum Design Loads for Buildings and Other Structures* had formed a new committee to develop a new chapter on tsunami design. While the committee's work is ongoing, we should update Appendix M with some of their work to date relating to the tsunami load criteria and associated design provisions for essential facilities, such as defining a Maximum Considered Tsunami.

The first Appendix M, adopted and published in the 2012 IBC, focused on keeping critical and high risk structures out of the tsunami inundation zone. This revision keeps that same philosophy but expands the description of what is a properly constructed Tsunami Vertical Evacuation Refuge that can withstand without collapse the hydrostatic, hydrodynamic, debris accumulation and impact loads, and scour associated with the Maximum Considered Tsunami.

The National Tsunami Hazard Mitigation Program is proposing this change to keep Appendix M as current as possible with the latest appropriate information to come out of the ongoing ASCE/SEI 7 Tsunami Loads and Effects Committee's development work. This change proposal has been reviewed by the committee.

Cost Impact: Since the primary difference between this proposed change and the current Appendix M is that it would allow for construction within the Tsunami Inundation Zone providing it meets certain criteria, cost impact is not applicable.

S336-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
				M101 (NEW)-S-MAHONEY.doc

S337-12 M101

Proponent: James Bela, Oregon Earthquake Awareness, representing self

Delete without substitution:

SECTION M101 TSUNAMI-GENERATED FLOOD HAZARD

M101.1 General. The purpose of this appendix is to provide tsunami regulatory criteria for those communities that have a tsunami hazard and have elected to develop and adopt a map of their tsunami hazard inundation zone.

M101.2 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein.

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

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

M101.3 Establishment of Tsunami Hazard Zone. Where applicable, if a community has adopted a Tsunami Hazard Zone Map, that map shall be used to establish a community's Tsunami Hazard Zone.

M101.4 Construction within the Tsunami Hazard Zone. Construction of structures designated Risk Category III and IV as specified under Section 1604.5 shall be prohibited within a Tsunami Hazard Zone.

Exceptions:

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

SECTION M102 REFERENCED STANDARDS

FEMA P646—08 Guidelines for Design of M101.4 Structures for Vertical Evacuation from Tsunamis

Reason: Given the recent M 9.1 Great 11 March 2011 Tohoku Earthquake and Tsunami disaster in Japan, I would view this code section to be extremely dangerous to public safety; and I believe that it should be removed. Vertical evacuation structures were overtopped in the Tohoku earthquake, and people were killed as a result. Even concrete structures (previously assumed to be "invincible" were overturned and destroyed.

This "weak" and very problematical FEMA effort has copied the same "failed approach" for U.S. Building design practice – it presupposes a "design tsunami event" – and somehow probabilistically determined. No one is accountable for its failures and tragic loss-of-life that could result if such a standard were "followed." They are too uncertain for "local tsunami" generated waves and coastal innundation.

There needs to be a more "stringent" for accepting something into the building code as a "standard". The fact that it is located in the appendix speaks for itself.

For further background information:

Union Frontiers of Geophysics Lecture: Tohoku to Tsunami: Personal Account From Science to Experience by Hiroo Kanamori http://sites.agu.org/fallmeeting/scientific-program/lectures/

Insights from the great 2011 Japan earthquake: The diverse set of waves generated in Earth's interior, oceans, and atmosphere during the devastating Tohoku-oki earthquake reveal some extraordinary geophysics Thorne Lay and Hiroo Kanamori http://www.physicstoday.org/resource/1/phtoad/v64/i12/p33_s1?bypassSSO=1
S23C Gutenberg Lecture* Great Earthquake Ruptures in the Age of Seismo-Geodesy Presented by Thorne Lay, University of California, Santa Cruz, USA http://sites.agu.org/fallmeeting/scientific-program/lectures/bowie-and-named-lectures/6dec/
U33C The Great 11 March 2011 Tohoku Earthquake I Moscone South, Room 104, 1340h http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-7-december/
U34A The Great 11 March 2011 Tohoku Earthquake II Moscone South, Room 104, 1600h http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-7-december/
U41D The Great 11 March 2011 Tohoku Earthquake III Moscone South, Room 104, 0800h http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-8-december/

U42A The Great 11 March 2011 Tohoku Earthquake IV Moscone South, Room 104, 1020h http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-8-december/

U23C Predicting Extreme Events in Natural and Socioeconomic Systems: State-of-the-Art and Emerging Possibilities II Moscone South, Room 103, 1340h http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-6-december/

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

S337-12				
Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	M101-S-BELA.doc

S338–12 Appendix L (NEW)

Proponent: Stephen V. Skalko, P.E., Portland Cement Association; Eric T. Stafford, representing Institute for Business and Home Safety

Add new text as follows:

APPENDIX L BUILDING RESILIENCE

The provisions in this appendix are not mandatory unless specifically referenced in the adopting ordinance.

SECTION L101 GENERAL

L101.1 Purpose. The purpose of this Appendix is to promote enhanced public health, safety and general welfare and to reduce public and private property losses due to hazards and natural disasters associated with fires, flooding, high winds and earthquakes.

SECTION L102 STRUCTURAL

L102.1 Ground snowloads. The ground snowloads to be used in determining the design snow loads for roofs shall be equal to 1.2 times the ground snowloads determined in accordance with ASCE 7 or Figure 1608.2 for the contiguous United States and Table 1608.2 for Alaska in the *International Building Code*. Site-specific case studies shall be made in areas designated "CS" in Figure 1608.2. Ground snow loads for sites at elevations above the limits indicated in Figure 1608.2 and for all sites within the CS areas shall be *approved*. Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2-percent annual probability of being exceeded (50-year mean recurrence interval). Snow loads are zero for Hawaii, except in mountainous regions as *approved* by the *building official*.

L102.2 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the 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. The design wind pressure, p, and design wind force, F, determined in accordance with ASCE 7 or 1609.6 shall be based on a design wind speed equal to the basic wind speed (or locally adopted basic wind speed in special wind zones, if higher) determined in accordance with Section 1609.3 as follows:

- 1. <u>Ultimate design wind speed from Figure 1609A plus 20-mph.</u>
- 2. Ultimate design wind speed from Figure 1609B plus 10 mph
- 3. <u>Ultimate design wind speed from Figure 1609C.</u>

Component and cladding loads shall be determined for the design wind speed defined assuming terrain Exposure C, regardless of the actual local exposure. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

L102.3 Flood loads. Buildings designed and constructed in flood hazard areas defined in Section 1612.2 of the Code shall comply with Sections L102.3.1 and L102.3.2.

L102.3.1 Floors above base flood elevation. Floors required by ASCE 24 to be built above base flood elevations shall have the floor and their lowest horizontal supporting member not less than the higher of the following:

- (a) Design flood elevation,
- (b) Base flood elevation plus 3 feet, or
- (c) advisory base flood elevation plus 3 feet, or
- (d) 500-year flood, if known

L102.3.2 Flood protective works. Buildings designed and constructed in accordance with ASCE 24 shall not consider levees or floodwalls for providing flood protection during the design flood.

L102.4 Earthquake loads. In order to limit the impact of seismic events on the *building* the *building* shall comply with Section L102.4.1 and L102.4.2

L102.4.1 Seismic design importance factor. Where the ASCE 7 mapped 0.2 sec spectral response acceleration parameter, $S_{\rm S}$, shown on Figures 1613.3.1(1), 1613.3.1(3), 1613.3.1(4) or 1613.3.1(6) is greater than or equal to 40%g, the importance factor, I, in Table 11.5-1 of ASCE 7 shall be:

- 1. Not less than 1.15 for Risk Category II buildings
- 2. Not less than 1.35 for Risk Category III buildings
- 3. Not less than 1.6 for Risk Category IV buildings

L102.4.2 Seismic Design Categories C, D, E and F. If the *seismic design category* is determined to be C, D, E or F in accordance with Section 1613.3.5 a site specific geotechnical report complying with the provisions of ASCE 7 Section 11.8 is required, and the building shall be designed by a *registered design professional*.

L102.5 Storm shelters. *Buildings* and structures shall be provided with storm shelters conforming to the requirements of Section 423 where required by Sections L102.5.1 through L102.5.2 of this code.

L102.5.1 Storm shelters required. Storm shelters shall be provided for occupants of buildings in accordance with Sections L102.5.1.1, L102.5.1.2 and L102.5.2.

Exceptions:

- 1. *Buildings* meeting the requirements for shelter design in ICC/NSSA 500.
- 2. Where storm shelters within 1/4-mile of the proposed *building* are available and have adequate size to accommodate the added occupant load of the proposed *building*.
- 3. <u>Where the code official determines the *building* size, location or occupant load does not warrant shelters.</u>

L102.5.1.1 Hurricane areas. In hurricane-prone regions as defined in Section 1609.2 the following buildings shall be provided with storm shelters:

- 1. <u>Community halls, gymnasiums and libraries assigned to Group A3 occupancy classification.</u>
- 2. <u>Civic administration facilities assigned to Group B occupancy classification.</u>
- 3. Buildings assigned to Group E, I-1, I-2, I-3, M or R occupancy classifications.
- 4. Buildings assigned to Risk Category I in accordance with Section 1604.5.

L102.5.1.2 Tornado areas. In areas where the shelter design wind speed for tornadoes of Figure 304.2.(1) of ICC/NSSA 500 is 160 mph or greater, tornado shelters shall be provided, except that such shelters shall not be required for buildings classified as Group U occupancies or classified as Risk Category I according to Table 1604.5. L102.5.2 Combined hurricane and tornado shelters. Where combined hurricane and tornado shelters are provided the shelter shall comply with the more stringent requirements of ICC/NSSA-500 for both types of shelters.

L102.6 Wildland In order to limit the impact of wildland fires on the *building* the *building* shall comply with Sections L102.6.1 through L102.6.3

L102.6.1 Wildland Fires. The provisions of the *International Wildland-Urban Interface Code* shall apply to the construction, alteration, movement, repair, maintenance and use of any building, structure or premises within the wildland interface areas in this jurisdiction.

L102.6.2 Exterior walls. Exterior wall requirements shall be based on the Fire Hazard Severity specified in Table 502.1 in the *International Wildland-Urban Interface Code*.

L102.6.3 Smoke Detection. An automatic smoke detection system shall be installed throughout buildings located within areas designated by the jurisdiction as being a wild land urban interface area.

L103 Reference Standards

<u>ASCE</u>

ASCE 7 Minimum Design Loads for Other Structures ASCE 24 Flood Resistant Design and Construction

<u>ICC</u>

ICC International Wildland-Urban Interface Code (IWUIC)

Reason: This reason statement has the following two segments to explain the reasons for this change: (A) The code change is explained with specific substantiation; and (B) General background information identifying the need for enhanced property protection and functional resilience for to strengthen the built environment;

(A)

The following are reports of dollar loss to property from wind, cold weather and fire disasters.

- The American Society of Civil Engineers reported in Normalized Hurricane Damage in the United States, 1900 2005, National Hazard Review, ASCE 2008, that property damage from hurricanes was 81 billion dollars in 2005.
- The National Weather Service reports that U.S. property damage due to winter storms and ice exceeded 1.5 billion dollars in 2009.
- Fire Losses in the United States During 2009 by the National Fire Protection Association, August 2010 shows that property loss due to structure fires in buildings other than one and two family dwellings was approximately 4.5 billion dollars.

Increasing the stringency of the design criteria of buildings for hazards such as wind, snow or fire results in more robust buildings. Such requirements reduce the amount of energy and resources required for repair, removal, disposal and replacement of building components and systems damaged from these disasters. A further benefit is a reduction in the amount of damaged building materials and content entering landfills.

Additional benefits are enhanced life safety, security and occupant comfort; potentially less demand on community resources required for emergency response; and allowing facilities to be more readily adapted for re-use if there is a change of occupancy in the future.

(B)

Minimum building requirements whether through energy codes, plumbing codes, mechanical codes, zoning codes, or basic building codes, do not encourage truly sustainable buildings. The proposal is one of several that attempt to integrate the concepts of the *Whole Building Design Guide* (WBDG) into the International Building Code as a non-mandatory Appendix. This allows adopting jurisdictions the option of incorporating code requirements into the building code to improve the resilience of the built environment without the need to add another code to the community requirements.

The WBDG, developed in partnership between the National Institute of Building Sciences (NIBS) and the Sustainable Building Industries Council (SBIC), has as its key concepts: accessible, aesthetics, cost-effective, <u>functional/operational</u>, historic preservation, productive, <u>secure/safe</u>, and sustainable.

There are numerous references about the economic, societal, and environmental benefits that result when enhanced functional resilience for <u>resource minimization</u> are integrated into building design and construction. Six examples demonstrating the importance and supporting the concepts are:

1. Natural Hazard Mitigation Saves: An Independent Study to Assess the Future Savings from Mitigation Activities National Institute of Building Sciences Multi-Hazard Mitigation Council - 2005

One of the findings in this report is "The analysis of the statistically representative sample of FEMA grants awarded during the study period indicates that a dollar spent on disaster mitigation saves society an average of \$4." The programs studied often addressed issues and strategies other than enhanced disaster resistance of buildings and other structures. However, more disaster-resistant buildings enhance life safety; reduce costs and environmental impacts associated with repair, removal, disposal, and replacement; and reduce the time and resources required for community recovery.

2. Five Years Later – Are we better prepared?

Institute for Business and Home Safety - 2010

This IBHS report states: "When Hurricane Katrina made landfall on Aug. 29, 2005, it caused an estimated \$41.1 billion in insured losses across six states, and took an incalculable economic and social toll on many communities. Five years later, the recovery continues and some residents in the most severely affected states of Alabama, Louisiana and Mississippi are still struggling. There is no question that no one wants a repeat performance of this devastating event that left at least 1,300 people dead. Yet, the steps taken to improve the quality of the building stock, whether through rebuilding or new construction, call into question the commitment of some key stakeholders to ensuring that past mistakes are not repeated." This report indicates that there is a need to implement provisions to make buildings more disaster-resistant. Clearly this suggests that functional resilience should at least be integrated into the design and construction of sustainable buildings.

3. National Weather Service Office of Climate, Water and Weather Services

National Oceanic and Atmospheric Administration (NOAA) - 2010

Data provided on the NOAA website [www.weather.gov/os/hazstats.shtml] indicates that the average annual direct property loss due to natural disasters in the United States exceeds of \$35,000,000,000. This does not include indirect costs associated with loss of residences, business closures, and resources expended for emergency response and management. These direct property losses also do not reflect the direct environmental impact due to reconstruction after the disasters. Functional resilience will help alleviate the environmental impact and minimize both direct and indirect losses from natural disasters.

4. Global Climate Change Impacts in the United States

U.S. Global Change Research Program (USGCRP) - 2009

The USGCRP includes the departments of Agriculture, Commerce, Defense, Energy, Health and Human Services, Interior, State and Transportation; National Aeronautic and Space Administration; Environmental Protection Agency, USA International Development, National Science Foundation and Smithsonian Institution

The report identifies that: "Climate changes are underway in the United States and are projected to grow. Climate-related changes are already observed in the United States and its coastal waters. These include increases in heavy downpours, rising temperature and sea level, rapidly retreating glaciers, thawing permafrost, lengthening growing seasons, lengthening ice-free seasons in the ocean and on lakes and rivers, earlier snowmelt, and alterations in river flows. These changes are projected to grow." The report further identifies that the: "Threats to human health will increase. Health impacts of climate change are related to heat stress, waterborne diseases, poor air quality, extreme weather events, and diseases transmitted by insects and rodents. Robust public health infrastructure can reduce the potential for negative impacts." Key messages in the report on societal impacts include:

- "City residents and city infrastructure have unique vulnerabilities to climate change."
- "Climate change affects communities through changes in climate-sensitive resources that occur both locally and at great distances."
- "Insurance is one of the industries particularly vulnerable to increasing extreme weather events such as severe storms, but it can also help society manage the risks."

Sustainable building design and construction cannot be about protecting the natural environment without consideration of the projected growth in severe weather. Minimum codes primarily based on past natural events are not appropriate for truly sustainable buildings. Buildings expected to have long term positive impacts on the environment must be protected from these extreme changes in the natural environment. The provisions for improved property protections are necessary to reduce the amount of energy and resources associated with repair, removal, disposal, and replacement due to routine maintenance and damage from disasters. Further such provisions reduce the time and resources required for community disaster recovery.

5. **Sustainable Stewardship -** Historic preservation plays an essential role in fighting climate change ,Traditional Building, National Trust for Historic Preservation - 2008

In the article Richard Moe summarizes the results of a study by the Brookings Institution which projects that by 2030 we will have demolished and replaced 82 billion square feet of our current building stock, or nearly 1/3 of our existing buildings, largely because the vast majority of them weren't designed and built to last any longer. Durability, as a component of functional resilience, can reduce these losses.

6. Opportunities for Integrating Disaster Mitigation and Energy Retrofit Programs

Senate Environment and Public Works Committee Room, Dirksen Senate Office Building, Washington, D.C. - 2010

During this panel discussion a representative of the National Conference of State Historic Preservation Officers noted that more robust buildings erected prior to 1950 tend to be more adaptable for reuse and renovation. Prior to the mid-1950s most local jurisdictions developed their own building code requirements that uniquely addressed the community's needs, issues and concerns. Pre-1950 building codes typically resulted in more durable and robust construction that lasts longer.

The total environmental impact of insulation, high efficiency equipment, components, and appliances, low-flow plumbing fixtures, and other building materials and contents are relatively insignificant when rendered irreparable or contaminated and must be disposed of in landfills after disasters. The US Army Corps of Engineers estimated that after Hurricane Katrina nearly 1.2 billion cubic feet of building materials and contents ended up in landfills. This is analogous to stacking enough refrigerators a fifth of the way to the moon or placing them end to end around the equator of the Earth twice.

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

Staff note: This proposal is one of several proposals adding a new appendix L. The intention of the proponent has been indicated that the contents of the proposals be combined if they should be approved into a single Appendix L Titled "Appendix L, Building Resilience."

S338-12

Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
•			APPENDIX I (NEW)-S-SKALKO-STAFE	ORD doc

S339-12

Proponent: Julie Ruth, P.E., JRuth Code Consulting, representing American Architectural Manufacturers Association (AAMA) (julruth@aol.com)

Revise as follows:

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 1710.5.2. Products <u>in *Risk Category* I or II</u> <u>buildings 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 <u>if one of the following is met:</u>

- 1. The required design pressure for the fenestration product does not exceed 60 psf or
- 2. All glass in the fenestration product is tempered or laminated.

Reason: The appropriate deflection limit for framing members supporting glass has been widely debated within the fenestration industry for many years.

Section 2403.2 of the 2012 IBC requires testing or analysis signed by a registered design professional for any glazed assembly when one or more sides of the glass is not firmly supported. Section 2403.3 defines firm support as not deflecting more than 1/175 of the length of glass edge being supported. These provisions have been in the IBC since its first edition in 2000.

Section 1710.5.1 provides an alternative to the requirements of Section 2403.2 and 2403.3 for fenestration products that are tested and labeled in accordance with AAMA/WDMA/CSA 101/I.S.2/A440. This is based upon the referenced specification providing a level of review equivalent to the requirements of Sections 2403.2 and 2403.3.

All editions of the IRC require exterior windows to be tested and labeled to AAMA/WDMA/CSA 101/I.S.2/A440, or one of its predecessor specifications. The provisions of Section 2403.2 and 2403.3 of the 2012 IBC do not occur in the 2012 IRC, or in any of its predecessor editions.

There has been strong evidence that homes built under the IRC have performed well under high wind conditions. This suggests that AAMA/WDMA/CSA 101/I.S.2/A440 adequately addresses all structural design considerations for fenestration in homes built under the IRC, including deflection of glass supporting framing. Additional requirements for framing deflection of fenestration under these applications do not appear necessary.

The scope of the IRC, however, is limited to one and two family dwellings and townhouses three stories or less in height, in regions with design wind speeds of 110 mph or less. The scope of the IBC is all buildings not included within the scope of the IRC. Hence, the IBC applies to all occupancies, and buildings of much greater height than those addressed in the IRC, including those in regions with much higher design wind speeds.

Use of the alternative provided in Section 1710.5 of the IBC for all fenestration in all buildings regardless of the intended occupancy of the building, its height, type of construction or design wind pressures may not be conservative. Therefore the scope of this alternative is being revisited at this time.

This proposal provides a moderately conservative solution to the question – Just how broadly should this alternative be permitted to be used?

It is appropriate to retain the current alternative to Sections 2403.2 and 2403.3 in the IBC for buildings that are similar to those within the scope of the IRC. This includes buildings of similar height in the same design wind speed region if they are not of an occupancy that is required to be designed to a greater level of stringency by the IBC for other reasons.

The only provisions of the IBC that distinguish design wind pressure of a building based upon its use or occupancy are the Risk Category provisions. Homes built under the IRC are Risk Category II. Risk Category I buildings are designed to a lower level of life safety than Risk Category II. Therefore this proposal limits the application of the alternate in Section 1710.5 to buildings in Risk Category I and II.

Based upon Tables R301.2 (2) and R301.2 (3) of the 2012 IRC, the highest possible required design pressure rating for components and cladding on a home built under the 2012 IRC is 54.4 psf, based upon the following calculation:

Max. DP_{reqd} = 29.1 psf (fenestration < 10 sq. ft. in size in zone 5 – near the corner of the building) X 1.87 (opening 60 feet above grade in Exposure D) = 54.4 psf

This proposal rounds that number up to the next nearest multiple of 10, which is 60 psf.

AAMA/WDMA/CSA 101/I.S.2/A440 permits Performance Class R windows to be rated up to 90 psf even if their framing deflects more than L/175 under design pressure. Therefore, requiring additional testing and analysis by a Registered Design Professional for windows with Design Pressure ratings greater than 60 psf is more conservative than the referenced specification.

This proposal also permits the use of either tempered or laminated glass throughout the fenestration product as an alternate to Sections 2403.2 and 2403.3 in Risk Category I and II buildings. The concern of Sections 2403.2 and 2403.3 is the potential breakage of glass due to deflection of the supporting framing. If the product is glazed entirely with tempered or laminated glass this is less of a concern for 2 reasons.

1) Both tempered and laminated glass are capable of resisting higher loads than a similar thickness of the more commonly used annealed glass. AAMA/WDMA/CSA 101/I.S.2/A440 requires the fenestration products to be glazed with the weakest glass permitted

by ASTM E1300 for that particular opening, prior to testing for resistance to structural load. In most cases that weakest glass would be annealed. The strength characteristics of tempered or laminated glass are typically 2 to 4 times greater than those of annealed glass in the same thickness.

2) If broken both tempered and laminated glass has less of a tendency to result in large shards of glass being flung from the opening than annealed glass. Tempered glass has a tendency to shatter into small pieces with less sharp edges than annealed glass. The plastic interlayer of laminated glass tends to hold any broken or shattered glass in place rather than permitting it to be flung from the opening.

Therefore the use of tempered or laminated glass is considered to provide an additional level of safety, particularly in openings with a required design pressure rating of 60 psf or greater.

This proposal permits the alternate of Section 1710.5 to continue to be used in those applications where evidence exists that it is conservative to do so, while addressing concerns that its use may not be appropriate for all buildings.

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

S339-12	
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Public Hearing: Committee:	AS	AM	D	
Assembly:	ASF	AMF	DF	
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