COMMENTS AND RESPONSES
ON 3rd MEMBER ORGANIZATION BALLOT
January 21, 2009 to March 6, 2009
PROPOSALS

PUC Note: *On March 26, 2009, during the PUC meeting to esolve the comments of the 3rd Member Organization Ballot, the PUC, having consulted with the BSSC Board chair, concluded that a member ballot including YR or N on 15 of the 20 proposals could not be counted in the tally on those items because no explanation or reason for the YR or N votes was provided.*

PART 1 PROPOSALS

Proposal 2-4R2 (Y= 22, YR= 9, N= 3, NV= 11 --91%)

TS2 provided the PUC with a modified proposal that included editorial and substantive changes to be made in response to the comments. The PUC accepted the TS2 responses to the ballot comments and agreed that the proposal would require reballoting (Y=21, N=0, NV=0).

ICSSC (YR): Page 1, Line 32: Replace "effective period" with "effective fundamental period" to be consistant with ASCE 41, otherwise "effective period" needs to be defined.

> ***TS2 Response – Persuasive Editorial.*** *"effective period" will be changed to "effective fundamental period."*

Page 1, Line 39 to page 2, Lines 1 and 2: This sentence should be consistent with ASCE 7-05, Section 16.5.

> ***TS2 Response - Nonpersuasive.*** *The peer review language has not been changed. The intention is that peer review requirements applicable to this provision need not be as demanding as those specified for nonlinear response history analysis specified in Chapter 16 and was the reason for the specific requirements in the exception.*

PCA (No): Here are our comments on Item 2-4 on the 2nd Member Organization ballot: P – Delta – Inadequacy of ASCE 7 provisions has not been demonstrated. This proposal causes hardship for heavy materials such as concrete and masonry, for which vertical load, P_x, is high. The proposal will permit the coefficient theta, θ, to exceed 0.10 only if one is able to jump through major hoops. ASCE 7 is not anywhere near as restrictive.

The above comments have not been resolved to our satisfaction. We are going to vote in favor of editorial changes to a proposal that we are still strongly opposed to.

__TS2 Response – Nonresponsive.__ The requirements of concern to the Member Organization were approved previously. Only the changes to the Exception are the subject of this ballot.

SEAoCC (YR): Editorial: Strike out the word "determined" in the first sentence under EXCEPTION, item 1.

__TS2 Response – Persuasive Editorial.__ This sentence will be revised to read "The resistance to lateral forces is determined to increase continuously in a monotonic nonlinear (static) pushover analysis ~~determined~~ according to…".

Add the following text to the EXCEPTION, Item 1:

Force-deformation curves of materials and components shall comply with Chapters 4 through 9 of ASCE/SEI 41 including Supplement No. 1 and shall include strength degradation.

Comment:
Just referencing Section 3.3.3 of ASCE 41 is not sufficient to implement the Nonlinear Static Procedure for the purposes of determining the effect of PΔ. Rational material and component model parameters must be included to effect strength degradation.

__TS2 Response: Persuasive.__ The exception will be modified to include the phrase "…, and degradation shall be modeled."

CMACN (YR): Our YR vote will be changed to affirmative if the following comment is addressed.

Comment: Concur with SEAOCC (Kersting) comment to add to the EXCEPTION item 1 text as follows: Force-deformation curves of materials shall comply with Chapters 4 through 9 of ASCE/SEI 41 including Supplement No. 1 and shall include strength degradation.

__TS2 Response—Persuasive.__ See response to SEAoCC.

SEAoSC (YR): Same as CMACN.
SEAoNC (YR): Same as CMACN.
NCSEA (YR): Same as CMACN.
SEAoC (YR): Same as CMACN.
SEAoSD (YR): Same as CMACN.

__TS2 Response: Persuasive.__ See response to CMACN.

CASSC (YR): Please add a requirement that the strength degradation provisions of Chapters 3 through 9 of ASCE-SEI 41 shall be included in the nonlinear static (pushover) analysis. The simplified nonlinear static analyses as defined in 3.3.3.2.2 may not sufficiently capture p-delta effects since it does not directly incorporate degradation, so it should be prohibited for use in 12.8.7 for degrading systems. The reason is that some degrading systems could become unstable and drifts could exceed limits prescribed in ASCE 7 if degradation were ignored in the analysis.

__TS2 Response: Persuasive.__ See response to SEAoCC.

PPCI (No): Our "No" comment is similar to our "No" comment for 2-4R (2009) from the 2nd Member Organization Ballot which has not yet been satisfied and is stated here again: "For the precast concrete industry, this proposal will cause a hardship. As the result of precast concrete being a heavy material, the vertical design load (P_x) will be large in the numerator of the equation for the stability coefficient. Especially with the Importance Factor also in the numerator, it will be more difficult for the stability coefficient not to exceed 0.10 resulting in having to follow either of the two exceptions. This proposal is much more restrictive than ASCE 7-05.

Justification for this exception to ASCE 7 has not been provided for other structural systems and materials besides steel structures."

We would change our negative vote to an affirmative vote if this proposal were to be limited to the steel structural systems referred to in the cited references or it has been proven through other cited references that this approach is applicable to other structural systems and materials.

> ***TS2 Response: Nonresponsive.*** *As stated in the response to the ballot of the previous version of this proposal (2-4R): "It appears the voter is concerned with the exception from ASCE-7 that limits θ to 0.10. This change was approved previously. The comment is not relevant to the proposal."*

WRI (No): Note on Proposals Numbered: 2-4R2, 2-4C and IT1-3R – I agree with Dr. S.K. Ghosh (PCA) on his statements of reason. I back him 100% on the above noted proposal numbers. You may use his statements on my ballot.

> ***TS2 Response: Nonresponsive.*** *Dr. Ghosh's reservations were addressed by the PUC following the 2nd ballot.*

Proposal 2-4C (Y= 32, YR= 0, N= 3, NV= 10--91%)

The PUC accepted the TS 2 responses to ballot comments (Y=21, N=0, NV=0).

PCA (No): We are not going to vote in favor of commentary on a proposal that we are still strongly opposed to.

Cited studies are for steel moment frames. No justification is offered as to why requirements apparently necessary for steel moment frames should apply to other materials and systems.

> ***TS2 Response: Nonresponsive.*** *The requirements of concern to the Member Organization were approved previously. Only the changes to the Exception are the subject of this ballot.*

PPCI (No): Please see comment for 2-4R2 (2009) above. Same comment applies to this proposal. We will not vote affirmatively to the commentary of a section in which we have a "No" vote.

> ***TS2 Response: Nonresponsive.*** *The requirements of concern to the Member Organization were approved previously. Only the changes to the Exception are the subject of this ballot.*

WRI (No): Note on Proposals Numbered: 2-4R2, 2-4C and IT1-3R – I agree with Dr. S.K. Ghosh on his statements of reason. I back him 100% on the above noted proposal numbers. You may use his statements on my ballot.

> ***TS2 Response: Nonresponsive.*** *Dr. Ghosh's reservations were addressed by the PUC following the 2nd ballot.*

Proposal 2-5RA (Y=28, YR= 2, N= 0, NV=15 --100%)

The PUC accepted the TS2 responses to comments noting, however, that the reason-for-proposal statements are not published in the Provisions (Y=21, N=0, NV=0).

ICSSC (YR): "Reason for Proposal" statement needs to be updated. Although this chapter aligns with 2-5R5, it is an autonomous chapter**.**

> ***TS2 Response: Persuasive Editorial.*** *The original intent of the proposal was to align Section 18.3 with the changes associated with Proposal 2-5R5. However, the modifications to Section 18.3 do stand alone and the "Proposal for Change" will be modified as follows: "Revise Part 1, Chapter 18, Sec. 18.3 of ASCE 7-05 ~~to align with the changes in Proposal 2-5R5 (Chapter 16)~~ as follows:"*
>
> *In addition, The Reason for Proposal will be modified as follows: "The modifications to Chapter 18 are being made to make the reference to the requirements to Chapter 16 more general rather than to the specific sections. While this proposal does stand on its own merits the changes are ~~requirements~~ consistent with the rewrite of Chapter 16 (Proposal 2-5). ~~As such this proposal is a companion proposal to 2-5 "~~*

SEAoW (YR): The reason statement (page 21) indicates that this proposal is a companion to proposal 2-5 but that proposal is not being balloted. Please verify that this proposal works as a stand alone change.

> ***TS2 Response: Comment noted. No action required.*** *The changes to Chapter 18 are editorial. References are now made to "Chapter 16" in lieu of pointing to specific sections.*

Proposal 3-5R2 (Y=25, YR= 8, N= 2, NV= 10 --94%)

The PUC agreed with the TS3 response to the ICSSC comment (Y=21, N=0, NV=0). After considerable discussion, the PUC found the HSEAC comment nonpersuasive (Y=21, N=0, NV=0). Discussion of the SEAoCC comment resulted in several editorial changes (i.e., adding "Geometric mean" to the definitions of PGAm and PGA as well as replacing 'Geomean" with "Geometric mean" in the map captions. The reason-for-proposal statement was edited to eliminate the discussion of Item 5, which had been deleted in an earlier version of the proposal. The CMACN comment also resulted in several editorial changes accepted by the PUC (Y=21, N=0, NV=0). The PUC accepted the TS3 response to the NAHB comment (Y=21, N=0, NV=0).

ICSSC (YR): Page 2, Line 31: Define "nearby sites." How nearby is nearby, within 1 mile or the next block? Provide examples either here or in Commentary.

> ***TS3 Response: Nonpersuasive.*** *The interpretation of the word "nearby" should be made by the geotechnical engineer on a case-by-case basis. It is noted that the word "nearby" is in the pre-existing Provisions and no modification of that section of the Provisions (Section 11.8.2) has been proposed in the current proposal.*

HSEAC (No): For regions of moderate seismic hazard, the MCE-level PGA should be just the de-aggregated portion of the hazard that results from earthquakes of sufficient magnitude (and thus, duration to result in repetitive cycles of strong motion above threshold) to be capable of generating liquefaction. Asseismic hazard lessens in the moderate regions, the net total MCE has contributions from closer smaller magnitude events that may probabilistically generate the PGA, but are not large enough to initiate damaging liquefaction. Therefore, the proposed procedure would result in disproportionate resources expended for an overestimate of liquefaction hazard in the moderate to low seismic hazard regions. USGS has been capable of performing the de-aggregation of hazard for many years, and should do so in deriving the maps to be used in the proposed procedure.

> ***TS3 Response: Nonpersuasive.*** *Because liquefaction can occur for earthquake magnitudes equal to or greater than 5 and these magnitudes are included in the USGS PSHA, the USGS deaggregation results will include the magnitudes of interest. The geotechnical engineer can interpret the dominant magnitude controlling the PGA hazard from the USGS deaggregation, or has the option of performing a site-specific evaluation to determine another PGA value to use in the liquefaction assessment.*

Unsolicited PUC Member Comment: Do we need to discuss MCE vs. RTE, or will this be a blanket issue?

> ***TS3 Response: Nonpersuasive.*** *No; PGA is for MCE not RTE.*

SEAoCC (YR): YR vote will be changed to affirmative if the primary comment below is followed.

Comments:

If the provisions of SDPRG-1R4 are adopted, then a definition of ground motion for use in this proposal should be made. This definition could be MCE, or for use in geotechnical analyses, the ground motion with a 2% probability of exceedence in 50 years. This modification is important because the fragility curves upon which the RTE are developed do not have meaning with geotechnical applications.

> ***TS3 Response: Persuasive.*** *The definition of ground motion used in this proposal is stated on the new PGA maps, which are attached. The new PGA maps are for MCE ground motions and not for RTE ground motion. Furthermore, the new PGA maps are based on geomean ground motions as directly incorporated in the ground motion attenuation relations used in making the PGA maps, and not the adjusted ground motion for maximum direction of ground motion. The use of geomean PGA values is consistent with the values used in geotechnical practice and that have been used in developing the empirically-based liquefaction potential evaluation correlations that are widely used in practice.*

This Proposal is missing item #5 under Section 11.8.3 (referenced in the Reason for Proposal).

> ***TS3 Response**: Item 5 was deleted from the proposal during final discussion at the PUC. Accordingly, the reference to Item 5 has been deleted from the Reason for Proposal.*

CMACN (YR): YR vote will be changed to affirmative if the primary comment below plus editorial comments item 1 through 5 are addressed.

Comments:

Concur with SEAOCC. If provisions of SDPRG-1R4 are adopted, then a definition of ground motion for use in this proposal should be made. This definition could be MCE, or for use in geotechnical analyses, the ground motion with a 2% probability of exceedence in 50 years. This modification is important because the fragility curves upon which the RTE are developed do not have meaning with geotechnical applications.

> ***TS3 Response:** Please see response to SEAoCC above.*

Editorial comments:

1. Page 1, line 40, Section 11.8.3, item 2: remove "including consequences in item 3," – this is redundant since items 1 through 5 are all required to be addressed.

   ***TS3 Response: Persuasive.** The text of the proposal has been edited accordingly.*

2. Page 2, line 18, Section 11.8.3, item 3: after "reduction in foundation soil-bearing capacity" add "and lateral soil reaction" to make consistent with adding loss of lateral soil reaction for pile foundations.

   ***TS3 Response: Persuasive.** The text of the proposal has been edited accordingly.*

3. Page 2, line 19, Section 11.8.3, item 3: after "soil downdrag and loss in" add "axial capacity and" to address reduction in axial capacity as well as reduction in lateral capacity.

   ***TS3 Response: Persuasive.** The text of the proposal has been edited accordingly.*

4. Page 8, line 9, Commentary Section C11.8, Reason for Proposal: delete ", where PGA is set equal to 0.4 S_s." Reason: Notation is confusing since the change is to use PGA attenuation relations rather than a function of S_s

   ***TS3 Response: Persuasive.** The text of the proposal has been edited accordingly.*

5. Page 8, line 12, Commentary Section C11.8, Reason for Proposal: delete sentence "Setting PGA equal to 0.4 S_s…… as a function of PGA"
   Reason: This may confuse the reader because the objective of this code change is to justify use of PGA rather than 0.4 S_s.

   ***TS3 Response: Persuasive.** The text of the proposal has been edited accordingly with some word retention for clarification.*

SEAoSC (YR): Same as CMACN.
SEAoNC (YR): Same as CMACN.
SEAoC (YR): Same as CMACN.

NCSEA (YR): YR vote will be changed to affirmative if the editorial comments are addressed.

TS3 Response: *Please see responses to Editorial Comments from CMACN*

Editorial comments:

1. Page 1, line 40, Section 11.8.3, item 2: remove "including consequences in item 3," – this is redundant since items 1 through 5 are all required to be addressed.
2. Page 2, line 18, Section 11.8.3, item 3: after "reduction in foundation soil-bearing capacity" add "and lateral soil reaction" to make consistent with adding loss of lateral soil reaction for pile foundations.
3. Page 2, line 19, Section 11.8.3, item 3: after "soil downdrag and loss in" add "axial capacity and" to address reduction in axial capacity as well as reduction in lateral capacity.
4. Page 8, line 9, Commentary Section C11.8, Reason for Proposal: delete ", where PGA is set equal to 0.4 S_s." Reason: Notation is confusing since the change is to use PGA attenuation relations rather than a function of S_s
5. Page 8, line 12, Commentary Section C11.8, Reason for Proposal: delete sentence Setting PGA equal to 0.4 S_s…… as a function of PGA" Reason: This may confuse the reader because the objective of this code change is to justify use of PGA rather than 0.4 S_s.

SEAoSD (YR): Same as NCSEA.

TS3 Response: *Please see responses to Editorial Comments from NCSEA and CMACN.*

NAHB (No): Given the impact of the change to an MCE level ground motion for determining the liquefaction potential, the provisions should provide more guidance for implementing the new requirement. In addition, a probabilistic basis for the "potential" of liquefaction should be specified. On the building design side, the ATC-63 methodology assumes that 10-20% of buildings are likely to collapse due to an MCE ground motion. What is the probability of liquefaction that is acceptable before mitigation measures are required? Is the foundation required to be designed for the MCE level lateral forces as well? The provisions are not clear on this issue.

***TS3 Response: Nonpersuasive**. No guidance is necessary. The procedures to evaluate and mitigate liquefaction potential are independent of the ground-motion level, regardless of whether the ground-motion level is the MCE or 2/3 of MCE. The standard practice is for the geotechnical engineer to determine whether the liquefaction potential is high. This determination can be based on probabilistic considerations, but that decision is best made by the geotechnical engineer and depends on the project. Thus, specifying an acceptable probability in the provisions is not appropriate. If the liquefaction potential is judged to be high, the geotechnical engineer evaluates the possible consequences, such as amounts of settlement and lateral spread. The geotechnical and structural engineer then determine the appropriate measures to mitigate the liquefaction potential, whether they are soil-improvement procedures and/or particular foundation types. The PUC decided that structural issues, such as lateral loads and/or displacements for foundation design in liquefiable soils, did not belong in this section of the provisions.*

In the Reason for Proposal, there is a reference to paragraph 5 in Section 11.8.3 addressing this issue, but the proposal does not contain paragraph 5 or any other language on this topic.

TS3 Response: Persuasive. *The described paragraph 5 refers to a paragraph that was deleted from the proposal during final discussion at the PUC. Accordingly, that paragraph has been deleted from the Reason for Proposal.*

Proposal 6-3R (Y=24, YR= 9, N= 0, NV= 12 --100%)

The PUC accepted the TS6 responses to comments and the need for the proposed changes to be reballoted (Y=21, N=0, NV=0).

ICSSC (YR): Page 2, Line 27: Insert "of the cited reference" after "shown in Figure 1."

TS 6 Response: Editorial. *This figure will either be included in the document or referenced.*

AISC (YR): Comment 1 -- In place of the proposal in Item 2, Section 15.7 of AISC 341-05 should be replaced with the following:

15.7 Beam-to-Column Connections

Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:

(1) The connection shall have sufficient capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height; or

(2) The connection shall be designed to resist a moment simultaneous with the required strength of the brace connection, equal to the lesser of the following:

(i) A moment corresponding to $1.1R_yF_yZ$ of the beam.

(ii) A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column.

Comment 2 -- In place of the proposal in Item 3, Section 16.7 of AISC 341-05 should be replaced with the following:

16.7 Beam-to-Column Connections

Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:

(1) The connection shall have sufficient capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height; or

(2) The connection shall be designed to resist a moment simultaneous with the required strength of the brace connection, equal to the lesser of the following:

(i) A moment corresponding to $1.1R_yF_yZ$ of the beam.

(ii) A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column.

TS 6 Response: Persuasive. *This language is being used in the upcoming edition of AISC 341-10 and will be adopted for consistency.*

AISI (YR): Why do the modifications indicate a change from “rotation of 0.02 radians” to “drift of 2.5%” but the Reason for Proposal refers to a change from “0.02 to 0.025 radians”?

TS6 Response: Editorial. *The Reason for Proposal was in error and should have referred to drift.*

SEAoCC (YR): YR vote will be changed to affirmative if the changes and comment below is addressed.

Under Item 1, permit EBF and BRBF systems with R = 7 (without moment-resisting connections) if those systems comply with one-half the allowable drifts given in ASCE 7-05 Table 12.12-1.

Under Items 2 and 3, add the following text to the end of paragraph (2): “in the presence of the amplified seismic axial load in the beam.”

Comment: Under the proposed Item 1, the elimination of the R = 7 system could make the use of the SCBF system more prevalent, and that system has been shown to have limited ductility capacity possibly inconsistent with its R = 6 assignment. Rather than completely eliminating better performing components, it is proposed to place these more restrictive drift limits to achieve more desirable performance. The reduction of drift will lessen the rotational demands on these beam-to-column connections, which were never qualified for inelastic rotations, which would occur using these systems. By placing more restrictive drift limits on these systems, the questionable performance of the beam-column joint will be mitigated.

Under the proposed Items 2 and 3, the paragraph (2) connection is not a fully restrained moment connection recognized by AISC or any other technical organization and can be connected only to the web of the braced beam section. Under that condition, it is imperative that the connection be **tested** for rotational capacity in the presence of the anticipated real axial load in the brace beam.

TS 6 Response: Nonpersuasive on the first point; persuasive on the second. *Reducing the allowable drift levels to ½ of that required of other systems (including SCBF) will do more to eliminate the use of the proposed systems than requiring fully restrained beam connections in BRBF and EBF systems. It is not clear that limiting the prescribed drift levels to ½ will cause a commensurate reduction in the actual deformation demands on these structures. The type of connections in question have been tested at the University of Washington, UC Berkeley and other locations over the years and have demonstrated the ability to meet the drift levels specified. Under item (1) we allow a pinned connection if it can accommodate the rotation (see the Lehigh tested connection, e.g.). We are not requiring a prequalified or tested connection, so the reference to testing in the comment is misleading. The intent is to include connection detail(s) in the commentary that in all likelihood will be considered pre-qualified without limits. This approach is similar to the reinforced link-to-column detail in the current commentary. The proposed consideration of axial forces on the beam is noted. Since the provisions for both EBF and BRBF require that the beams outside the link (in the case of EBF) and the beam members (in the case of BRBF) be designed to develop the capacity of the link or braces, this force should be accounted for in the design of these members. What is not well defined is the case in which more force comes from the collector than from the brace above. We propose to add* *the following statement to the end of the AISC language noted above: "This moment shall be considered in combination with the required strength of the brace connection and beam connection, including amplified diaphragm collector forces."*

CMACN (YR): YR vote will be changed to affirmative if the comment below is addressed.

Comments:

1. Concur with SEAOCC. Under items 2 and 3, Add the following text to the end of paragraph (2): "in the presence of the amplified seismic axial load in the beam."

TS6 Response: Nonpersuasive. *See response to SEAOCC.*

SEAoSC (YR): Same as CMACN.

TS6 Response: Nonpersuasive. *See response to SEAOCC.*

SEAoNC (YR): Same as CMACN.

TS6 Response: Nonpersuasive. *See response to SEAOCC.*

SEAoC (YR): Same as CMACN.

TS6 Response: Nonpersuasive. *See response to SEAOCC.*

CASSC (YR): Please add "in the presence of amplified seismic axial load in the beam" to the ends of 15.7 (2) and 16.7 (2).

Reason: Connection rotation capacities can be adversely affected by axial loads in beams. Thus, axial loads should be included in the tests of connections pursuant to Subsections 15.7(2) and 16.7 (2)

TS6 Response: Nonpersuasive. *See response to SEAOCC.*

Proposal 6-4R2 (Y=19, YR= 1, N= 10, NV= 15 --67%)

A technical paper (included at the end of the 4th ballot version of this proposal) was distributed to the PUC for review prior to consideration of the ballot results and the TS6 responses.

HSEAC (No): The reason for the proposal states that AISI S110 intends that the R = 3.5 be utilized only for preliminary sizing and not for final design. However, there are no conditional statements to that effect in the proposed new entry in Table 12.2.-1. Nevertheless, it would appear that there is more uncertainty in this particular system's R factor than for other systems.

TS6 Response: Nonresponsive or Editorial. *This comment relates to the reason statement and not the provisions being balloted. However, see Item 2 under the SEAoCC comments.*

The PUC agreed that this comment was nonpersuasive (Y = 20, N = 0, NV = 1). It was agreed, however, that the reason statement was misleading and should not have included the text cited by the commenter. A modified reason appears as part of the fourth ballot proposal.

AISI (YR): Consider making the following modifications:

1. Section 14.1.4.1.1: Instead of “grid lines” in Line 33, use the more widely accepted terminology “independent lines of resistance.”

   TS6 Response: Editorial. *Change has been made.*

2. Section 14.1.4.1.4, Line 16: Based upon additional review of the system, it is recommended that the limit on width-thickness ratio be reduced from 1.58xsqrt(E/Fy) to 1.40xsqrt(E/Fy) for HSS sections. The purpose of this reduction is to take into account the fact that axial load was not applied to the column during testing. According to the 2005 AISC Specification, the values of lambda(p) and lambda(r) for stiffened elements of HSS sections subjected to uniform compression due to bending or compression are 1.22xsqrt(E/Fy) and 1.40xsqrt(E/Fy). The test results of two specimens (Specimens 7 and 9) were compared. Specimens 7 and 9 were classified as noncompact and slender sections, respectively, according to the AISC Specification. Based upon the performance of Specimen 9, it is recommended that the width-thickness ratio be reduced from 1.58xsqrt(E/Fy) to 1.40xsqrt(E/Fy) for HSS sections.

   TS6 Response: Persuasive. *Change has been made.*

3. Section 14.1.4.1.5, Line 28: Modify the sentence to read, “*Where a drift limit is required by the applicable building code*, the design story drift shall not exceed *0.03h*, unless approved by the authority having jurisdiction.” The purpose of the reference to the applicable building code is to take into consideration Footnote C of ASCE 7 Table 12.12-1, which states the following for Occupancy Category I and II buildings: “[c]There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.”

   TS6 Response: Persuasive. *Change has been made.*

Rather than adopt the change recommended in Comment 1 above, the PUC preferred the following language to ensure that all the major columns of SFRS are detailed as SBMF: “The SBMF shall engage all columns supporting the roof or floor above.” The PUC agreed with the change proposed in Comment 2.
Rather than adopt the change recommended in Comment 3, the PUC preferred to make the following modifications to AISI S110, Section D1.3:

- *The first sentence was deleted, since it was not necessary.*
- *The last sentence was moved into the first paragraph.*
- *“In no case shall the design story drift exceed 0.05h” was added to ensure an absolute upper bound on the drift limit.*

The PUC voted to modify the proposal as described above and to submit the proposed changes for reballoting by the BSSC member organizations (Y = 20, N = 0, NV = 1).

SEAoCC (No): Negative vote will be changed to affirmative if this entire proposal is place in Part 3 instead of Part 1.

> ***TS6 Response: Nonpersuasive.*** *It was the intent of the ballot that this system and the associated requirements in AISI S110 be included in Part 1. Please note, placing the proposal in Part 3 does not preclude the system from being considered for adoption in ASCE 7. Also, it is worth noting that this type of structure has been in use since the early 60's and several thousand have been installed in California and through the western region for use as mezzanines, support platforms and as specialty support structures for the aero-space industry and even in state and city facilities. These structures, of many different sizes and configurations, have been installed in high seismic zones through California and have experienced most of the major earthquakes without any evidence of structural damage. The ductility of this type of structure has been helpful in the dissipation of the seismic energy and the purpose of the UCSD testing was to add tested evidence of the structures' performance in earthquakes and aid in the standardization of the system. It is our hope to reference AISI S110 directly in ASCE 7 and the model building codes instead continuing to use the alternate means and methods section of the building code for this type of structure.*

The PUC agreed with TS6 that this comment was nonpersuasive (Y = 19, N = 0, NV = 2).

1. This is a moment connection based upon slip and bolt bearing using snug tight HS bolts loaded in shear only. The field tightened snug tight bolts could result in highly variable frictional force with variable energy dissipation. The bolted connection occurs only in the web of the beam and column and not the flanges. The primary mechanism is bolt hole elongation due to bearing failure in the cold-formed light gage metal.

   TS6 Response: Nonpersuasive. *If bolt over-tightening beyond the snug-tight condition were to occur, it is expected that the story drift will be reduced, which to some extent will offset the increase of the maximum moment developed in the bolted group. To examine the effect of over-tightening on the maximum moment, a sensitivity study through nonlinear time-history analyses of a typical CFS-SBMF was conducted; a suite of 20 earthquake ground motions was used as the input motion. The analysis results are provided in the attached document. Assuming that over-tightening resulted in a 50% increase of the bolt tension, it was concluded that, on average, the maximum story drift was reduced by 3%, while the maximum base shear (or the maximum moment in the bolt group) was increased by 11%. The analysis results also showed that the scatter (i.e., the standard deviation) of the predicted response was reduced when the bolts were over-tightened. It should be noted that the bolt grip length in a CFS-SBMF is very small, thereby the increase in the bolt tension due to over-tightening is also expected to be limited. Together with the observation made from the sensitivity study, it is concluded that the impact of over-tightening is insignificant.*

2. The Reason for Proposal states that the R = 3.5 is for preliminary design only citing a more comprehensive design procedure in AISI S110. This is not typical of all other steel seismic force resisting systems in ASCE 7 and AISC 341 in which a direct design approach is used based upon the applicable R in the code.

   TS6 Response: Nonresponsive or Editorial. *This comment relates to the reason statement, not the provisions. The reason statement was unintentionally confusing. It is* *the intent of the standard that R = 3.5 be used in design, and the designer does not need to iterate on the R value. The value of R was developed based on the high ductility capacity observed from cyclic testing of 9 full-scale beam-column subassemblies. The reason statement has been corrected.*

3. The connection is supposed to be the weak link in this system and is used to protect the column and beam.

   TS6 Response: *Agree.*

   The lack of a specific tension force in the HS bolts could result in excessive frictional forces which when combined with bearing exceed the column or beam design moment capacity.

   TS6 Response: *See response to Comment 1.*

   The column and beam moment capacity is limited by local buckling of the cold-formed steel members which provide little energy dissipation.

   TS6 Response: *This has been considered and is the reason why capacity design is required and the reason for the dimensional limitations on the system's beams and columns.*

   There is a concern that the gravity load carrying portion of the connection is also the primary energy dissipation system for earthquake loads.

   TS6 Response: *Within ASCE 7, Table 12.2-1, this is not unique – consider bearing wall systems and other moment frames.*

4. The $0.03h_x$ design story drift limit set for this seismic force resisting system in Section 14.1.4.1.5 does not comply with ASCE 7 Table 12.12-1 Allowable Story Drift.

   ***TS6 Response:* Nonpersuasive**. *The original value determined from the ATC-63 90% draft study was $0.05h_x$. The more conservative $0.03h_x$ drift limit was recommended and agreed to by the PUC in response to a comment by Aschheim on an earlier version of this proposal (Proposal 6-4R): "TS 6 Response: Persuasive. According to Section 12.12.1 in ASCE 7-05, the allowable drift is 0.025 of the story height for Occupancy Category I or II building structures 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts. For nonbuilding structures, Section 15.4.5 stipulates that drift limit in Section 12.12.1 need not apply if a rational analysis indicates that they can be exceeded without adversely affecting structural stability or attached or interconnected components. Considering that AISI S110 is intended for industrial platforms and that these frames usually do not have partitions and walls, it is suggested that the 0.05h drift limit in Section D1.3 of AISI S110 be reduced to 0.03h."*

5. As the documentation states, this connection and system was intended for industrial platforms and non-building structures and should not be applied to occupied building structures without further study.

   ***TS6 Response:* Nonpersuasive.** *Research did not distinguish between occupied and unoccupied spaces. Therefore, there is no reason to limit applicability of system at this time.*

With respect to Comment 1, the PUC indicated that this system specifies snug-tight bolts for which the nut is turned by the ironworker's spud wrench. It is anticipated that there will be a small pretension in the bolt which will help prevent the nut from backing off, and 10 kips is considered a realistic estimate (and much less than the usual pretension force in a bolt). (See AISI S110, Section D1.2.3.1(1).) This was confirmed by Drs. Uang and Sato who used the trial-and-error procedure to determine the bolt tension for slip resistance. The cyclic responses of 9 full-scale beam-column subassemblies were used for calibration purposes. For bolt tension (T), they observed that T = 10 kips correlated well with the response of Specimens 1 to 7 and T= 25 kips correlated well with the two larger specimens (Specimens 8 and 9).

If bolt over-tightening beyond the snug-tight condition were to occur, it is expected that the story drift will be reduced, which to some extent will offset the increase of the maximum moment developed in the bolted group. To examine the effect of over-tightening on the maximum moment, a sensitivity study through nonlinear time-history analyses of a typical CFS-SBMF was conducted; a suite of 20 earthquake ground motions was used as the input motion. The analysis results are provided in the attached document. Assuming that over-tightening resulted in a 50% increase of the bolt tension, it was concluded that, on average, the maximum story drift was reduced by 3%, while the maximum base shear (or the maximum moment in the bolt group) was increased by 11%. The analysis results also showed that the scatter (i.e., the standard deviation) of the predicted response was reduced when the bolts were over-tightened.

It should also be noted that the bolt grip length in a CFS-SBMF is very small, consequently the increase in the bolt tension due to over-tightening is also expected to be limited. The grip length is critical, because that is the length over which the bolt is strained during the pretensioning procedure. For joints in hot-rolled construction one may have grip lengths of 2 to 4 inches and more; the specifics depend on the structure in question. For such bolt grip lengths the strain can be sufficiently accommodated to produce the requisite force. For joints in cold-formed construction, however, the grip lengths will normally be much smaller than one inch, and the pretensioning strain and the force simply cannot be developed. If one attempts to turn the nut as specified for hot-rolled construction, the strain can only be accommodated through deformations along the threads of the bolt, and the threads will be quickly sheared off (the threads are stripped). So the end result is that there will be no pretension - in fact, there won't be a bolt either.

Therefore, given the small bolt grip length in combination with the observations made from the sensitivity study, it is concluded that the impact of over-over-tightening is insignificant. Finally, it is worth noting that inspection of bolted connection is covered in AISI S110, Section E4 as follows: ***"E4 Inspection of Bolted Connections.*** *Connections shall be inspected to verify that the fastener components are as specified and that the joint plies have been drawn into firm contact. A representative sample of bolts shall be evaluated using an ordinary spud wrench, to assure that the bolts in the connections have been tightened to a level equivalent to that of the full effort of a worker with such wrench."*

With respect to Comment 2, the PUC agreed that the Reason for Proposal can be adjusted as proposed and considered it editorial.

The PUC agreed with the TS6 responses on Comment 3. The third comment had four parts. The PUC found the first part persuasive and the following three parts nonpersuasive.

The PUC agreed with the TS6 response that Comment 4 is nonpersuasive. However, TS 6 noted that language will be added to the commentary of AISI S110 cautioning the user that finishes and nonstructural items need to be designed to accommodate this larger anticipated drift.

The PUC agreed with the TS6 response that Comment 5 is nonpersuaisive.

The PUC vote on the comments as described above was Y =20, N= 0, NV =1.

SEAoSD (No): Same as SEAoCC.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

CMACN (No):
Position: Concur with SEAOCC.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

Negative vote will be changed to affirmative if this entire proposal is placed in Part 3 instead of Part 1

1. This is a moment connection based upon slip and bolt bearing using snug tight HS bolts loaded in shear only. The field tightened snug tight bolts could result in highly variable frictional force with variable energy dissipation. The bolted connection occurs only in the web of the beam and column and not the flanges. The primary mechanism is bolt hole elongation due to bearing failure in the cold-formed light gage metal.

2. The Reason for Proposal states that the R =3.5 is for preliminary design only citing a more comprehensive design procedure in AISI S110. This is not typical of all other steel seismic force resisting systems in ASCE 7 and AISC 341 in which a direct design approach is used based upon the applicable R in the code.

3. The connection is supposed to be the weak link in this system and is used to protect the column and beam. The lack of a specific tension force in the HS bolts could result in excessive frictional forces which when combined with bearing exceed the column or beam design moment capacity. The column and beam moment capacity is limited by local buckling of the cold-formed steel members which provide little energy dissipation. There is a concern that the gravity load carrying portion of the connection is also the primary energy dissipation system for earthquake loads.

4. The 0.03hx design story drift limit set for this seismic force resisting system in Section 14.1.4.1.5 does not comply with ASCE 7 Table 12.12-1 Allowable Story Drift.

5. As the documentation states, this connection and system was intended for industrial platforms and non-building structures and should not be applied to occupied building structures without further study.

SEAoSC (No): Same as CMACN.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

SEAoNC (No): Same as CMACN.

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

NCSEA (No): Same as CMACN

TS6 Response: Nonpersuasive. *See the above response to SEAoCC.*

SEAoC (No): Same as CMACN.

> ***TS6 Response: Nonpersuasive.*** *See the above response to SEAoCC.*

The PUC agreed with TS 6 that the comments from SEAoSD, CMACN, SEAoSC, SEAoNC, and SEAoC were similar to those of SEAoCC and therefore the PUC decision regarding the SEAoCC comments applies. No PUC vote was needed since no PUC member objected to the TS6 position.

CASSC (No): The assignment of R values for this moment frame system and the performance characteristics are inconsistent with special moment frames. Therefore it should be prohibited from use in SDC D, E, and F and not be called "special."

> ***TS6 Response: Nonpersuasive.*** *The term "Special" used in ASCE 7 Table 12.2-1 implies a high ductility capacity built into the system. The high value (= 8) of R for the Special Moment Frame is partially due to the high system overstrength resulting from the redundant, multistory nature of the frame. The proposed CFS-SBMF system is for one-story application, therefore the system overstrength is low because the frame is expected to behave as a single-degree-of-freedom system. The low value of R (= 3.5) assigned to the CFS-SBMF is consistent with the low system overstrength of the frame. But the ductility capacity is comparable to that in a Special Moment frame.*

The proposed use of an R value for preliminary design while referencing a more sophisticated design method in AISI 110 for final design is not consistent with other similar seismic force resisting systems.

> ***TS6 Response: Nonpersuasive or Editorial.*** *See the above response to SEAoCC Comment 2.*

The potential consequences of over-tightening bolts beyond snug tight require clarification. The requirements should specify a means of preventing over-tightening.

> ***TS6 Response: Nonpersuasive.*** *See the above response to SEAoCC.*

The PUC agreed with the TS6 responses that the CASSC comments are nonpersuasive (Y = 19, N = 0, NV = 2) and that the second and third comments are similar to the SEAoCC comments.

AMCBO (No): The vote for this proposal shall change to "Y" if the following issues are addressed:

1. Typo errors are corrected @ line 16 to reflect proposal that "Overstrength Factor" = 3.0 and "Deflection Amplification Factor" = 3.5.

> ***TS6 Response: Nonpersuasive.*** *That is what the proposal currently states.*

BSSC Staff Note: *It appears on some paper copies that the decimal point was missing on both 3.0 and 3.5. The original proposal had yellow highlighting and it was lightened in printing which inadvertently eliminated or faded the decimal points.*

2. Field tightened "snug tight" bolt without specific tension force will result unpredictable surface contact friction and energy dissipation. If bolts are over-tensioned, it may result in excessive frictional forces and could exceed the beam or column design moment capacity controlled by local buckling of cold-formed steel members with little ductility. Specific tension force or torque in bolt tightening should be prescribed.

> ***TS6 Response: Nonpersuasive.*** *See response to SEAoCC.*

The PUC agreed with the TS6 responses that these comments were either editorial or nonpersuasive (Y = 20, N = 0, NV = 1).

Unsolicited PUC Member Comment: This proposal needs a lot of work.
P1 Ln 30: We use the term "herein amended." Do we really need to appear to write like attorneys? This should be deleted.

> ***TS6 Response: Nonpersuasive***. *The phrase communicates the requirements succinctly.*

P2 Ln 29 We need to define "significant".

> ***TS6 Response: Editorial.*** *Deleted "significant" – it is not needed.*

P2 Ln 33: The use of at all "grid lines" should be either defined or modified.

> ***TS6 Response: Editorial.*** *See response to AISI Bullet #1 for modification.*

P2 Ln 36: Why does this need to be on a "level floor". Is a foundation not appropriate?

> ***TS6 Response: Editorial.*** *Yes, this was the intent, so"or foundation" has been added.*

P3 Ln 23 Why are we referencing the "building code" as opposed to ASCE 7. The IBC references the ASCE 7.

> ***TS6 Response: Nonpersuasive.*** *AISI standards policy is to reference the Applicable Building Code, which is the overriding document.*

P 3 Ln 26: We should reference ASCE 7 not "building code".

> ***TS6 Response: Nonpersuasive.*** *AISI standards policy is to reference the Applicable Building Code, which is the overriding document.*

Although these comments did not necessarily need to be considered, the PUC agreed with the TS6 responses.

Proposal 8-43R (Y= 33, YR= 2, N= 1, NV= 10--97%)

TS8 explained that all the comments were basically the same. The PUC agreed with the TS8 responses finding the comments nonpersuasive (Y=19, N=0, NV=0).

ICSSC (YR): Page 1, Line 2: Two R/C chimney failures observed in Turkey is not sufficient to change R value from 3 to 2.

Page 1, Line 6: Define "substantiated analysis." Is it by tests? Provide comments in Commentary regarding the "spectacular" chimney failures and Turkish chimney design details. Are they comparable to U.S. design pactices?

__TS8 Response: Nonpersuasive.__ See response to SEAoKM. Analysis is not testing. We believe the requirement is clear. The referenced paper in the Commentary provides discussion on the failures.

Unsolicited PUC Member Comment: We need to resolve the issues between ASCE 7 and ACI 307. We should define and limit breach sizes and require FEA for relatively larger breach openings.

__TS8 Response: Nonpersuasive.__ See response to SEAoKM.

MIA (No): First, this change is in error. All chimneys (concrete, clay brick) have been removed from one line of the table, replaced only by concrete chimneys. Other types of chimneys (clay) have been excluded. Second, where is the research to support the change? While the concept may be undisputed, it appears that changing the R value from 3 to 2, a 33% decrease, is entirely arbitray and without merit. If the concept or rewducing the R value based on ductility is valid, perhaps 2.5 or 2.75 may be more accurate. More research is needed to support any change.

__TS8 Response: Nonpersuasive.__ See response to SEAoKM. The ACI 307 committee also believes the R value is too high. The original table entry never included clay masonry chimneys and was written only to apply to cast-in-place concrete chimneys. The lead in phrase "Cast-in-place concrete" applied to silos, stacks, and chimneys. "Clay" chimneys are not specifically in the table. "Clay" chimneys would fall under plain masonry or reinforced masonry.

SEAoKM (YR): I am voting yes with reservations, because this proposal does not reference the recently published ACI 307-08, *Code Requirements for Reinforced Concrete Chimneys.* It also does not agree with ACI 307-08. Almost all industrial concrete chimneys in the US are designed using ACI 307-08, not the *Provisions*. ACI 307-08 uses R=1.5, and has different detailing requirements around openings. The *Provisions* and ACI 307-08 need to be coordinated.

__TS8 Response: Nonpersuasive__. ASCE 7 has developed a modified proposal in coordination with ACI 307 which will soon be balloted by the ASCE 7 Seismic Subcommittee. ACI 307 will be preparing an a coordinated addendum to their standard to match the ASCE 7 proposal using R=2.0 and Ω_o=1.5. These values were based on testing and analysis done by Prof. John Wilson of Austraila. The values also match values current in the CICIND standards (An International Standard) for concrete chimneys and stacks. The design and detailing requirements for the opening from the PUC proposal will be retained in ASCE 7 with some minor modifications (deletion of option b). Because the modified ASCE 7 proposal is not significantly different from the PUC proposal and the ASCE 7 proposal will be balloted around April 15, no change to the PUC proposal is being considered.

Proposal SDPRG-1R4 (Y=17, YR= 6, N= 8, NV= 14 --74%)

The SDPRG chair distributed and explained three handouts in support of this proposal using a ppt presentation. The PUC chair allowed PUC member Jonathan Stewart to explain the concerns of EERI. (See also the detailed SDPRG responses presented following this text box.)

The ICSSC comment to move to proposal to Part 3 and to delete the reference to ATC 63 was found nonpersuasive (Y=20, N=0, NV=1).

The HSEAC comment was found nonpersuasive because the SDPRG had been directed to follow a different strategy in developing the design maps (Y = 20, N=0, NV=0).

The first SEAoU comment highlighted the possibility for confusion but the SDPRG found this comment nonpersuasive in that the Provisions is a resource document and the PUC agreed (Y=19, N=0, NV=2). The PUC accepted the SDPRG conclusion that the second comment about replacing geometric mean was nonpersuasive (Y=20, N=1,NV=0). The third comment highlighted that the maps are hard to read. The SDPRG recommended that this comment be found nonpersuasive because the USGS developed website will provide the needed information and the PUC agreed (Y=21, N=0, NV=0). The SDPRG recommended that the final SEAoU comment be found unpersuasive and the PUC agreed (Y=21, N=0, NV=0).

The EERI comments were addressed in detail. The PUC accepted the SDPRG response and found them nonpersuasive (Y=20, N=1, NV=0).

The PCA comment was accepted; no vote was needed.

The SEAoCC comment was referred to the EERI response and the PUC found it nonpersuasive (Y=20, N=1, NV=0).

The first CMACN comment was deemed similar to the SEAoCC comment and was found nonpersuasive (Y=18, N=0, NV=1). The second comment addressed the possibility for confusion and the PUC again agreed with SDPRG and found this comment nonpersuasive (Y=21, N=0, NV=0). The third comment focused on the impact of the proposed design ground forces on the values for specific cities. The PUC agreed with the SDPRG response that the Provisions addresses the entire United States and found this comment nonpersuasive (Y=20, N=0, NV=1). The fourth comment on economic impacts was found to be nonpersuasive (Y=18, N=0, NV=1). The member organization then offered editorial comments and the first five were were accepted. Items 6 and 7 were deemed substantive; the SDPRG found them nonpersuasive and the PUC agreed (Y=18, N=0, NV=1).

SEAoSC and SEAoNC were the same as CMACN and the PUC agreed with the SDPRG position (Y=18, N=0, NV=1).

The SEAoC comments were determined to be similar to those of CMACN and EERI and the PUC agreed with the SDPRG position finding them nonpersuasive (Y=18, N=0, NV=1).

The PUC agreed with the SDPRG position finding the first two CASSC comments nonpersuasive (Y=19, N=0, NV=0). The third comment was editorial and the change will be made.

The SDPRG concluded that the NCSEA comment was similar to the CMACN comments found nonpersuasive and the PUC agreed (Y=18, N=0, NV=1).

The SEAoSD also were similar to those of CMACN and the PUC agreed with the SDPRG position (Y=18, N=0, NV=1).

The NAHB comments were addressed as a group and found to be nonpersuaive (Y=18, N=0, NV=0).

ICSSC (YR): Agree with the general concept of SDPRG proposal. It is not clear not to be more realistic to place this proposal in Part 3 to provide time for design professionals to use the proposaed changes including "maximum direction ground motion." Furthermore, ATC 63 is not currently available.

> ***SDPRG Response: Nonpersuasive.*** *Proposal SDPRG-1R4 (and related Proposal 2-8R3) should be in Part 1 since not doing so would leave the 2002 maps and the associated process as the main recommendation of the Provisions and would not be consistent with the action taken at the PUC over the past 2 years. Proposed Commentary (and Reason for Proposal section) references the 90% Draft Report of ATC-63 Project, which is available (FEMA P695, April 2008). The final version of the ATC-63 Project Report, scheduled for release in mid-2009, will be substantially the same as the 90% Draft in terms of collapse risk performance objectives and the criteria discussed in the proposed Commentary and Reason for proposal.*

HSEAC (No): The composite single set of RTE maps that results from the overlay of the probabilistic and deterministic hazards should still be included first, with the other three sets of maps as further elaboration to how the composite map set is constructed. Therefore, the single set of composite maps should be in the provisions, and the other three sets of maps in the commentary (opposite from the placement in this proposal.)

> ***SDPRG Response: Not Persuasive (but very sympathetic).*** *To provide transparency to the process (as requested by the PUC), Proposal SDPRG-1R4 includes individual maps of probabilistic and deterministic ground motions (and risk coefficients), and related procedures for combining parameters from these maps, rather than a set of composite RTE maps. This approach is considered appropriate for the Provisions, which is a resource document, but not necessarily for Seismic Code applications. Note: A parallel proposal to ASCE 7 is based on the same technical material, but incorporates a set of composite maps, as suggested.*

SEAoU (YR):

1. The changes in terminology may make perfect sense but are going to cause a lot of confusion to the engineers who are just becoming familiar with the concept of designing for earthquakes. It is going to cause confusion to engineers who are already familiar with earthquake engineering. Will it really improve our understanding of seismic design concepts by changing MCE to RTE? The terms S_{SD} and S_{1D} are going to be mistaken for S_{DS} and S_{D1} respectively.

> ***SDPRG Response: Nonpersuasive.*** *To provide transparency to the process (as requested by the PUC), Proposal SDPRG-1R4 includes individual maps of probabilistic and deterministic ground motions (and risk coefficients), and related procedures for combining parameters from these maps, including changing the name MCE to RTE. This approach is considered appropriate for the Provisions, which is a resource document, but not necessarily for Seismic Code applications. Note: A parallel proposal to ASCE 7 is based on the same technical material, but incorporates a set of composite maps and uses the name MCE, rather than RTE, to avoid confusing the design engineer.*

2. I am a little concerned that the "geometric mean" that has been a totally acceptable method is being replaced by "spectral response in maximum direction" which produces higher responses.

   SDPRG Response: Nonersuasive. *For use in seismic provisions, response spectral acceleration in the maximum direction (i.e., peak resultant of bi-directional response in the horizontal plane) is considered the appropriate parameter for defining ground motion intensity for structural design. Note: Previous versions of the Provisions (and other Seismic Codes and resource documents) did not specify spectral response acceleration in terms of the geometric mean (although this measure of ground motion intensity is used by a number of attenuation functions). Response spectral accelerations based on maximum direction intensity are somewhat larger than those corresponding to geometric mean intensity, but will not necessarily increase seismic design values from those of the 2003 Provisions (when considered with other proposed changes). As noted in Comment No. 4, proposed design ground motions for a Salt Lake City site (Site Class D conditions) are* $S_{DS} = 1.03$ *g and* $S_{D1} = 0.52$*, as compared to* $S_{DS} = 1.15$ *g and* $S_{D1} = 0.70$ *g of the 2003 Provisions (see Tables 3 and 4 of proposed Commentary).*

3. I could not determine what values the C_{RS} and C_{R1} would have from the reduced maps included. I assume that they are in the 0.9 range for Salt Lake City, Utah. A better discussion of how the C_R adjustment transforms the 2% in 50 year spectral response maps to achieve a 1% in 50 year probability of collapse would be helpful. Also, clarify the difference between the "uniform-hazard" maps (2% in 50 year) and the current MCE maps (also 2% in 50 year).

   SDPRG Response: Nopersuasive. *We agree that the values of* C_{RS} *and* C_{R1} *cannot be accurately read from the small scale maps provided with the proposal (and included in the Provisions). However, design engineers and other users will be able to obtain accurate values of these and other mapped parameters from the Ground Motion Parameters Calculator at the USGS web site (see bottom of Page 25 of 59 of proposal). Example values of risk coefficients and other mapped parameters are summarized in accompanying Tables R-1 and R-2. The values of* C_{RS} *and* C_{R1} *for Salt Lake City are 0.82 and 0.81, respectively; although values of risk coefficients are closer to 1.0 for the other WUS cities included in these tables.*

   We feel proposed commentary adequately discusses C_R *adjustment of 2% in 50-year hazard. In proposed Commentary Section 11.4.3, Step 1 (of a three step process) describes adjustment of uniform-hazard ground motions using risk coefficients; proposed Commentary Section C21.2.1 discusses the use of the risk integral to determine risk-targeted ground motions; and proposed Commentary to Chapter 22 refers the interested reader to a SEAOC conference paper (Luco et al., 2007 and a USGS Open-File report (Luco et al., 2009) for detailed development of risk coefficients.*

   We feel the terminology is clear (although complex). The proposed uniform hazard maps of 2% in 50-year ground motions are purely probabilistic maps. The MCE maps of the 2003 Provisions are a composite of probabilistic MCE (2% in 50-year) ground motions and deterministic MCE ground motions. To provide transparency to the process (as requested by the PUC), Proposal SDPRG-1R4 no longer uses the term MCE, and defines a new term RTE to identify composite maps of (risk-targeted) probabilistic and deterministic ground motions.

4. The “NGA” reduces the spectral accelerations, at least in high seismic areas. This seems to counteract the increases due to the change from geometric mean to spectral response maximum direction. The S_{DS} and S_{D1} values presented in Tables 3 and 4 indicate 10% and 22% reductions, respectively in spectral accelerations for Salt Lake City. Other areas will have significant increases.

   SDPRG Response: Nonpersuasive. *(See Tables R-1 and R-2.)*

Table R-1. Proposed short-period design values (S_{DS}) for Site Class D, MCE parameters, return periods and 50-year collapse risk probabilities for 34 city sites

Region	City (Site Location)	Design	MCE Ground Motions				Return Period (years)	50-Year Collapse Rate
		S_{DS} (g)	F_a	S_{SUH} (g)	C_{RS}	S_{SD} (g)		
Southern California	Los Angeles	1.60	1.00	2.55	0.94	2.81	2,077	1.0%
	Century City	1.44	1.00	2.25	0.96	2.40	2,223	1.0%
	Northridge	1.13	1.00	1.95	1.04	1.69	1,504	1.6%
	Long Beach	1.10	1.00	1.73	0.95	2.27	2,130	1.0%
	Irvine	1.03	1.00	1.60	0.97	3.33	2,331	1.0%
	Riverside	1.00	1.00	1.68	1.11	1.50	1,619	1.8%
	San Bernardino	1.58	1.00	3.20	1.01	2.37	947	2.1%
	San Luis Obispo	0.78	1.05	1.16	0.96	1.59	2,236	1.0%
	San Diego	0.84	1.00	1.50	0.84	2.42	1,805	1.0%
	Santa Barbara	1.89	1.00	3.13	0.90	2.96	1,881	1.0%
	Ventura	1.59	1.00	2.54	0.94	3.28	2,075	1.0%
	Weighted Mean	1.22	1.00	2.04	0.97	2.43	1,881	1.2%
Northern California	Oakland	1.24	1.00	2.82	1.03	1.86	656	2.9%
	Concord	1.38	1.00	2.80	0.99	2.08	991	2.0%
	Monterey	1.02	1.00	1.59	0.96	2.23	2,225	1.0%
	Sacramento	0.57	1.26	0.61	1.11	1.50	3,437	1.0%
	San Francisco	1.00	1.00	1.99	1.05	1.50	965	2.3%
	San Mateo	1.23	1.00	2.42	0.96	1.85	1,197	1.7%
	San Jose	1.00	1.00	1.98	1.14	1.50	826	2.9%
	Santa Cruz	1.01	1.00	1.67	1.02	1.52	1,814	1.3%
	Vallejo	1.00	1.00	1.79	1.10	1.50	1,242	2.1%
	Santa Rosa	1.67	1.00	3.36	0.92	2.51	1,112	1.6%
	Weighted Mean	1.08	1.04	2.10	1.05	1.75	1,369	2.1%
Pacific Northwest	Seattle	0.88	1.00	1.39	0.95	2.71	2,182	1.0%
	Tacoma	0.83	1.00	1.26	0.99	1.50	2,383	1.0%
	Everett	0.83	1.00	1.33	0.93	3.44	2,104	1.0%
	Portland	0.72	1.11	1.09	0.90	3.02	1,947	1.0%
	Weighted Mean	0.82	1.04	1.26	0.94	2.71	2,128	1.0%
Other WUS	Salt Lake City	1.03	1.00	1.89	0.82	3.44	1,703	1.0%
	Boise	0.32	1.55	0.33	0.94	1.50	2,163	1.0%
	Reno	1.03	1.00	1.62	0.95	1.65	2,160	1.0%
	Las Vegas	0.46	1.41	0.52	0.93	2.43	2,168	1.0%
	Weighted Mean	0.65	1.28	0.98	0.91	2.48	2,043	1.0%
CEUS	St. Louis	0.42	1.45	0.51	0.87	1.50	1,838	1.0%
	Memphis	0.74	1.10	1.24	0.81	1.50	1,680	1.0%
	Charleston	0.80	1.04	1.46	0.79	2.99	1,747	1.0%
	Chicago	0.14	1.60	0.15	0.92	1.50	2,155	1.0%
	New York	0.29	1.58	0.32	0.87	1.50	2,058	1.0%
	Weighted Mean	0.29	1.54	0.34	0.88	1.53	2,047	1.0%

Table R-2. Proposed 1-second design values (S_{D1}) for Site Class D, MCE parameters, return periods and 50-year collapse risk probabilities for 34 city sites

Region	City (Site Location)	Design	MCE Ground Motions				Return Period (years)	50-Year Collapse Rate
		SD1 (g)	F_v	S_{1UH} (g)	C_{R1}	S_{1D} (g)		
Southern California	Los Angeles	0.84	1.50	0.88	0.96	1.01	2,228	1.0%
	Century City	0.80	1.50	0.84	0.96	1.05	2,240	1.0%
	Northridge	0.60	1.50	0.69	1.04	0.60	1,558	1.6%
	Long Beach	0.62	1.50	0.65	0.96	0.98	2,233	1.0%
	Irvine	0.57	1.50	0.56	1.01	1.24	2,556	1.0%
	Riverside	0.60	1.50	0.67	1.07	0.60	1,657	1.6%
	San Bernardino	1.08	1.50	1.43	0.96	1.08	1,155	1.7%
	San Luis Obispo	0.45	1.57	0.43	0.98	0.60	2,349	1.0%
	San Diego	0.49	1.52	0.56	0.87	1.05	1,940	1.0%
	Santa Barbara	0.99	1.50	1.10	0.90	1.17	1,863	1.0%
	Ventura	0.90	1.50	0.97	0.93	1.27	2,096	1.0%
	Weighted Mean	0.70	1.50	0.77	0.97	0.98	1,993	1.2%
Northern California	Oakland	0.75	1.50	1.07	1.01	0.75	832	2.4%
	Concord	0.73	1.50	0.99	0.98	0.73	1,054	1.9%
	Monterey	0.56	1.50	0.59	0.95	0.93	2,189	1.0%
	Sacramento	0.35	1.81	0.26	1.12	0.60	3,805	1.0%
	San Francisco	0.64	1.50	0.85	0.99	0.64	1,064	1.9%
	San Mateo	0.86	1.50	1.06	0.92	0.86	1,441	1.3%
	San Jose	0.60	1.50	0.72	1.09	0.60	1,262	2.0%
	Santa Cruz	0.60	1.50	0.64	0.98	0.60	2,012	1.1%
	Vallejo	0.60	1.50	0.65	1.08	0.60	1,838	1.5%
	Santa Rosa	1.04	1.50	1.42	0.90	1.04	1,135	1.5%
	Weighted Mean	0.65	1.54	0.81	1.02	0.71	1,616	1.7%
Pacific Northwest	Seattle	0.52	1.50	0.56	0.93	1.07	2,056	1.0%
	Tacoma	0.49	1.51	0.52	0.95	0.60	2,128	1.0%
	Everett	0.48	1.53	0.52	0.92	1.00	2,012	1.0%
	Portland	0.44	1.58	0.48	0.87	1.20	1,814	1.0%
	Weighted Mean	0.48	1.53	0.52	0.91	1.03	1,984	1.0%
Other WUS	Salt Lake City	0.54	1.50	0.67	0.81	1.55	1,785	1.0%
	Boise	0.17	2.38	0.11	0.97	0.60	2,292	1.0%
	Reno	0.52	1.50	0.55	0.95	0.60	2,137	1.0%
	Las Vegas	0.24	2.14	0.17	0.98	0.63	2,385	1.0%
	Weighted Mean	0.34	1.93	0.33	0.93	0.87	2,186	1.0%
CEUS	St. Louis	0.24	2.13	0.20	0.83	0.60	1,717	1.0%
	Memphis	0.40	1.70	0.44	0.80	0.60	1,706	1.0%
	Charleston	0.41	1.67	0.45	0.81	0.91	1,865	1.0%
	Chicago	0.10	2.40	0.07	0.87	0.60	1,850	1.0%
	New York	0.11	2.40	0.08	0.91	0.60	2,129	1.0%
	Weighted Mean	0.14	2.34	0.11	0.88	0.61	1,992	1.0%

SDPRG Response (to SEAoU Comment 4): Nonpersuasive. *We agree that the "NGA" relations tend to reduce the spectral accelerations in Western United States (WUS) regions, although these are not always high seismic areas, and that such reductions tend to offset increases associated with the proposed change from geometric mean to maximum direction intensity. However, proposed ground motions are also significantly influenced by other proposed changes to seismic hazard (e.g., fault characteristics) and the use of risk-targeted ground motions. For example, conversion to 1% in 50-year collapse risk reduces ground motions substantially for Salt Lake City ($C_R \approx 0.8$). Proposed ground motion values reflect the combined effects of proposed changes to the underlying science (work by the USGS), incorporation of risk-targeted concepts and use of maximum direction intensity.*

As shown in Table R-3, design values based on proposed ground motions compare well, on average, with design values of current and prior editions of Seismic Codes on a regional basis. Proposed design values for California tend to be a little greater, on average, than those of ASCE/SEI 7-05, but lower, on average, than those of the 1997 UBC. Proposed ground motions for some California areas (e.g., Santa Barbara) are significantly greater due to changes to seismic hazard (USGS science). Proposed ground motions for Other WUS areas, including Boise, Reno and Las Vegas, as well as Salt Lake City, are about the same or a little lower, on average, as those of ASCE/SEI 7-05. In the CEUS, proposed ground motions are significantly less (at short-periods), on average, from of those of the 2005 Provisions (ASCE/SEI 7-05) due to changes in seismic hazard and adjustment for risk.

Table R-3. Regional comparison of design values (S_{DS} and S_{D1}) based on Proposal SDPRG-1R4 with design values of current (ASCE/SEI 7-05) and older Seismic Codes (Site Class D)

Region (No. of City Sites)	2.75*Z	C_a	S_{DS} - ASCE/SEI 7			What If Geomean?
	1994 UBC	1997 UBC	7-02 (98)	7-05	SDPRG-1R4	
Southern Calif. (11)	1.10	1.25	1.06	1.16	1.22	1.11
Northern Calif. (10)	1.06	1.18	1.01	1.00	1.08	0.98
Pacific Northwest (4)	0.83	0.90	0.90	0.84	0.82	0.74
Other WUS (4)	0.68	0.80	0.72	0.70	0.65	0.59
CEUS (5)	0.31	0.40	0.39	0.36	0.29	0.26
All - Average (34)	0.69	0.80	0.72	0.73	0.72	0.65
Region (No. of City Sites)	1.25*ZS	C_v	S_{D1} - ASCE/SEI 7			What If Geomean?
	1994 UBC	1997 UBC	7-02 (98)	7-05	SDPRG-1R4	
Southern Calif. (11)	0.75	0.83	0.63	0.65	0.70	0.54
Northern Calif. (10)	0.73	0.81	0.64	0.61	0.65	0.50
Pacific Northwest (4)	0.56	0.54	0.46	0.44	0.48	0.37
Other WUS (4)	0.47	0.46	0.41	0.39	0.34	0.26
CEUS (5)	0.21	0.24	0.16	0.14	0.14	0.11
All - Average (34)	0.47	0.52	0.39	0.38	0.40	0.31

EERI (No): Letters from EERI follow:

February 23, 2009

Mr. David R. Bonneville
EERI Liaison to BSSC
Degenkolb Engineers
235 Montgomery Street, Suite 500
San Francisco, CA. 94104
E-Mail: dbonne@degenkolb.com

RE: EERI Member ballot for proposed revisions to NEHRP Provisions: PROPOSAL SDPRG-1R4 (2009)

Dear David:

I am writing to ask you to cast a "NO" vote for EERI on Proposal SDPRG-1R4 (2009). This was a very difficult decision which was reached only after I received a unanimous recommendation to vote as such from an advisory group of experts that I asked to render an opinion on how EERI vote regarding this matter should be cast. I have attached a copy of the letter from this group for your information and for sharing with others as necessary.

As described in the attached letter, EERI's sole concern is with the provision in Proposal SDPRG-1R4 (2009) modifying the basis of horizontal ground motion specification from geometric mean to maximum component. If this issue is subsequently addressed by the BSSC Provisions Update Committee and rectified, EERI would then be happy to support the proposal as modified.

I am keenly aware that a number of distinguished EERI members have had a leading role in preparing Proposal SDPRG-1R4 (2009) and that is what makes this decision particularly difficult. However, in good conscious, for the reasons stated in the attached letter, EERI cannot support this proposal as it stands.

Please confirm the receipt of this letter and your willingness to cast EERI vote as indicated in this letter. If you have any questions or concerns, please do not hesitate to contact me. Thank you for all you have done and continue to do for EERI.

Sincerely yours,

Farzad Naeim
President

cc: Executive Committee of the Board; members of the advisory group; Mr. William Holmes (EERI Liaison-Elect to BSSC).

499 14TH Street, Suite 320, Oakland, California 94612-1934
☎ (510) 451-0905 FAX (510) 451-5411 eeri@eeri.org http://www.eeri.org

February 20, 2009

Dr. Farzad Naeim
EERI President
Email: FARZAD@johnmartin.com

RE: Member ballot for proposed revisions to NEHRP Provisions: PROPOSAL SDPRG-1R4 (2009)

Dear President Naeim:

We are writing you in your capacity as the EERI President who is responsible for casting EERI's vote on various proposals for revision of the ASCE 7-05 provisions. Our task group was appointed by you to provide a recommendation regarding Proposal SDPRG-1R4 (2009) on behalf of EERI. Proposal SDPRG-1R4 concerns revisions to the procedures by which ground motions are specified in the ASCE 7-05 Provisions. Three general categories of revisions are being proposed: (1) revisions that modify the basis of ground motion specification from probabilistic seismic hazard analysis to risk-based analysis focusing on structural collapse; (2) modifying the basis of horizontal ground motion specification from geometric mean to maximum component; and (3) modification of the basis for deterministic motions from a 1.5 factor on the median to median + one standard deviation.

Our concern is with the second revision above. Past practice has specified ground motions using the geometric mean, which is the average of the horizontal motions in natural log units. The proposed change is to use the maximum possible horizontal component for both deterministic and probabilistic assessments of ground motion. For any individual recording, this is obtained by rotating the two horizontal components and finding the rotation angle that provides the maximum component. This process is repeated for each spectral period. This maximum component is typically larger than the average by factors of about 1.1 to 1.4 (see Table 1 on Page 35 of the proposal). The rotation angle that leads to the maximum will, in general, be different for each spectral period.

Why do we oppose this change? Other than sites very near faults (within about 3 to 5 km), the orientation of the maximum component is random and cannot be predicted. Hence, the probability that the weak direction of a building will happen to coincide with the orientation of the maximum component is low. From a probabilistic standpoint, the most likely ground motion component oriented with any particular direction in a building is the geometric mean. If we specify something larger than geometric mean such as maximum component, we are increasing the ground motion level arbitrarily. In a probabilistic analysis, this amounts to a significant and unspecified increase in the return period of the earthquake ground motions. Using ground motions with unknown probability levels compromises the validity of probabilistic analysis. Furthermore, ground motions with unknown probability levels cannot be used in risk analyses, so the proposed change to the maximum component is inconsistent with the move to a risk-based approach (Item 1 listed above). For deterministic ground motions, using the maximum rotated motions leads to a spectrum that could not occur in a single acceleration history because the maximum component will occur at different rotation angles for different spectral periods. Using unrealizable spectra is a step backwards. Ground motions for design should be realistic and reasonable (e.g., have a known return period). Using the maximum rotated component meets neither of

499 14TH Street, Suite 320, Oakland, California 94612-1934
☎ (510) 451-0905 FAX (510) 451-5411 eeri@eeri.org http://www.eeri.org
a nonprofit corporation

these objectives. We therefore object to the philosophical underpinning of the change and to the unnecessary increase of mapped ground motions that would result from it.

Based on the above, we strongly recommend that EERI vote <u>no</u> on this proposal. If this issue is subsequently addressed by the BSSC Provisions Update Committee, we would then support the proposal. Thank you for considering our point of view on this important issue.

Respectfully submitted,

Norm A. Abrahamson, Ph.D.   Yousef Bozorgnia, Ph.D., P.E.   Marshall Lew, Ph.D., G.E.

Michael Mehrain, Ph.D. S.E.   Jack P. Moehle, Ph.D., P.E.   Jonathan P. Stewart, Ph.D., P.E.

SDPRG Response: Nonpersuasive. *The EERI letter provides a number of comments, but does not make a specific suggestion for change (as required by ballot instruction). However, from the comments, it would appear that EERI would support changing "maximum" to "geomean" in Section 21.2.1 and revising uniform-hazard and deterministic maps of Chapter 21, accordingly. While we understand concerns regarding use of the maximum direction (of bi-directional response) as the ground motion intensity parameter, and the related increase in ground motions, particularly for certain California areas, we do not believe that use of the geomean would be appropriate. Clearly, there is no perfect measure of ground motion intensity, but it is our opinion that the maximum direction is a better scientific and engineering measure than geomean for structural design. The following sections discuss this preference and address specific points and comments of the EERI letter.*

Maximum Direction Intensity - Simple Definition and Record Orientation Independent

The EERI letter states that maximum direction is found by rotating records. This statement is flawed. Rotating records is not necessary and implies a potential misunderstanding on the part of the authors of the EERI letter. The maximum direction is simply the peak resultant response of a bi-directional elastic SDOF (5%-damped) system at the period of interest. Peak resultant response is independent of record orientation and there is no need to rotate records. There is only one, unique, value of maximum direction intensity at any given period. In contrast, the geometric mean is dependent on the orientation of the record and generates different values for different orientations. This concern regarding the use of the geometric mean led to a study and paper "Orientation-Independent Measures of Ground Motion" (Boore, Watson-Lamprey, Abrahamson, BSSA, 2006) that developed and proposed two definitions of the geometric mean, denoted as GMRotDpp and GMRotIpp, respectively, that are record orientation independent, but require complex calculations to implement (and are hard to understand even by earthquake experts). One version of the orientation-independent definition, GMRotI50, was calculated for records of the PEER NGA database and the results used to develop the new NGA relations.

In summary, the geometric mean is not desirable because it is record-orientation dependent and the GMRotI50 definition is too complex and arbitrary for use in Seismic Codes. Maximum direction is both simple and record-orientation independent.

Maximum Direction Intensity - Physically Realizable Response (of SDOF system)

The EERI letter states that the geometric mean is the average of horizontal motions in log unit. This statement is misleading, assumes that spectral ordinates alone are sufficient to characterize ground motion, and ignores the temporal correlation (or lack thereof) between spectral responses to orthogonal components of ground motion. For illustration, consider the two components of horizontal ground motions from the 1999 Kocaeli earthquake (Duzce record) shown in Figure R-1. In this case, the two components are shown in their as-recorded orientation. The first step in calculating the geometric mean is to separately calculate the response of 5%-damped SDOF system to each component at the period of interest. This is shown in Figure R-2 for response at a period of 1-second and the strongest portion of 1-second response is shown in Figure R-3. The next step involves finding the peak response of each component that generally occur at different points in time, as shown in Figure R-3. The geometric mean is then calculated as the square root of the product of the two, non-coincident (absolute) values *of peak response. The geomean may be a convenient method of combining peak responses from the two orthogonal directions, but it has no physical meaning. From a statistical standpoint, averaging the two orthogonal directions, reduces some of the scatter (uncertainty) associated with peak response of an individual component. That is, two records with the same geomean intensity could have very different values of peak response in the two directions (at different points in time).*

In contrast to the geometric mean, the maximum direction combines the two directions of response in a physically realizable process, that is, as the resultant response of a bi-directional SDOF. Figure R-4 illustrates this calculation by plotting coupled response of the 1-second system in the X-Y plane. Note: Rotating the axes (but not the record trace) changes the peak response values of the two components (C1 and C2) that are measured along their respective axes, but not the peak resultant (maximum direction intensity). We recognize the inherent limitations of response spectra that are based on peak response of a linear-elastic SDOF system (hardly the most realistic representation of most structures), but given these limitations, feel that the maximum direction intensity provides a more appropriate and realistic measure of peak response at the period of interest.

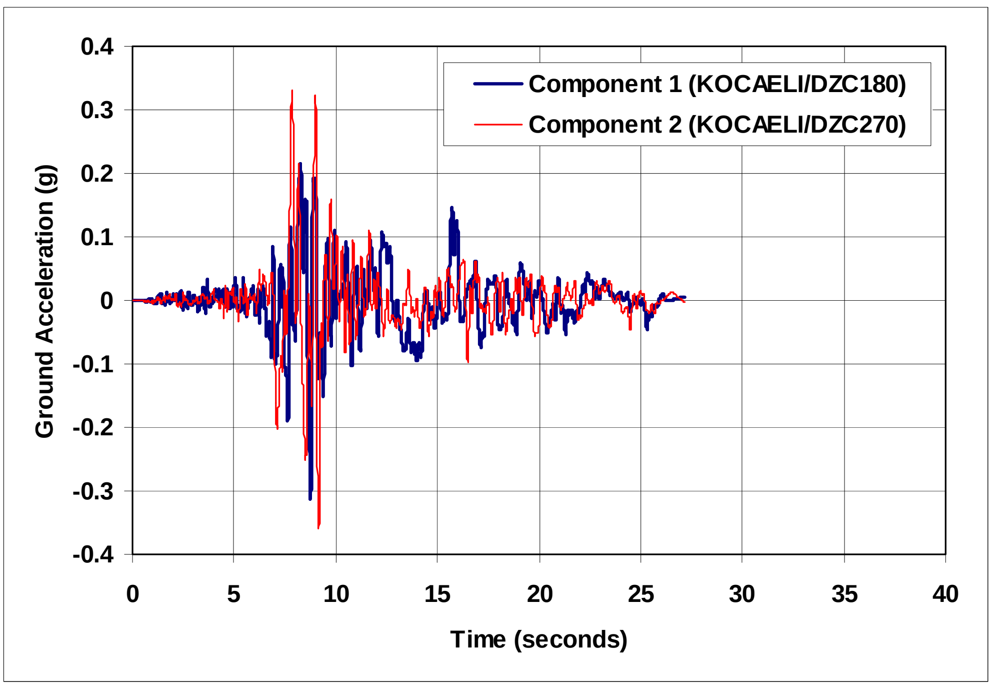

Figure R-1. Example – Ground Acceleration from the 1999 Kocaeli Earthquake – Duzce Record

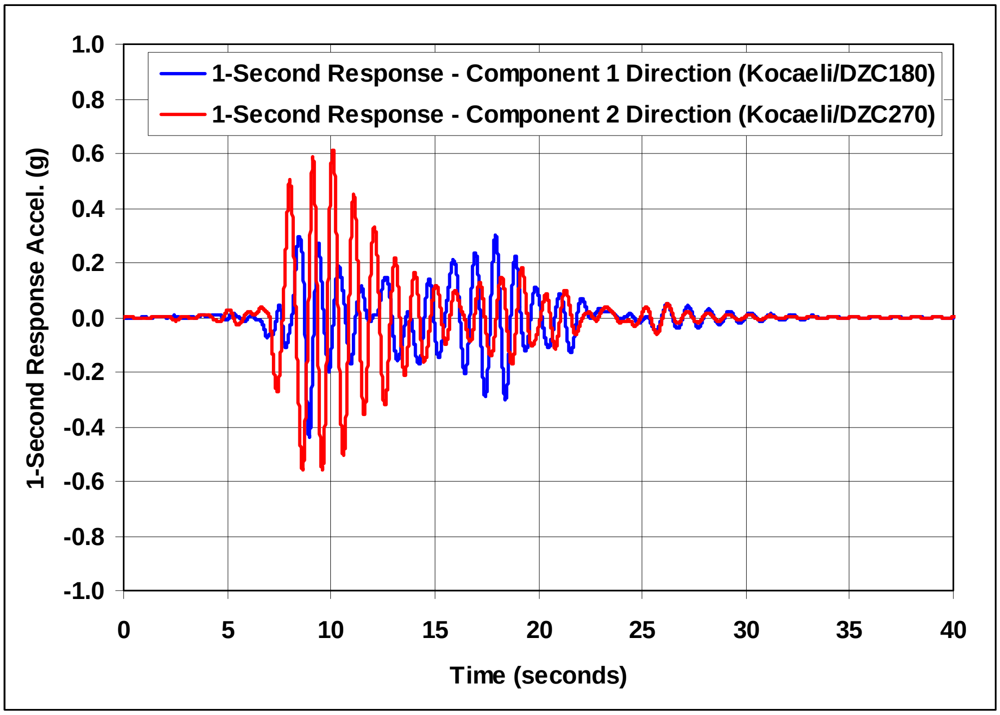

Figure R-2. Example – 1-Second Response of SDOF (5% Damped) System to 1999 Kocaeli Earthquake – Duzce Record

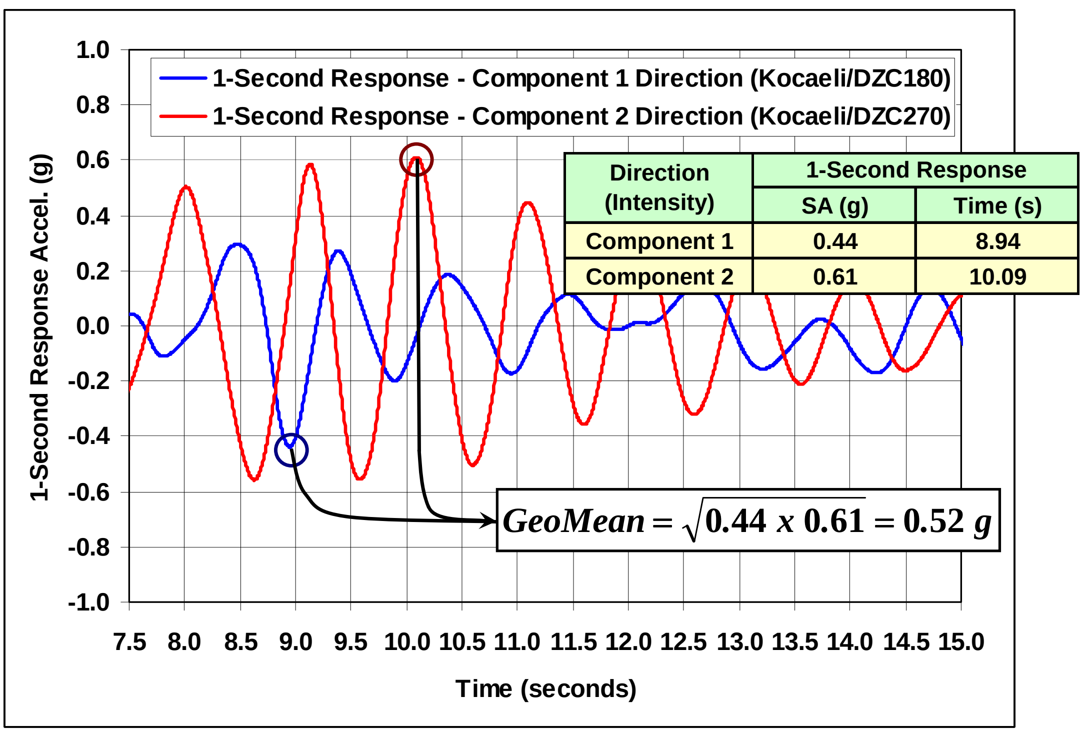

Figure R-3. Example – Calculation of Geometric Mean Intensity for 1-Second Response of the Kocaeli-Duzce Record

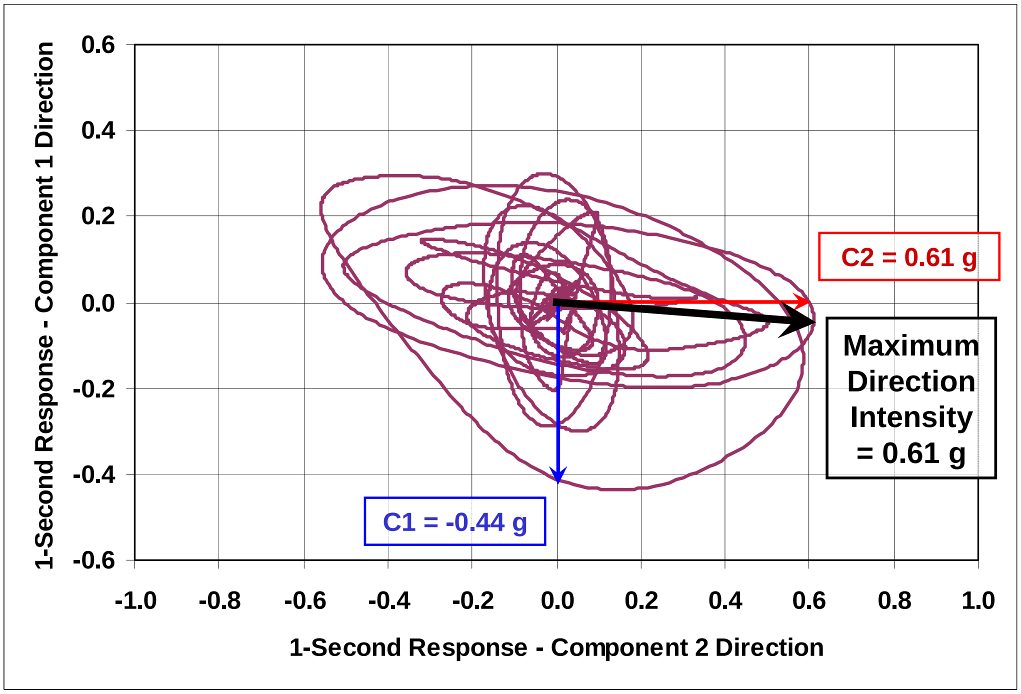

Figure R-4. Example – Calculation of Maximum Direction Intensity for 1-Second Response of the Kocaeli-Duzce Record (X-Y Plane)

Maximum Direction Intensity - Period Dependency of Resultant Vector

The EERI letter states that the angle of the resultant maximum direction vector will in general, be different for different periods. No evidence is provided by the authors of the EERI letter to support this statement. Indeed, a study of 147 near-fault ground motions recorded from events with magnitudes greater than 6.5 at site-to-source distances of 15 km and less shows that there is an axis of strong shaking over which the spectral demands are very close to the maximum spectral demands over a wide range of period ("Orientation of Maximum Spectral demand" by Huang, Whittaker and Luco, to appear in the August 2009 issue of Earthquake Spectra.

Second, response spectra used to define seismic design values for ELF design require only response at the design period (so relationship with other periods is moot).

Further, the shapes of response spectra based on uniform hazard (with or without risk adjustment) are problematic for rare ground motions and/or for sites with hazard significantly influenced by multiple sources. Baker and Cornell (2006) have shown that rare ground motions in the Western United States, such as those corresponding to the MCE, have a distinctive spectral shape that, for a given fundamental-period spectral acceleration, causes the record to be less damaging than other records of less intensity. In essence, the shape of the spectrum of rare ground motions drops off more rapidly at periods both greater and less than the fundamental period of interest (i.e., has less energy), as compared to spectra of other (less rare) records. The point here is that there are more basic problems with the shape of the spectrum (and the relationship of spectral response at different periods), than those which may be caused by use of maximum direction intensity.

Maximum Direction Intensity - Approximately Same Return Period as Geomean

The EERI letter states that the use of maximum direction, rather than geomean, intensity would result in a significant and unspecified increase in return period. This is not accurate (and implies a potential misunderstanding on the part of the authors). Consider, for example, the Kocaeli-Duzce record shown in Figure R-1. The example shows that the 1-seond geometric mean intensity of this record is 0.52 g, based on the as recorded orientation of the record, and that the maximum direction intensity is 0.61 g (see Figure R-4). We agree that the value of maximum direction intensity is greater than the geomean intensity for this record as must be the case because of the definition of geomean, but we do not agree that this record is any different because one measure of intensity (maximum direction) has a different value from that of another measure of intensity (geomean).

We have proposed an approximate method for converting hazard based on GMRotI50 (and geometric mean) intensity to maximum direction intensity based on the study referenced in the proposal (Huang, Whittaker and Luco), since hazard functions do not exist at this time for the maximum direction. For example, proposed 1-second hazard for the maximum direction is based on 1.3 times 1-second geomean-based hazard. If the process used by the USGS to calculate hazard was repeated, only using ground motion relations representing maximum direction response, rather than geomean response, then such hazard maps would of course be different. However, we would expect 1-second hazard functions of maximum response to have approximately the same annual frequency at a given value of maximum response, as that of 1-second hazard functions of geomean response at a value of geomean response equal to maximum response divided by 1.3.

For reference, approximate return periods are given in Table R-1 for short-period systems and in table R-2 for 1-second period systems for each of 34 city sites. For city sites controlled by probabilistic hazard, these values reflect the effective return period after adjustment for 1% in 50-year collapse risk. In general, the effective return period for probabilistic regions is about 2,000 years. Contrary to the contention of the EERI letter, we consider these return periods to be both known and reasonable (for RTE, formerly MCE, ground motions).

Maximum Direction Intensity - Not Contrary to Risk-based Approach

The EERI letter states that the use of maximum direction, rather than geomean, intensity would be contrary to probabilistic analysis and a risk-based approach. This is not accurate (and implies a potential misunderstanding on the part of the authors). First, our primarily concern as members of the BSSC PUC is life-safety and related collapse risk (not probabilistic studies of ground motions). Second, nothing precludes use of the maximum direction intensity (rather than the geomean or other intensity measure) for evaluation of collapse risk, provided analysis methods and ground motions are scaled accordingly. In this case, we have proposed a 1% in 50-year collapse objective based on idealized structure fragility anchored to a conditional 10% collapse probability given RTE (MCE) ground motions occur. The 10% conditional risk objective comes from the ATC-63 Project and the use of maximum (rather than geomean) intensity to define RTE (MCE) *hazard is consistent with the nonlinear dynamic (time history) analysis methods of that project. However, as noted previously, conversion to geomean is always possible for any project that would require ground motions to be based on geomean intensity.*

For reference, values of the probability of collapse in 50 years are given in Table R-1 for short-period systems and in Table R-2 for 1-second period systems for each of 34 city sites. These collapse probabilities are based on idealized structure collapse fragility and other criteria of the proposal and commentary. Values of 1% indicate city sites controlled by probabilistic hazard. Contrary to the contention of the EERI letter, we consider the proposed ground motions and underlying concepts to be compatible with (probabilistic) risk-based methods and provide example return periods and collapse probabilities for 34 city sites demonstrating proposed concepts.

Maximum Direction Intensity - Conservatism Appropriate for ELF Design

The EERI letter states that the use of maximum direction, rather than geomean, intensity would cause unnecessary increase of mapped ground motions (and, presumably, related design values). We agree that the use of maximum direction intensity would increase ground motions (by 30% at 1-second) over those based on the geometric mean, all else equal, but disagree strongly that this is inappropriate for Seismic Codes when considered with other proposed changes. Our reasons include the following four points: (1) changes to ground motions proposed by SDPRG-1R4 do not substantially increase or decrease, ground motions from those of ASCE/SEI 7-05, on average, for all regions of greatest seismic risk, (2) a companion proposal (Proposal 2-8R3) modifies time history scaling requirements for 3-D analysis to account for ground motion parameters based on the maximum direction so that time histories are not conservatively biased, (3) conservatism provided by maximum direction ground motions is considered appropriate and necessary for 2-D (ELF) design, and (4) paucity of recorded data for large magnitude events at sites relatively close to fault rupture (conditions that dominate collapse risk) require caution when considering radical reductions in ground motions for use in Seismic Codes. These four points are discussed below.

<u>Proposed Ground Motions Same as those of ASCE-SEI 7-05 - On average</u>

Table R-3 summarizes proposed regional averages of design values for 34 city sites and compares these value with those of ASCE/SEI 7-05 and other Seismic Codes. As shown in this table, the average of proposed short-period design values (S_{DS}) for all regions is 0.72g, as compared to an average ASCE-SEI 7-05 value of 0.73 g; and the average of 1-second design values is 0.40 g, as compared to an average ASCE/SEI 7-05 value of 0.38 g. Proposed values are, on average, essentially the same as those of ASCE/SEI 7-05, and substantially less than the corresponding design coefficients of the1997 UBC, which was used until very recently for seismic design in California and other WUS areas. While proposed design values is some areas of certain regions, such as Southern California, are indeed higher (due to changes in the underlying hazard), the trend is not universal and design values in other regions are smaller.

<u>Scaling of Ground Motions to Maximum Direction Intensity</u>

Proposal 2-8R3 modifies time history scaling requirements for 3-D analysis to account for ground motion parameters based on maximum direction intensity so that time history is not conservatively biased. Proposal 2-8R3 reduces the scaling factor from 1.3 to 1.0 (times the target spectrum); offsetting the 1.1 (short-period) and 1.3 (1-second) increases in geomean ground motions to represent maximum direction grounds motions. Thus, records used for time history analysis are appropriately scaled to ground motions based on maximum direction response.

Appropriate Conservatism for ELF Design

The use of design values based on the maximum direction is considered appropriate for ELF design which does not consider simultaneous application of two horizontal directions of seismic input. Based on nonlinear (incremental) dynamic analysis of light-frame wood, reinforced- concrete special moment frame and reinforced-concrete ordinary moment frame structures, performed as part of the ATC-63 Project, the approximate difference in collapse margin of a structure loaded along only one horizontal axis at a time and the same structure simultaneously loaded along both horizontal axes, is 1.2, on average (e.g., see Table VI of Christivalisilis, Filiatrault, Constantinou and Wanitkorkul, EESD, 2008). That is, the intensity of ground motions must be about 20% stronger to collapse the structure when applied on only one axis, in contrast to the more realistic simultaneous application of ground motions on both horizontal axes. Proposed use of design values based on maximum direction intensity provides additional margin appropriate for ELF design (which considers only one horizontal direction at a time).

Paucity of Large Magnitude Records Relatively Close to Fault Rupture

Large magnitude events are rare, and few existing earthquake ground motion records are strong enough to collapse large fractions of modern, code-compliant buildings. In the United States, strong-motion records date to the 1933 Long Beach earthquake, with only a few records obtained from each event until the 1971 San Fernando earthquake. Even with many instruments, strong motion instrumentation networks (e.g., Taiwan and California) provide coverage for only a small fraction of all regions of high seismicity. Considering the size of the earth and period of geologic time, the available sample of strong motion records from large-magnitude earthquakes is still quite limited (and potentially biased by records from more recent, relatively well-recorded events). For example, the PEER-NGA database is composed of over 3,550 ground motion recordings that represent over 160 seismic events (including aftershock events) ranging in magnitude from M4.2 to M7.9, but only 11 records from 6 events have a magnitude $M_w > 7.0$ on a strike-slip source mechanism, recorded at sites less than 20 km from fault rupture with typical site conditions (180 m/s < v_{s30} < 760 m/s) by instruments accurate to at least 4 seconds. These 11 records and their key site and source properties are listed in Table R-4. For use in Seismic Codes, it is essential to understand the limited number of the records that matter the most to building collapse performance.

Table R-4. Properties of (all) 11 Strike-Slip Records in the PEER NGA Database of $M_w > 7$, $D_f < 20$ km, 180 m/sec < $v_{s,30}$ < 760 m/s

Earthquake			Source Characterisitcs				Site Conditions	
Year	Name	Record Station	Mag. (M_w)	Distance D_f (km)		Fault Mechanism	Site Class	$v_{s,30}$ (m/sec.)
				BJ	Campbell			
1992	Landers	Coolwater	7.3	19.7	20.0	Strike-slip	D	271
1992	Landers	Joshua Tree	7.3	11.0	11.4	Strike-slip	C	379
1992	Landers	Lucerne	7.3	2.2	3.7	Strike-slip	C	685
1999	Kocaeli, Turkey	Arcelik	7.5	10.6	13.5	Strike-slip	C	523
1999	**Kocaeli, Turkey**	**Duzce**	**7.5**	**13.6**	**15.4**	**Strike-slip**	**D**	**276**
1999	Kocaeli, Turkey	Yarimca	7.5	1.4	5.3	Strike-slip	D	297
1999	Duzce, Turkey	Bolu	7.1	12.0	12.4	Strike-slip	D	326
1999	Duzce, Turkey	Duzce	7.1	0.0	6.6	Strike-slip	D	276
1990	Manjil, Iran	Abbar	7.4	12.6	13.0	Strike-slip	C	724
1999	Hector Mine	Hector	7.1	10.4	12.0	Strike-slip	C	685
2002	Denali, Alaska	TAPS Pump St. #10	7.9	0.2	3.8	Strike-slip	D	329
Average Value of Eleven Records			**7.37**	**8.5**	**10.6**			**434**

The new NGA relations benefit from the additional records obtained from the 1999 Kocaeli and Ducze earthquakes (5 records in Table R-4) and the only very large-magnitude, very close to fault rupture, strike-slip record (at Pump Station # 10) from the 2002 Denali earthquake. It is somewhat surprising that the new NGA relations predict substantially lower ground motions at periods of 1-second (about 30% lower) than those of predecessor attenuation functions, although the 1999 and later records are generally stronger than the (4) pre-1999 records.

Figure R-5 compares response spectra of the 11 recorded ground motions with NGA ground motions predicted for the same magnitude, fault type, site-source distance and site shear wave velocity as each record (using a PEER spreadsheet, and the same relations/weighting factors as those used by the USGS to develop hazard functions). Two important points. First, mean NGA ground motions are slightly lower, on average, but reasonable representations of the mean of recorded ground motions. Second, the mean of recorded data is not well known due to the paucity of records (only 11 records).

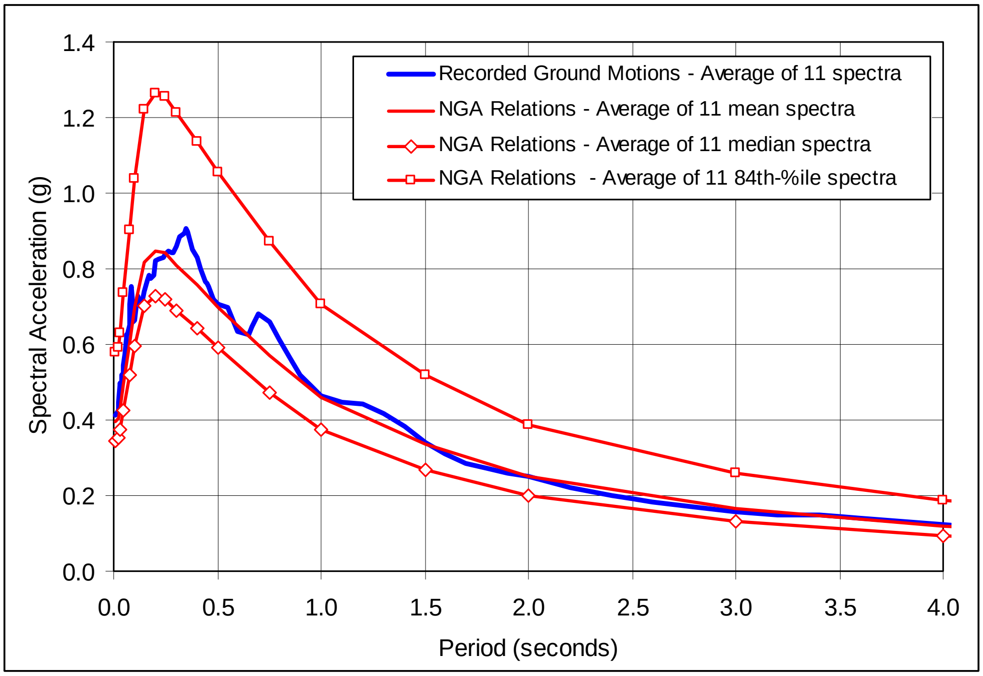

Figure R-5. Comparison of geometric mean spectra – (all) 11 strike-slip records in the PEER NGA database of M_w > 7, D_f < 20 km, 180 m/sec < $v_{s,30}$ < 760 m/s and NGA ground motions based on site/source properties of individual records (Atik. PEER, spreadsheet, 1/3 BA08, 1/3 CB08, 1/3 CY08)

Considering the limited nature of the records most important for building design and the associated potential for significant change in ground motion relations as more records are obtained from future earthquakes, it is considered prudent to include conservatism in the design ground motions of Seismic Codes. In contrast, use of the geomean intensity (rather than maximum direction intensity) would radically reduce design values for 1-second response below those of ASCE/SEI 7-05 (and other Seismic Codes - see right-hand column of Table R-3),

PCA(YR): There are four major parts to this proposal package. Our Yes vote is strongly contingent upon all four of the parts going through. Despite my early skepticism and continued reservations about certain aspects of the proposal, I have come to accept that the overall package represents a step forward. However, the integrity of the package is important. I understand that strong opposition is developing to one or more parts of the proposal. If one or more parts are stripped off this package, then our vote will change to No.

SDPRG Response: Very Persuasive. *All parts (including new USGS hazard maps, risk-targeted concepts, and maximum direction intensity) are considered an integral part of the proposal, and none of these parts are proposed to be "stripped off this package."*

SEAoCC (No): Considerable dissention is occurring in the geotechnical community regarding this proposal which should be addressed. Based upon their concerns (see comments from the EERI member ballot), a proposal should be provided which results in the use of the geomean for the probabilistic hazard analysis determination while still approximately maintaining the ground motion design acceleration levels used in the recent *NEHRP Provisions*.

SDPRG Response: Nonpersuasive. *See response to EERI concerns. Note. Proposed ground motions based on the maximum direction (rather than geomean intensity) approximate the ground motion levels of current NEHRP Provisions (ASCE/SEI 7-05), on average, over all regions of significant seismic risk (see average of all regions - Table R-3).*

CMACN (No): Concur with SEAOCC. Negative vote will be changed to affirmative if the entire proposal is placed in Part 3 instead of Part 1, and also modification to address the three primary concerns plus editorial comments:

SDPRG Response: Nonpersuasive. *See response to SEAOCC. Proposal SDPRG-1R4 (and related Proposal 2-8R3) should be in Part 1 since not doing so would leave the 2002 maps and the associated process as the main recommendation of the Provisions and would not be consistent with the action taken at the PUC over the past 2 years. Response to comments follows:*

Comments:

1. Section 21.2.1 (Page 7, lines 17 and 18) and Section 21.2.2 (Page 8, lines 3 and 4) – the idea of using "the maximum direction of ground motions" is not accepted. When performing a probabilistic ground motion evaluation, if the maximum direction of ground motion is used as if it is the mean anticipated direction, then the ground motion computed actually has a lower risk than the stated probability. Mathematically, the integration of various parameters, including direction of shaking, must use a distribution defined by a mean and a standard deviation – the use of the largest direction in place of the mean skews the results. In addition, the "maximum direction" occurs in different directions at different periods, meaning that the end result is a spectrum in an undefined direction that is a conglomeration of worst-case directions. For time-history analyses based on the spectrum, this will result in time-histories that do not represent a possible ground motion in one direction. It is suggested that if the objective is to provide higher ground motions for design, that the risk level be modified or another mechanism be applied. If the objective is to assure that unidirectional ELF procedures do not use underestimated forces, than it is suggested that an additional factor be applied for that type of analysis.

   SDPRG Response: Nonpersuasive. *See response to EERI comments regarding use of the maximum direction. Note: With respect to concerns regarding "probabilistic analysis", we are proposing ground motions for use in Seismic Codes, not for performing probabilistic analyses (even if we are using risk concepts to establish appropriate hazard levels).*

2. Introduction of risk target coefficient may be valid for performance base design. This concept can potentially complicate design considerations. While the model code has been in existence since 2000, the wide use of the provisions in many areas including California is not more than two years. Building officials need to get used to the 2006 IBC and 2009 IBC before newer concept can be accepted. The force level we are designing nowadays is substantially higher in some geographic locations than before current maps were in used.

   SDPRG Response: Nonpersuasive. *We do not agree that the introduction of risk-targeted coefficients will complicate design considerations. Although complex, the formulas proposed for Chapter 11 provide transparency to the process (as requested by* *the PUC). This approach is considered appropriate for the Provisions, which is a resource document, but not necessarily for Seismic Code applications. Note: A parallel proposal to ASCE 7 is based on the same technical material, but emulates current NEHRP Provisions (ASCE/SEI 7-05) format and terminology, to avoid confusing the design engineer (and building official). Note also: Proposed ground motions approximate the ground motion levels of current NEHRP Provisions (ASCE/SEI 7-05), on average, for all regions of significant seismic risk, and are generally less than the design coefficients of the 1997 UBC (see Table R-3).*

3. This proposed fails to consider the implication of the anchorage force (equation 12.14-10), which is a direct function of S_{DS}. The anchorage force will increase when S_{DS} is increase, as is shown on Table 3 for 9 out of 12 selected cities in southern California.

   SDPRG Response: Nonpersuasive. *Proposed ground motions are based on broad range of considerations, including implications of increased design forces (e.g., on anchorages). However, we do not agree that proposed ground motions should always be the same, or less, than previous/current Seismic Code values. In the case of Southern California, significant changes to the seismic hazard (i.e., due to changes to fault characteristics, etc.) tend to increase design values more than other regions, which on average do not change significantly from the current NEHRP Provisions (ASCE/SEI 7-05). Referring to Table 3 of the proposal, proposed values of S_{DS} are about equal to (i.e., within ± 5%) of current values in 14 of the 34 city sites, significantly greater than current values in 9 of the 34 city sites and significantly less than current values in 11 of the 34 city sites.*

4. Provision should consider economical impact throughout different parts of the United States, and in particularly in Southern California where the design force would increase substantially.

   SDPRG Response: Nonpersuasive. *While we are concerned about the economic impact of proposed ground motions, we disagree that economic concerns should trump life-safety concerns. Proposed ground motions are based on the same concepts and methods for all regions. We disagree that Southern California should be treated differently from other regions (because design forces increase in certain areas of Southern California).*

Editorial comments:

1. Section 11.3, definition for CRS: change "at a period of 0.2 s" to "at short periods" to be consistent with description of the short period parameters used.

   SDPRG Response: Persuasive Editorial.

2. Section 11.3, definition for S_{1UH} (Page 2, line 15) – add "at" after "acceleration parameter"

   SDPRG Response: Persuasive Editorial.

3. Section 11.4.3 (Page 3, line 37) – change "at a period of 0.2 s" to "at short periods" to be consistent with description of the short period parameters used.

   SDPRG Response: Persuasive Editorial.

4. Section 11.4.3 (Page 4, line 5) – add "at" after "acceleration parameter"

SDPRG Response: Persuasive Editorial.

5. Section 11.4.4 (Page 5, line 6) – change “and a 1 s period” to “and at 1-s period”

    SDPRG Response: Persuasive Editorial. *Changed “and a 1 s period” to “and at a 1-s period”.*

6. Figures 22-1 and 22-2. These figures were computed using “maximum direction” of ground motion. Either revise figures to utilize geomean, or revise note, since the risk is lower than the stated 2% in 50-Year uniform hazard. If maximum component is used, the notes should be changed to indicate that the modifications result in a lower risk of occurrence than stated, such as “Hazard level for arbitrary direction of shaking is lower than 2% probability of occurrence in 50 Years because of modification by use of maximum direction of acceleration.”

    SDPRG Response: Nonpersuasive. *Not Editorial. See response to EERI comments.*

7. Figures 22-3 and 22-4. These figures were computed using mean plus 1 standard deviation in the attenuation relation. This should be indicated in the notes: “Ground motion values contoured on these maps are for the mean plus one standard deviation in the ground motion attenuation relation, and with the mean estimate of Magnitude as a function of rupture length.”

    SDPRG Response: Nonpersuasive. *Explaining how the deterministic ground motion maps were computed takes too much text to be included in the figure notes (the suggested note is not fully accurate), but it is explained in Section 21.2.2 (site-specific procedure) and its commentary. Furthermore, explaining how the deterministic ground motions maps were computed in the figure notes would be inconsistent with the figure notes for the uniform-hazard and risk coefficient maps. The purpose of the figure notes is not to explain how these maps were computed.*

SEAoSC (No): Same as CMACN.

> ***SDPRG Response: Nonpersuasive.*** *See response to CMACN.*

SEAoNC (No): Same as CMACN.

> ***SDPRG Response: Nonpersuasive.*** *See response to CMACN.*

SEAoC (No): Negative vote will be changed to affirmative if the entire proposal is placed in Part 3 instead of Part 1, and also modification to address the three primary concerns plus editorial comments:

> ***SDPRG Response: Nonpersuasive.*** *See response to CMACN and EERI comments, as indicated below:*

Comments:

1. Section 21.2.1 (Page 7, lines 17 and 18) and Section 21.2.2 (Page 8, lines 3 and 4) – the idea of using “the maximum direction of ground motions” is not accepted. When performing a probabilistic ground motion evaluation, if the maximum direction of ground motion is used as

if it is the mean anticipated direction, then the ground motion computed actually has a lower risk than the stated probability. Mathematically, the integration of various parameters, including direction of shaking, must use a distribution defined by a mean and a standard deviation – the use of the largest direction in place of the mean skews the results. In addition, the "maximum direction" occurs in different directions at different periods, meaning that the end result is a spectrum in an undefined direction that is a conglomeration of worst-case directions. For time-history analyses based on the spectrum, this will result in time-histories that do not represent a possible ground motion in one direction. We suggest if the objective is to provide higher ground motions for design, the risk level need to be modified or another mechanism be applied. If the objective is to assure that unidirectional ELF procedures do not use underestimated forces, than it is suggested that an additional factor be applied for that type of analysis.

SDPRG Response: Nonpersuasive. *See response to CMACN Comment 1.*

2. We also concur with observation from EERI in their position letter dated February 20, 2009 that other than for very near fault sites, the orientation of the maximum components are random and not predictable.

   SDPRG Response: Nonpersuasive. *See response to EERI comments regarding use of the maximum direction. We recognize that the ground motions based on maximum direction response will be used for ELF design in a conservative manner, but consider this design conservatism appropriate, as discussed in our response to EERI.*

3. Introduction of risk target coefficient may be valid for performance base design. This concept can potentially complicate design considerations. While the model code has been in existence since 2000, the wide use of the provisions in many areas including California is not more than two years. Building officials need to get used to the 2006 IBC and 2009 IBC before newer concept can be accepted. The force level we are designing nowadays is substantially higher in some geographic locations than before current maps were in used.

   SDPRG Response: Nonpersuasive. *See response to CMACN Comment 2.*

4. This proposed fails to consider the implication of the anchorage force (equation 12.14-10), which is a direct function of S_{DS}. The anchorage force will increase when S_{DS} is increase, as is shown on Table 3 for 9 out of 12 selected cities in southern California.

   SDPRG Response: Nonpersuasive. *See response to CMACN Comment 3.*

5. Provision should consider economical impact throughout different parts of the United States, and in particularly in Southern California where the design force would increase substantially.

   SDPRG Response: Nonpersuasive. *See response to CMACN Comment 4.*

Editorial comments: The same seven as were submitted by CMACN.

SDPRG Response: See the responses to same seven editorial comments submitted by by CMACN.

CASSC (No):

1. Use of GMRotI50 is not consistent with Next Generation Attenuation relationships that are based on geometric mean values and are used for creating ground motion maps. The BSSC should coordinate with NGA to ensure that a future proposal has fully coordinated and consistent ground motion definitions.

   SDPRG Response: Nonpersuasive. *The commenter appears confused; the NGA functions are based on the GMRotI50 definition of ground motion intensity (which is a very complex version of the geometric mean).*

2. It is premature for the BSSC to base the definition of risk-targeted earthquake ground motions on a single mean probability of collapse since, for most of the seismic force resisting systems in ASCE 7, substantiating research that documents their probabilities of collapse are not yet available. By defining RTE as such, the BSSC's implied warranty appears too precise and potentially unrealistic given what the BSSC knows about the performance of engineered structures and their variability. The definition of RTE could be misconstrued by the public and policymakers as implying that earthquake engineers know much more about the mean value and variation of the collapse potential of building systems than is currently available. Additional provisions and commentary are needed to clearly communicate the limitations of this proposed approach. More work needs to be completed on the vulnerability aspects of this issue before the BSSC can confidently base risk-targeted ground motions on collapse probability. Seismic design provisions and other provisions in the materials sections including those controlled by other standards development organizations must be calibrated or developed to ensure that proposed probabilities of collapse can be realistically achieved before this proposal can be justified. The BSSC should call for cooperation from the material standards organizations to help meet this goal before resubmitting this proposal. The provisions and commentary in this proposal are currently and inappropriately silent regarding this matter.

   SDPRG Response: Nonpersuasive. *We disagree that it is premature to use a risk-based approach to establish an appropriate level of ground motions for seismic design (since these concepts date from the time of ATC-3), and recognize that structure collapse capacity is not necessarily the same as that of the idealized collapse fragility curve used in this proposal to establish risk-targeted ground motions. We do not feel that the use of risk methods implies a warranty on performance, but by establishing clear risk objectives of a 1% probability of collapse in 50 years and a conditional 10% probability given RTE (MCE) ground motions we are, for the first time, quantitatively defining acceptable collapse performance of structures. The subject proposal deals with ground motions which are only a part of total equation. The other side of the equation, building capacity, must be addressed by projects such as ATC-63 that use the same conditional risk objective to establish appropriate design values of seismic performance factors.*

3. As an editorial comment, the definition for "Risk-Targeted (RTE) Ground Motions" should be revised to include the word "Earthquake" so that it reads: "Risk-Targeted Earthquake (RTE) Ground Motions." Otherwise there is no reason for the "E" in the acronym.

   SDPRG Response: Persuasive Editorial. *Typo, which is in Section 11.2 (only), corrected.*

NCSEA (YR): YR vote will be changed to Y if the following comments are addressed:

> ***SDPRG Response: Nonpersuasive.*** *See response to CMACN comments, as indicated below:*

Comments:

1. Section 21.2.1 (Page 7, lines 17 and 18) and Section 21.2.2 (Page 8, lines 3 and 4) – forcing the use of "the maximum direction of ground motions" is not consistent with the supporting science. When performing a probabilistic ground motion evaluation, if the maximum direction of ground motion is used as if it is the mean anticipated direction, then the ground motion computed actually has a lower risk than the stated probability. Mathematically, the integration of various parameters, including direction of shaking, must use a distribution defined by a mean and a standard deviation – the use of the largest direction in place of the mean skews the results. In addition, the "maximum direction" occurs in geographically different directions at different periods, meaning that the end result is a spectrum in an undefined direction that is a conglomeration of worst-case directions. For time-history analyses based on the spectrum, this will result in time-histories that do not represent a possible ground motion in any single geographical direction. It is suggested that if the objective is to provide higher ground motions for design, that the risk level be modified or the methodology increases the ground motion level through the use of a justifiable mechanism. If the objective is to assure that unidirectional ELF procedures do not use underestimated forces, than it is suggested that an additional factor be applied for that type of analysis.

    SDPRG Response: Not Persuasive. *See response to CMACN Comment 1.*

2. Introduction of risk target coefficient may be valid for performance base design, but could complicate design of systems under current regulations. While the model code has been in existence since 2000, many jurisdictions, including all of California, has been using the model code for only two years. Building officials may struggle to transition from 2006 IBC and 2009 IBC to this radically different concept. Further clarification of purpose (via commentary) may be warranted.

    SDPRG Response: Nonpersuasive. *See response to CMACN Comment 2. Note: A parallel proposal to ASCE 7 is based on the same technical material, but emulates current NEHRP Provisions (ASCE/SEI 7-05) format and terminology, to avoid confusing the design engineer (and building official).*

3. This proposed fails to consider the implication of the change in ground motion and its effects related to developing anchorage forces (equation 12.14-10; currently a direct function of S_{DS}). Anchorage forces increase when S_{DS} increases, as is shown on Table 3 for 9 out of 12 selected cities in southern California.

    SDPRG Response: Nonpersuasive. *See response to CMACN Comment 3.*

Editorial comments: The same seven as were submitted by CMACN.

> ***SDPRG Response: See the responses to same seven editorial comments submitted by by CMACN.***

SEAoSD (YR): YR vote will be changed to Y if the following comments are addressed:

__SDPRG Response: Nonpersuasive.__ See responses to CMACN comments as indicated below:

Comments:

1. Section 21.2.1 (Page 7, lines 17 and 18) and Section 21.2.2 (Page 8, lines 3 and 4) – forcing the use of "the maximum direction of ground motions" is not consistent with the supporting science. When performing a probabilistic ground motion evaluation, if the maximum direction of ground motion is used as if it is the mean anticipated direction, then the ground motion computed actually has a lower risk than the stated probability. Mathematically, the integration of various parameters, including direction of shaking, must use a distribution defined by a mean and a standard deviation – the use of the largest direction in place of the mean skews the results. In addition, the "maximum direction" occurs in geographically different directions at different periods, meaning that the end result is a spectrum in an undefined direction that is a conglomeration of worst-case directions. For time-history analyses based on the spectrum, this will result in time-histories that do not represent a possible ground motion in any single geographical direction. It is suggested that if the objective is to provide higher ground motions for design, that the risk level be modified or the methodology increases the ground motion level through the use of a justifiable mechanism. If the objective is to assure that unidirectional ELF procedures do not use underestimated forces, than it is suggested that an additional factor be applied for that type of analysis.

   __SDPRG Response: Nonpersuasive.__ See response to CMACN Comment 1.

2. Introduction of risk target coefficient may be valid for performance base design, but could complicate design of systems under current regulations. While the model code has been in existence since 2000, many jurisdictions, including all of California, has been using the model code for only two years. Building officials may struggle to transition from 2006 IBC and 2009 IBC to this radically different concept. Further clarification of purpose (via commentary) may be warranted.

   __SDPRG Response: Nonpersuasive.__ See response to CMACN Comment 2. Note: A parallel proposal to ASCE 7 is based on the same technical material, but emulates current NEHRP Provisions (ASCE/SEI 7-05) format and terminology, to avoid confusing the design engineer (and building official).

3. This proposed fails to consider the implication of the change in ground motion and its effects related to developing anchorage forces (equation 12.14-10; currently a direct function of S_{DS}). Anchorage forces increase when S_{DS} increases, as is shown on Table 3 for 9 out of 12 selected cities in southern California.

   __SDPRG Response: Nonpersuasive.__ See response to CMACN Comment 3.

Editorial comments: The same seven as were submitted by CMACN.

> ***SDPRG Response: See the responses to same seven editorial comments submitted by by CMACN.***

NAHB (YR): NAHB fully supports the work of the USGS and SDPRG in developing these new maps and proposals. The improved science underlying the ground motions, together with the move to a risk-targeted basis, takes long strides in addressing the design and code issues in the CEUS which result from the current maps.

We understand that substantial opposition to this proposal may arise due to increased ground motions in California under the new approach. While we understand and sympathize with the seismic hazards faced by the WUS, and California in particular, I feel the BSSC and the PUC would be doing a disservice to the rest of the country in allowing these local issues to sink an otherwise worthy proposal.

In the event that the issues with California and the WUS prove to be a "fatal flaw", we would urge the SDPRG and the PUC to find a way to separately implement the ground motion and attenuation changes for the CEUS. Even without the move to the uniform-risk approach, the new data represents a substantial improvement in defining the seismic hazards for the New Madrid and Charleston areas.

> ***SDPRG Response: Nonpersuasive (but sympathetic).*** *We agree with the sentiments of the NAHB regarding the CEUS, but would not be able to develop separate WUS and CEUS ground motion proposals at this point in the process. Further, it would not make sense to have different rules for different regions. Project 07 (SDPRG) and the predecessor Project 97 worked hard to develop and maintain consistency in the methods used to develop ground motions for different regions of the US.*

Proposal 2-8R3 (Y= 23, YR= 3, N= 6, NV= 13--81%)

TS2 found the SEAoU comment nonpersuasive and the PUC agreed (Y=18, N=0, NV=0). TS2 found the first SEAoCC comment editorial and the PUC agreed (Y=18, N=0, NV=1). The second SEAoCC comment suggesting this proposal be placed in Part 3 was found nonpersuasive because it is a companion proposal to SDPRG 1R4, which is in Part 1 and the PUC agreed (Y=19, N=0, NV=0). The third comment refers to an as-yet-unpublished PEER study that could require a future change; since time is not available for the PUC/TS2 to address the issue, the PUC found the comment nonpersuasive (Y=18, N=0, NV=1). CMACN's comment requested that this proposal be moved to Part 3 was found nonpersuasive (Y=18, N=0, NV=1). The PUC agreed that remaining comments from CMACN and the following comments from the SEAs and CASSC are similar to the SEAoCC comments and warrant the same responses (Y=18, N=0, NV=1).

SEAoU (YR): On page 1 of 4, line 15, the term " average" should be replace "mean value". The term "average" can have different interpretations.

> ***TS2 Response: Nonpersuasive.*** *The word "average" is not being modified by this proposal. "Average" has been in the Provisions for several cycles and the intent of the requirements is fairly clear. There isn't enough justification to make the change.*

SEAoCC (YR): Spectral matching of ground motion acceleration histories to the design response spectra should be explicitly permitted in addition to the scaling method now given. This proposal also seems closely related to 2-5R5, which is now being considered for part 3. Therefore, these provisions should also be considered for part 3. If they are maintained in part 1, then further coordination with the remaining, unmodified sections of ASCE 7-05 chapter 16 needs to take place. In particular, the provisions of 16.1.4 call for determination of response parameters of a linear RHA by multiplying results by I/R. This needs to be coordinated with the proposal since the scaling under this proposal would be to the MCE spectrum but the derivation of response parameters is typically based on design response spectrum.

TS2 Response: Persuasive Editorial -- Spectral Matching. *This is a companion to proposal SDPRG-1R4 that modifies the scaling approach so that the selected time histories will be consistent with using the maximum direction of ground motion (in lieu of geomean). This change was not meant to prohibit the use of spectral matching as the scaling procedure. The Part 2 Commentary on this section clarifies this point. Still, there is a parenthetical in Section 16.1.3.2 which creates confusion in this regard. To eliminate this confusion, the sentence the following will be eliminated: "For each pair of horizontal ground motion components, a square root of the sum of squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5-percent-damped response spectra for the scaled components (for direct scaling ~~where~~ an identical scale factor is applied to both components of a pair)."*

TS2Response: Nonpersuasive -- Coordination with Proposal 2-5R5. *As stated above, this is a companion to proposal SDPRG-1R4 and should be located in the same "Part" accordingly. Proposal 2-5R5 is a complete re-write of Chapter 16 which is slated for Part 3. In future editions of the Provisions, these three proposals (SDPRG-1R4, 2-8R3 and 2-5R5) will require coordination, especially if they all are located in Part 1.*

Comments:

The *Provisions* imply that scaling of ground motion acceleration histories to the design response spectra using a scale factor times the time history accelerations for a specific period range is the only approved method of matching the response spectra. Notable expert seismologists in this field regularly use spectral matching algorithms to match the time histories to the response spectra across the full period range. We understand there is a PEER ground motion scaling study and a complete re-write of this section to align better with this part and current practice is recommended.

TS2 Response: Nonpersuasive -- PEER Scaling Approach. *The Provisions update schedule does not allow for the completion of the PEER study. Future updates will be able to consider the results from PEER.*

CMACN (No): Concur with SEAOCC. Our negative will be changed to affirmative if this provision is placed in Part 3 instead of Part 1.

TS2 Response: Nonpersuasive. *Proposal SDPRG-1R4 (and related Proposal 2-8R3) should be in Part 1 since not doing so would leave the 2002 maps and the associated process as the main recommendation of the Provisions and would not be consistent with the action taken at the PUC over the past two years.*

Comments:

1. Spectral matching of ground motion acceleration histories to the design response spectra should be explicitly permitted as well as the scaling method now given.

TS2 Response: Persuasive Editorial. *This is a companion to proposal SDPRG-1R4 that modifies the scaling approach so that the selected time histories will be consistent with using the maximum direction of ground motion (in lieu of geomean). This change was not meant to prohibit the use of spectral matching as the scaling procedure. The Part 2 Commentary on this section clarifies this point. Still, there is a parenthetical in Section 16.1.3.2 which creates confusion in this regard. To eliminate this confusion, the* *following will be eliminated: "For each pair of horizontal ground motion components, a square root of the sum of squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5-percent-damped response spectra for the scaled components (for direct scaling~~where~~ an identical scale factor is applied to both components of a pair)."*

2. This proposal is closely related to the SDPRG-1R4 above, which we post strong objection.

   TS2 Response: Persuasive. *Since this is a companion proposal, the final disposition this proposal will be consistent with that of SDPRG-1R4.*

SEAoSC (No): Same as CMACN.
SEAoNC (No): Same as CMACN.
SEAoC (No): Same as CMACN.

TS2 Response: Various. *See response to CMACN.*

SEAoSD (No): N will be changed to Y if this provision is placed in Part 3 instead of Part 1.

TS2 Response: Nonpersuasive. *Proposal SDPRG-1R4 (and related Proposal 2-8R3) should be in Part 1 since not doing so would leave the 2002 maps and the associated process as the main recommendation of the Provisions and would not be consistent with the action taken at the PUC over the past 2 years.*

Comments:

1. Spectral matching of ground motion acceleration histories to the design response spectra should be explicitly permitted as well as the scaling method now given.

TS2 Response: Persuasive Editorial. *This is a companion to proposal SDPRG-1R4 that modifies the scaling approach so that the selected time histories will be consistent with using the maximum direction of ground motion (in lieu of geomean). This change was not meant to prohibit the use of spectral matching as the scaling procedure. The Part 2 Commentary on this section clarifies this point. Still, there is a parenthetical in Section 16.1.3.2 which creates confusion in this regard. To eliminate this confusion, the following will be eliminated: "For each pair of horizontal ground motion components, a square root of the sum of squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5-percent-damped response spectra for the scaled components (for direct scaling~~where~~ an identical scale factor is applied to both components of a pair)"*

CASSC (YR): Has the BSSC coordinated its efforts with PEER's and COSMOS's Ground Motion Selection and Modification studies and recommendations? I encourage rapid incorporation of these more comprehensive and more current techniques into acceptable alternatives to 16.1.3.2 that can be readily worked into future ASCE 7 provisions.

Response: Nonpersuasive: PEER Scaling Approach
The Provisions update schedule does not allow for the completion of the PEER study. Future updates will be able to consider the results from PEER.

NCSEA (No): Our negative will be changed to affirmative if this provision is placed in Part 3 instead of Part 1.

TS2 Response: Nonpersuasive. *Proposal SDPRG-1R4 (and related Proposal 2-8R3) should be in Part 1 since not doing so would leave the 2002 maps and the associated process as the main recommendation of the Provisions and would not be consistent with the action taken at the PUC over the past 2 years.*

Comments:

1. Spectral matching of ground motion acceleration histories to the design response spectra should be explicitly permitted as well as the scaling method now given.

TS2 Response: Persuasive Editorial.
This is a companion to proposal SDPRG-1R4 that modifies the scaling approach so that the selected time histories will be consistent with using the maximum direction of ground motion (in lieu of geomean). This change was not meant to prohibit the use of spectral matching as the scaling procedure. The Part 2 Commentary on this section clarifies this point. Still, there is a parenthetical in Section 16.1.3.2 which creates confusion in this regard. To eliminate this confusion, the sentence including this text will be eliminated as follows:

For each pair of horizontal ground motion components, a square root of the sum of squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5-percent-damped response spectra for the scaled components (for direct scaling~~where~~ an identical scale factor is applied to both components of a pair).

PART 2 PROPOSALS

Proposal 2-111 (Y= 34, YR= 5, N= 0, NV= 7 --100%)

TS2 found the ICSSC comment nonpersuasive stating that this is the commentary for ASCE 7-05 and passage of SDPRG 1R4 would not affect the Part 2 Commentary and the PUC agreed (Y=21, N=0, NV=0). The HSEAC and SEAoCC comments were the same. TS2 found the DSA(CA) comment nonpersuasive and the PUC agreed. The ASME comment was found editorial.

ICSSC (YR): It is recognized that RTE is not included since SDPRG 1R4 has not been passed, and that ASCE 7-10 needs commentary to ballot for ASCE 7-10. However, it would be useful to know how RTE will be handled if SDPRG 1R4 passes in the Reason Statement.

TS2 Response: Nonpersuasive. *The commentary associated with SDPRG-1R4 will be located in Part 1 of the Provisions, where all the new commentaries for proposals passed this revision cycle (2009) will reside. Part 2 is the commentary for the Provisions adopted at the outset of the cycle, which was ASCE 7-05 with some modifications to correlate the differences between the 2003 Provisions and ASCE 7-05. As such, the Part 2 commentary will not be modified to reflect the changes associated with SDPRG-1R4.*

HSEAC (YR): If SDPRG-1R4 passes, will this commentary be re-drafted to the RTE terminology rather than MCE?

TS2 Response: Nonpersuasive. *See response to ICSSC.*

DSA(CA) (YR): In addition, we noted one inconsistency between Proposal 2-111 and Proposal 3-5 as follows:

Proposal 2-111 - Commentary to Chapter 11, Page 11, line 13-14 - the phrase "... consistent with peak ground accelerations of SDS/2.5." is in conflict with language in proposal 3-5, which specifies liquefaction analysis "... consistent with the maximum considered earthquake ground motions."

> ***TS3 Response: Nonpersuasive.*** *Proposal 2-111 is commentary to ASCE 7-05. The Proposal 3-5 commentary, if adopted, would modify Chapter 11 for ASCE 7-10.*

SEAoCC (YR): ***Comment****:* Needs to be updated to the RTE and PGA maps if SDPRG 1R4 is approved.

> ***TS2 Response: Nonpersuasive.*** *See response to ICSSC.*

ASME (YR): Pages 6 and 7 of 12, text descriptive of and including Fig. C11.5-1. The text describes triangles, squares, and circles which do not appear in the figure and the text and figure are garbled. Text and figure need work to clarify.

> ***TS2 Response: Persuasive Editorial.*** *Comment noted. Revisions will be made.*

Proposal 2-112 (Y= 28, YR= 9, N= 0, NV= 8 --100%)

The ICSSC comment is the same as that submitted for Proposal 2-111 and it was found nonpersuasive for the same reason. The EERI comment was found nonpersuasive. The first SEAoCC comment was found nonpersuasive because the commentary adequately addresses this concern. The second comment was found editorial and the change has been made. The remaining comments were similar to those of SEAoCC and the responses are the sames. The PUC accepted the TS2 responses (Y=21, N=0, NV=0).

ICSSC (YR): See comment on Proposal 2-111.

> ***TS2 Response –Nonpersuasive.*** *The commentary associated with SDPRG-1R4 will be located in Part 1 of the Provisions, where all the new commentaries for proposals passed this revision cycle (2009) will reside. Part 2 is the commentary for the Provisions adopted at the outset of the cycle, which was ASCE 7-05 with some modifications to correlate the differences between the 2003 Provisions and ASCE 7-05. As such, the Part 2 commentary will not be modified to reflect the changes associated with SDPRG-1R4.*

EERI (YR): In Section C12.8.6.1 Minimum Base Shear for Computing Drift, it should be noted that Equation 12.8-5 does not apply.

> ***TS2 Response – Nonpersuasive.*** *The requirements in Section 12.8.6.1 state that the analysis for drift must include the prescribed seismic forces in Section 12.8. There is no exception for Equation 12.8.5-1.*

SEAoCC (YR):

1. P61, line 33: Add an equation for separation between buildings.

    TS2 Response – Nonpersuasive. *The standard does not require that a specific formulation be used. It is felt that the commentary adequately describes a recommended method that could be used to determine the separation distance.*

2. P61, line 34, clarify that the separation should allow for maximum inelastic response displacement, which should be determined at critical locations with consideration for both translational and torsional displacements of the structure.

    TS2 Response – Persuasive Editorial*. The text will be changed to read: "…the lateral deflections<u>, which represent the anticipated maximum inelastic deformations including torsion,</u> of the two units…"*

SEAoSD (YR): YR will be changed to Y if the following comments are addressed:

Comments:
1. P61, line 33: Add an equation for separation between buildings.

    TS2 Response – Nonpersuasive. *See response to SEAoCC.*

2. P61, line 34, clarify that the separation should allow for maximum inelastic response displacement, which should be determined at critical locations with consideration for both translational and torsional displacements of the structure.

    TS2 Response – Persuasive Editorial. *See response to SEAoCC.*

CMACN (YR):

1. Concur with SEAOCC. P61, line 33: Add a equation for separation between buildings.

    TS2 Response – Nonpersuasive. *See response to SEAoCC.*

2. P61, line 34, clarify that the separation should allow for maximum inelastic response displacement, which should be determined at critical locations with consideration for both translational and torsional displacements of the structure.

    TS2 Response – Persuasive Editorial. *See response to SEAoCC.*

SEAoSC (YR): Same as CMACN.
SEAoNC (YR): Same as CMACN.
NCSEA (YR): Same as CMACN.
SEAoC (YR): Same as CMACN.

TS2 Response – Nonpersuasive and Persuasive Editorial. *See response to SEAoCC.*

Proposal 8-113R1 (Y= 33, YR= 4, N= 0, NV= 8--100%)

The ICSSC comment was the same as that submitted regarding Proposal 2-111 and the TS 8 response was the same as that of TS2. No response to the comments submitted by SEAoCC, SEAoSC, and SEAoSD. The NEI comment was found nonpersuasive as was the TMS comment. The ASME comment is editorial and the prefix "C" will be added. The remaining comments were found nonpersuaive either because they do not pertain to ASCE 7-05 or were similar to other comments found nonpersuaisive. The PUC agreed with TS8 responses (Y=20, N=0, NV=0).

ICSSC (YR): See comment on Proposal 2-111.

> ***TS8 Response: Nonpersuasive.*** *The commentary provided is based on ASCE 7-05. If SDPRG 1R4 passes, additional commentary, where appropriate, will be added with the new provision.*

SEAoCC (Yes): Well composed commentary chapter on non-structural components.

> ***TS8 Response:*** *Thank you.*

SEAoSC (Yes): Well compose commentary chapter on non-structural components.

> ***TS8 Response:*** *Thank you.*

SEAoSD (Yes): Well composed commentary on nonstructural components.

> ***TS8 Response:*** *Thank you.*

NEI (YR): Proposal 2-113R1 -- Item 13.6.10.3 -- I would like the commentary to further address the different seismic switches specified in ASCE 7-05 and A17.1. If acceptable, I would like the commentary to note that the A17.1 seismic switch is an acceptable alternative where A17.1 is in effect. Both switches, when triggered, initiate a shutdown of the elevator. If both the switch and derailment device have triggered, A17.1 will not allow the elevator to be put back in service without a complete inspection. For overhead traction elevators, both A17.1 and ASCE 7-05 require placement of the switch high in the building. The A17.1 switch has its sensitivity in the vertical direction and set to trigger at 0.15 g. This axis was chosen to initiate elevator shutdown on the earthquake P wave so that, in many cases, it will provide adequate time to stop the elevator and allow passengers to exit the elevator before severe shaking in the building starts. ASCE 7 -05 switch has its sensitivity in the horizontal direction and set to trigger at 0.30 g. IBC 2009 will list the A17.1 seismic switch as an acceptable alternate to the ASCE 7 switch. A proposal has been submitted to ASCE 7 to consider the A17.1 seismic switch as an acceptable alternate as well.

> ***TS8 Response: Nonpersuasive but sympathetic.*** *This is a commentary on ASCE 7-05. What is suggested would be adding a new provision. However, as indicated in the comment, ASCE 7 will soon be balloting a new proposal that will address the seismic switch issue.*

TMS (YR): Chapter 13, page 22, line 11, item 13.4.2 – Replace the reference to "ACI 530" with the "2005 MSJC Code."

TS8 Response: Nonpersuasive. *This is a commentary on ASCE 7-05, which references ACI 530, the the MSJC code.*

ASME (YR): Editorial. For several paragraph numbers (pages 22, 35, 36) the prefix "C" is missing.

TS8 Response: Persuasive - Editorial.

Pages 6 and 7 of 12, text descriptive of and including Fig. C11.5-1. The text describes triangles, squares, and circles which do not appear in the figure and the text and figure are garbled. Text and figure need work to clarify.2-113 YR Editorial - For several paragraph numbers (pages 22, 35, 36) the prefix "C" is missing.

TS8 Response: Nonpersuasive. *This statement appears to reference the Chapter 11 commentary.*

Page 16 of 38, Para. C13.3. In preparation for future recommended changes in ASCE 7, add a paragraph (probably at the end) that states: "In addition to the acceleration and relative displacement demands discussed herein, interaction effects, e.g., mechanical and electrical components in close proximity to other components or structures, such that impacts from seismic displacements can occur should be evaluated. And, some mechanical components, e.g., piping, may be designed with gapped supports to permit operational effects (thermal expansion or contraction) but still limit seismic effects. The impact loads inevitable in design seismic events of gapped supports must be considered."

TS8 Response: Nonpersuasive. *This is a commentary on ASCE 7-05. "Future recommended changes" are outside the scope. However, it is suggested that it be offered as proposed change for the ASCE 7-10 Commentary.*

Page 16 thru 19 of 38, Para. C13.3.1. In preparation for future recommended changes in ASCE 7, add a paragraph (probably at the end) that states: "While the provisions do not presently provide explicit guidance to develop spectra at heights above grade, some guidance might be found in AC156, "Acceptance Criteria for Seismic Qualification by Shake-Table Testing of Nonstructural components and Systems."

TS8 Response: Nonpersuasive. *This is a commentary on ASCE 7-05. "Future recommended changes" are outside the scope. However, it is suggested that it be offered as proposed change for the ASCE 7-10 Commentary.*

Page 19 of 38, Para. C13.3.2. In preparation for future recommended changes in ASCE 7, add a paragraph (probably at the end) that states: "In addition to seismic relative displacements within structures and between structures discussed in paragraphs below, mechanical and electrical components, e.g., piping and conduit, are attached to other components, e.g., vessels and control centers, which are in turn attached to structures. Thus, relative displacements between components and other components, and components and structures must be considered."

TS8 Response: Nonpersuasive. *This is a commentary on ASCE 7-05. "Future recommended changes" are outside the scope. However, it is suggested that it be offered as proposed change for the ASCE 7-10 Commentary*

Pages 31 and 32 of 38, Para. C13.6.1. In preparation for future recommended changes in ASCE 7, add a paragraph (probably at the end) that states: “When designing for the effects of seismic forces and displacements on mechanical and electrical components, the effects of all coincident loads shall be added to the seismic forces and displacements, as appropriate.”

> ***TS8 Response: Nonpersuasive.*** *This is a commentary on ASCE 7-05. “Future recommended changes” are outside the scope. However, it is suggested that it be offered as proposed change for the ASCE 7-10 Commentary*

Page 35 of 38, Para. C13.6.9. It is recommended that on line 45 the words “...those outlined in Section 13.3" be appended with “or as described in Section 13.1.5.”

> ***TS8 Response: Nonpersuasive****. The reference to 13.1.5 is not correct. TC-8 feels that 13.3 is the correct section and the proposed editorial revision does not add additional clarity.*

Page 36 of 38, Para. C13.6.10.3. The para should address the current seismic switch called out by A17.1-2004 *Safety Code for Elevators and Escalators*. Both the A17.1 switch and that specified by ASCE 7 require the elevator to shut down when the switch is triggered. However, the A17.1 switch activates on a vertical axis where the ASCE 7 switch activates on a horizontal axis. Mentioning that it is acceptable to use the A17.1 switch in lieu of the ASCE 7 switch is requested. IBC 2009 has made an exception to permit use of the A17.1 switch and a request for exception to ASCE 7 has also been submitted, but not yet approved.

> ***TS8 Response: Nonpersuasive****. See Response to above to NEI. This is a commentary on ASCE 7-05. What is suggested would be adding a new provision.*

Page 38 of 38. The reference cited on page 38, lines 12 and 13, “Housner, G. W., and M. A. Haroun. 1980. “Seismic Design of Liquid Storage Tanks” in *ASCE Convention Proceedings”* should be deleted and moved to the reference pages of Commentary Chapter 15. Seismic design of liquid storage tanks is covered in ASCE 7 Chapter 15, Section 15.7.

> ***TS8 Response: Nonpersuasive.*** *Liquid storage tanks, elevated in structures/buildings, are covered in Chapter 13. Therefore, the reference is appropriate.*

Unsolicited PUC Member Comment: Provide a statement about amplification of seismic accelerations and equipment supported on raised floors. A raised access floor can amplify the free field seismic ground motions by a factor of 4 or more.

> ***TS8 Response: Nonpersuasive but sympathetic.*** *We are of the opinion that design values for raised floors are fine. The amount of amplification that a raised floor will vary considerable depending on whether for floor is braced or contained on all sides, the effective lateral frequency and damping of the access floor and the magnitude and frequency content of the floor motions. We believe the current commentary, in a more general way, already discusses the amplification issue. Providing a more comprehensive set of* a_p *and* R_p *values for raised floors is something we that should be considered in the next NEHRP cycle.*

Proposal 6-114 (Y= 23, YR= 3, N= 7, NV= 12--79%)

TS 6 found the AISI comment editorial and the word "Steel" has been added (it was not in ASCE 7-05 originally but is in errata). TS 5 addressed the BIA comment and proposed a clearer explanation than what was provided in the first sentence of Sec. C14.4.8. The PUC found the BIA proposed solution nonpersuasive (Y=21, N=0, NV=0). TS4 addressed the SEAoCC comment on wall piers and found it persuasive (Y=21, N=0, NV=0). The last sentence in Sec. C14.2.2.14 has been stricken and the TS4 proposed wording was inserted as underlined text. Since this is considered a substantive change, the changed wording will be reballoted (Y=21, N=0, NV=0). The similar CMACN and SEA/NCSEA comments are therefore persuasive. The TMS comment was discussed by the PUC and it was determined that the existing commentary is fair and no additional wording is required as proposed by the first TMS comment. The second TMS comment was found Persuasive / Editorial and the change adding, "for this requirement" has been made.

AISI (YR): On page 3 of 14, line 15 the section title should read "C14.1.4.1 Light-Framed Cold-Formed <u>Steel</u> Construction." (i.e., insert word "Steel")

> ***TS 6 Response: Editorial.*** *"Steel" is not in ASCE 7-05 diectly put has been entered in errata.*

BIA(No): I object to the following: "**C14.4.8 Modifications to Chapter 6 of ACI 530/ASCE 5/T MS 402.** This requirement addresses an apparent inconsistency in the 2005 MSJC Code. Chapter 6 of that document, dealing with masonry veneer, permits corrugated sheet-metal anchors. Chapters 2 and 3 of that document do not permit multi-wythe, noncomposite masonry (functionally identical to veneer) to be bonded by corrugated sheet-metal anchors."

Reason: I believe that the content of C14.4.8 does not address the content of the NEHRP *2003 Provisions* in Section 11.4.3.2, which is: 11.4.3.2 For structures in Seismic Design Category E, corrugated sheet metal anchors shall not be used.

The first sentence of the proposed C14.4.8 is not correct. There is no inconsistency. The remaining sentences of the proposed C14.4.8, which are correct statements, point this out.

The Code simply excludes the use of corrugated sheet metal anchors in SDC E.

Recent testing in the NSF NEES Project on Seismic Performance of Modern Masonry now underway does not support content of Code Section 11.4.3.2. This section was based on opinions of TS5 members who wrote the 2003 *NEHRP Provisions*.

Solution: Replace the current wording of Section C14.4.8 with: "C14.4.8 Sheet metal corrugated anchors are prohibited because of their suspected poor performance at the highest seismic loading."

> ***TS5 Response: Nonpersuasive.*** *TS 5 recognized that there is a need to have a clearer explanation than what is provided in the first sentence but it proposes to solve the problem by adding the following:*

C14.4.8 Modifications to Chapter 6 of ACI 530/ASCE 5/TMS 402. There is an apparent difference in the treatment of corrugated sheet metal anchors in different chapters of the 2005 MSJC Code. ………………..

SEAoCC (YR): YR vote will be changed to affirmative if modifications are made to better address the fundamental difference of C14.2.2.9 and C14.2.2.14 on concrete wall piers.

Comments: C14.2.2.9 and C14.2.2.14 - The design provision for wall pier detailing was originally introduced by SEAOC in 1987 to legacy UBC and was included in 1988 UBC through 1997 UBC. The wall pier detailing was intended for high seismic zones equivalent to current SDC D, E or F. Whereas ACI 318-05 Sec. 21.2.1.4 emphasized special structural wall in regions of high seismic risk, ASCE 7 Table 12.2-1, Design Coefficient and Factors for Seismic-Resisting System, permits intermediate precast structural wall system in SDC D, E or F. Current Section 14.2.2.14 does not limit to just structures assigned to SDC C. The required shear strength in 21.12.3 is based on V_u under either nominal moment strength or two times the code prescribed earthquake force. The required shear strength in 21.4.5.1 is based on the probable shear strength, V_e under the probable moment strength, M_{pr}. In addition, the spacing of required shear reinforcement is 8 inches on center under current 21.4.6 instead of 6 inches on center with seismic hooks at both ends under 21.7.10.2.

Current practice in commercial buildings constructed using precast panels wall system have large window and door openings and/or narrow wall piers. Wall panels varying up to three stories high with openings resembles wall frame which is not currently recognized under any of the defined seismic-force resisting systems other than consideration of structural wall system. Conformance to special structural wall system detailing will ensure minimum life safety performance in resisting earthquake forces in SDC D, E or F.

This commentary should discuss the conceptual difference of wall pier in the two sections cited.

TS4 Response: Persuasive. *In Proposal 6-114, C14.2.2.9 deals with Wall Piers and Wall Segments for special structural walls. C14.2.2.14 deals with Intermediate Precast Structural Walls. Most of the discussion in the existing C14.2.2.14 concerns the introduction of the intermediate wall concept and the insertion of the requirement that connections be designed to maintain 80% of their design strength at the deformation induced by the design displacement, etc. The only statement on wall piers in C14.2.2.14 is the following: "The wall pier requirements of Section 21.13.5 duplicate the same requirements of Section 14.2.2.9 for wall piers in special structural walls." That statement is incorrect. The wall pier requirements of 14.2.2.14 do not duplicate those of 14.2.2.9. The requirements of 14.2.2.14 are more liberal than those of 14.2.2.9.*

The suggested fix is to delete the existing statement on wall piers for intermediate precast walls and replace it with the following: "The wall pier requirements in the modified Section 21.13.5 are less stringent than those for wall piers for special structural walls as specified in the modified Section 21.7.10. Where intermediate precast structural walls are used in SDC D, E and F, wall piers should satisfy the requirements of 21.7.10 rather than 21.13.5."

CMACN (No): Concur with SEAOCC. Negative vote will be changed to affirmative if modifications are made to better address the fundamental difference of C14.2.2.9 and C14.2.2.14 on concrete wall piers. [See SEAoCC comments above.]

TS 4 Response: Persuasive. See response to SEAoCC.

SEAoSC (No): Same as CMACN.
SEAoNC (No): Same as CMACN.
NCSEA (No): Same as CMACN.
SEAoC (No): Same as CMACN.
SEAoSD (No): Same as CMACN

TS 4 Response: Persuasive. See response to SEAoCC.

TMS (YR):
1. Chapter 14, page 10, lines 7-11, item C14.4.6.1 – Add the following "The MSJC is working to resolve this conflict."

TS 4 Response: Nonpersuasive. Presnt commentary is fair.

2. Chapter 14, page 10, lines 25-26, item C14.4.6.2.2 – The Commentary should read "However, there is some controversy concerning the technical validity and necessity <u>for this requirement</u> for masonry walls."

TS 4 Response: Persuasive/ Editorial. Underlined text will be added.

Proposal 8-115R1 (Y= 26, YR= 4, N= 0, NV= 15 --100%)

The PUC voted to accept the TS8 responses to comments (Y=20, N=0, NV=1).

PCA (YR): Section C15.6.1 on earth-retaining structures doesn't say much. A more useful commentary would be most desirable. The most important consideration when it comes to seismic design of chimneys is detailing of reinforcement around breachings/openings. This is not even mentioned in Section C15.6.2 on stacks and chimneys.

TS8 Response: Nonpersuasive. The commentary provided is based on ASCE 7-05 which does not have detailing requirements for the openings. Commentary related to Proposal 8-43R will be added with that new provision in Part 1. Concerning C15.6.1, any suggested commentary language would be appreciated.

SEAoSC (NV): Lack of time to give this commentary a thorough review. We reserve the right to furhter comment on this chapter.

TS8 Response: Noted.

SEAoNC (NV): Lack of time to give this commentary a thorough review. We reserve the right to furhter comment on this chapter. Since the code cycle has been lenghtened, peharps more time could be given for the MO vote.

TS8 Response: Noted.

SEAoKM (YR): I am voting yes with reservations, because this proposal does not reference the recently published ACI 307-08, *Code Requirements for Reinforced Concrete Chimneys,* nor

ASME STS-1-2006, *Steel Stacks.* These two documents are the widely used reference "standards" for industrial chimneys and stacks.

> ***TS8 Response: Nonpersuasive.*** *The commentary provided is based on ASCE 7-05. Because ACI 307-08 and ASME STS-1-2006 are not referenced in ASCE 7, the commentary does not discuss these standards.*

SEAoC (YR): Our YR is based on the following observation.

Comment: While we appreciate the author(s) diligence in writing this commentary chapter. Many of the commentary sections appear to have expanded the provision. For example section C 15.1.3 on Structural Analysis Procedure Solution, took a short two paragraph descriptive provision under ASCE 7 to six pages including seven example cases, which may or may not totally reflect the divergence of the non-building structure in the industry. The committee should consider placing part of the material in Part 3 to supplement the primary commentary.

> ***TS8 Response: Nonpersuasive***. *TS8 disagrees that it expanded the provisions. In the case of C 15.1.3, the short length of the provision contributes to the need to explain it in detail. The explanation does not add new requirements but gives examples of how to apply the provisions.*

ASME (YR): Page 21 of 30, Para. C15.7.3. In preparation for future recommended changes in ASCE 7, add a paragraph (probably at the end) that states: "In particular, vessels and tanks for low temperature services (see C15.7.13) need to meet operating temperature toughness requirements at found in reference documents."

> ***TS8 Response: Nonpersuasive.*** *The intent of the provision is for the seismic design of such tanks and vessels and not material selection. The reference standards (including the standards for ambient temperature tanks) contain material selection provisions for notch toughness. These provisions do not need to be referenced or repeated.*

Page 21 of 30, Para. C15.7.4. On line 38 the ASME disagrees with the word "negligible" and the implication that vessel or tank nozzle loads should be negligible and suggests it be replaced with the word "acceptable."

> ***TS8 Response: Nonpersuasive.*** *We really mean negligible loads. Determining the actual magnitude of the loads due to seismic displacements is very difficult. The connection details need to be such that the load imparted is as low as possible.*

Page 21 of 30, Para. C15.7.4. The subject of lines 43 and 44 do not agree with the paragraph title. A title change (coincident with a title change to ASCE 7) to "Connecting Elements between Vessels and Tanks" might serve to clarify that elements other than piping between free-standing vessels and tanks need to be considered.

> ***TS8 Response: Nonpersuasive.*** *While the primary section deals with connecting piping and piping supports, this was the appropriate section to remind the user about other connected items.*

Page 21 of 30, Para. C15.7.4. On line 45 the ASME disagrees with the statement "Unless connected tanks and vessels are founded on a common rigid foundation,..." and recommends instead the statement read “Unless connected tanks and vessels, including their foundations, are found to have similar vibratory characteristics,...”

> ***TS8 Response: Nonpersuasive.*** *We believe the current wording is more correct.*

Proposal 2-116 (Y= 28, YR= 0, N= 0, NV= 17 --100%)

The three comments provided below were on votes of “Not Voting”. Each stated they wanted more time to comment. Since the TS did not know there would be a fourth ballot at the time responses were developed, TS2 found all comments nonresponsive.

SEAoSC (NV): Lack of time to give this commentary a thorough review. We reserve the right to furhter comment on this chapter.

> ***TS2 Response – Nonresponsive.*** *Since this is the last MO ballot, further comments cannot be provided.*

SEAoNC (NV): Lack of time to give this commentary a thorough review. We reserve the right to furhter comment on this chapter. Since the code cycle has been lenghtened, peharps more time could be given for the MO vote.

> ***TS2 Response – Nonresponsive.*** *Since this is the last MO ballot, further comments cannot be provided.*

SEAoC (NV): Lack of time to give this commentary a thorough review. We reserve the right to furhter comment on this chapter.

> ***TS2 Response – Nonresponsive.*** *Since this is the last MO ballot, further comments cannot be provided.*

PART 3 PROPOSALS

Proposal IT1-3R(Y= 24, YR= 7, N= 5, NV= 10--86%)

IT1 found the HSEAC comment to be partially persuasive but editorial and the PUC agreed (Y=21, N=0, NV=0). IT1 found the PCA, PPCI, and WRI comments to be nonpersuasive but a slight editorial change in Sec. 11.1.4.2 is made, and the PUC agreed (Y=20, N=0, NV=1). The NCSEA comments are the same as those submitted by several SEAs and related organizations. IT1 found the three NCSEA comments to be nonpersuasive; the PUC voted to accept the IT1 response to each of the comments (Y=21, N=0, NV=0). The three NAHB comments were similar to the earlier ones and the PUC voted to accept the IT1 responses on all three (Y=21,N=0, NV=0), (Y=20, N=0, NV=0), (Y=20, N=0, NV=0).

HSEAC (No): In principle, it is undesirable to utilize a 90% draft of a report (ATC-63) as the basis for (alternate) seismic design criteria.

> ***IT1 Response: Partially persuasive--Editorial****. It is probable that the 100% version will be published before going to print with the 2008 Provisions. This section is only going*

into Part 3 anyway. We will add, "90% Draft <u>or latest published</u> version." IT1 Vote: Yes: 5, No: 0, Abstain: 1, No Vote: 0.

PCA (No): This proposal is not the same as the Proposal IT1-3R, on which you have enclosed PUC comments. This is confusing. However, it is entirely possible that I am mixed up somehow. In any case, this is not the basis of my No vote.

> ***IT1 Response: Nonpersuasive.*** *Editorial changes were made at the last PUC meeting when it was decided to place this proposal in Part 3. The responses to comments indicate what changes were made. IT1 Vote: Yes: 6, No: 0, No Vote: 0.*

I refuse to accept that the procedures in ASCE/ACI 41-06 and ATC 63 are the only acceptable procedures for establishing equivalency. That has not been demonstrated or proven to me in a way that I can understand. In the case of precast concrete special moment frames and special shear walls, we have required in ACI 318 that acceptable performance be demonstrated through validation testing. ACI Publications 374.1 and ITG 5.1 spell out the details of this approach. Because of the physical testing element involved, this approach seems to me to be more desirable than what is in ATC 63. I intend to continue to oppose this proposal to the best of my ability.

PPCI (No): We do not agree that section 11.1.4.2.3 should limit the way of demonstrating how the required seismic performance will be met to one of the two methods stated. It is suggested that a third method be included in the provision which may be equivalent to the two methods currently stated (ASCE/SEI 41-06 and ATC 63). A third method may demonstrate acceptable performance through validation testing. The precast concrete industry has spent many resources through the last twenty years successfully developing acceptance criteria for special moment frames (ACI 374.1) and special shear walls (ACI ITG 5.1) based on validation testing. Even though it can be extremely expensive and time consuming, validation testing is a proven way of demonstrating how the required seismic performance can be met using a particular seismic force resisting system.

We would change our negative vote to an affirmative vote if this proposal were to include a third possible method that included validation testing which could establish equivalency to the other two methods.

WRI (No): Note on Proposals Numbered: 2-4R2, 2-4C and IT1-3R – I agree with Dr. S.K. Ghosh on his statements of reason. I back him 100% on the above noted proposal numbers. You may use his statements on my ballot.

> ***IT1 Response: Persuasive editorial.*** *This proposal would not prohibit "validation testing," particularly if it is defined in an ACI Standard. SEAONC (below) also asks for the guideline developed for San Francisco high rises to be referenced; the LA guideline would not be far behind. These two guidelines have been transformed from generic criteria to local criteria by local experts and building officials. Locally approved or material specific guidelines are not appropriate here. Section 11.1.4.2 makes it clear that other demonstrations of equivalence are potentially acceptable. This section is for the local building official who has no criteria to lean on. However, to address these concerns, the words "or accept" will be added after "to develop" in line 13. IT1 Vote: Yes: 5, No: 1, No Vote: 0.*

SEAoCC (YR):

1. There are many aspects of this issue that have not been addressed to this date. The commentary developed in BSSC proposal is nonspecific as to actual use. It contains comments that appear to lead to definitions but they are hollow. Comments such as ". . . have been developed over a period of more than 100 years of observation of the actual performance of buildings . . ." With respect to Light-Frame buildings, those constructed after each significant earthquake such as Northridge EQ in 1994, have different response characteristics from buildings constructed before the Northridge EQ. How do we have more than 100 years of experience with these? They have yet to meet their first earthquake. Not to mention that the lessons of the Northridge EQ were not interpreted correctly in the first place. Also, entire testing programs have been based on the misinterpretations of the lessons learnt. Also ". . . knowledge of seismic hazards and ground motions for the region in which the structure is to be constructed . . ." cannot be known when evaluation testing is done and evaluation reports are prepared.

2. Performance of different material or component does vary. Evaluation criteria should be reviewed with care for "equivalency", if the engineering profession can ever agree on whatever "equivalency" mean. Recent attempt at ICC-ES to develop such a uniform criteria led to much debate and discontent among professionals and industrial representatives. One big hurdle to overcome is that basic test protocol cannot be unified among industry.

3. An additional authoritative document on performance based design is the recently published, "SEAONC Recommended Administrative Bulletin on the Seismic Design & Review of Tall Buildings Using Non-Prescriptive Procedures" April 2007." This document is in use in many jurisdictions of California. Please reference this document in 11.1.4.2.3.

CMACN (YR): Concur with SEAoCC.

1. There are many aspects of this issue that have not been addressed to this date. The commentary developed in BSSC proposal is nonspecific as to actual use. It contains comments that appear to lead to definitions but they are hollow. Comments such as ". . . have been developed over a period of more than 100 years of observation of the actual performance of buildings . . ." With respect to Light-Frame buildings, those constructed after each significant earthquake such as Northridge EQ in 1994, have different response characteristics from buildings constructed before the Northridge EQ. How do we have more than 100 years of experience with these? They have yet to meet their first earthquake. Not to mention that the lessons of the Northridge EQ were not interpreted correctly in the first place. Also, entire testing programs have been based on the misinterpretations of the lessons learnt. Also ". . . knowledge of seismic hazards and ground motions for the region in which the structure is to be constructed . . ." cannot be known when evaluation testing is done and evaluation reports are prepared.

2. Performance of different material or component does vary. Evaluation criteria should be reviewed with care for "equivalency", if the engineering profession can ever agree on whatever "equivalency" mean. Recent attempt at ICC-ES to develop such a uniform criteria led to much debate and discontent among professionals and industrial representatives. One big hurdle to overcome is that basic test protocol cannot be unified among industry.

3. An additional authoritative document on performance based design is the recently published, "SEAONC Recommended Administrative Bulletin on the Seismic Design &

Review of Tall Buildings Using Non-Prescriptive Procedures" April 2007." This document is in use in many jurisdictions of California. Please reference this document in 11.1.4.2.3.

SEAoSC (YR): Same as CMACN.
SEAoNC (YR): Same as CMACN.
SEAoC (YR): Same as CMACN.

SEAoSD (YR):

1. There are many aspects of this issue that have not been addressed to this date. The commentary developed in BSSC proposal is nonspecific as to actual use. It contains comments that appear to lead to definitions but are hollow and will be worthless when used to develop code language. Comments such as ". . . have been developed over a period of more than 100 years of observation of the actual performance of buildings . . ." is not helpful in the development of regulations. With respect to Light-Frame buildings, those constructed after an earthquake such as San Fernando (1971) are different from those constructed after Northridge (1994) and will therefore respond differently in subsequent earthquakes. Therefore, even in this short time frame, the dataset is dynamic and does not enable the community to maintain comparative observations over a period of 100 (or even 20) years.

2. Performance of different systems and different materials or components does vary. Evaluation criteria related to "equivalency" should be carefully considered and empirically comparative. Recent attempts at ICC-ES (e.g. AC 322) to develop uniform criteria led to much debate and discontent among professionals and industrial representatives. Without irrefutable data and methods, assignment of equivalency tends to represent opinion.

3. An authoritative document on performance based design has been recently published, "SEAONC Recommended Administrative Bulletin on the Seismic Design & Review of Tall Buildings Using Non-Prescriptive Procedures" April 2007." Please refer to this document in 11.1.4.2.3

NCSEA (YR):

1. There are many aspects of this issue that have not been addressed to this date. The commentary developed in BSSC proposal is nonspecific as to actual use. It contains comments that appear to lead to definitions but are hollow and will be worthless when used to develop code language. Comments such as ". . . have been developed over a period of more than 100 years of observation of the actual performance of buildings . . ." is not helpful in the development of regulations. With respect to Light-Frame buildings, those constructed after an earthquake such as San Fernando (1971) are different from those constructed after Northridge (1994) and will therefore respond differently in subsequent earthquakes. Therefore, even in this short time frame, the dataset is dynamic and does not enable the community to maintain comparative observations over a period of 100 (or even 20) years.

   IT1 Response: Nonpersuasive. *The "100 years of observation" is a phrase in the commentary and is factual (give or take a few years) when applied generally to US Codes. In the context of the commentary discussion, the phrase may or may not "be helpful in the development of regulations," but this is not the only criterion used to judge commentary sentences. IT1 Vote: Yes: 5, No: 0, No Vote: 1.*

2. Performance of different systems and different materials or components does vary. Evaluation criteria related to "equivalency" should be carefully considered and empirically

comparative. Recent attempts at ICC-ES (e.g. AC 322) to develop uniform criteria led to much debate and discontent among professionals and industrial representatives. Without irrefutable data and methods, assignment of equivalency tends to represent opinion.

> ***IT1 Response: Nonpersuasive****. This section is not directly addressing or refining methods of demonstrating exact equivalence, but analytical methods that are intended to show equivalent system performance. Traditional and current methods of demonstrating component equivalence are allowed under Section 11.1.4.2. IT1 Vote: Yes: 5, No: 0, No Vote: 1.*

3. An authoritative document on performance based design has been recently published, "SEAONC Recommended Administrative Bulletin on the Seismic Design & Review of Tall Buildings Using Non-Prescriptive Procedures" April 2007." Please refer to this document in 11.1.4.2.3

> ***IT1 Response: Nonpersuasive.*** *The SEAONC document is applicable for the purpose intended in the City of San Francisco. Another document is used in LA. PCA and PPCI are suggesting methods appropriate for certain concrete components or systems. This section is for the local building official "In the absence of such criteria…" [section 11.1.4.2] IT1 Vote: Yes: 5, No: 0, No vote: 1.*

NAHB (No): This proposal is not ready for prime time. The primary motivation for this proposal appears to be to encourage innovation by providing local jurisdictions with an evaluation process for alternative materials and methods. However, an unintended practical implication of this proposal may be an attempt to change the ICC-ES process. The current Section 11.1.4 as written provides a basis for evaluation services to issue approval reports for alternative materials and methods. The current language is general in nature, providing evaluation services with the flexibility to develop appropriate procedures based on a specific seismic hazard (i.e., seismic design category), a specific material or method (e.g., a new seismic force resisting system vs. a new fastener vs. a new anchorage system), and other criteria. The proposed language lays out a specific methodology that every evaluation will have to follow. This can trigger a significant change how ICC-ES process and other evaluation services work. While the proposed language in Section 11.1.4.2 attempts to provide an exemption for reports from evaluation services, in practice the proposed procedure will be viewed as the new requirement.

The proposed procedure appears more appropriate for the evaluation of new seismic force-resisting systems by local jurisdictions in situations where evaluation reports are not yet available, or where the system may be unique in nature (project-specific) and a national evaluation may not be needed or even practical. This can be addressed by better scoping and a reorganization of the proposed language.

> ***IT1 Response: Nonpersuasive.*** *See Section 11.1.4.2: 11.1.4.2 Approval of proposals under Sec 11.1.4. Nothing in this section shall limit the ability of the authority having jurisdiction to develop general requirements for proposals under Sec. 11.1.4 or specific requirements for particular components or systems, such as acceptance of reports from evaluation services or other demonstration of equivalence as specified in Sec 11.1.4.1. See also response to PCA/PPCI/WRI. IT1 Vote: Yes: 5, No: 0, No vote: 1.*

It is further unclear whether the peer review specified in Section 11.1.4.2.1 is required in addition to the peer review provisions of ATC-63.

__IT1 Response: Nonpersuasive.__ No additional peer review is required. Clarification is not needed. IT1 Vote: Yes: 5, No: 0, No vote: 1.

It is also will be unclear to the users of this section how the limit states and procedures used in ASCE 41 are applicable to the provisions of ASCE 7 for the purpose of establishing equivalency.

__IT1 Response: Nonpersuasive.__ The alternate materials, design, and methods concept is broader than equivalency. In many cases, systems are new and not equivalent to anything used before. In those cases, we are trying to demonstrate acceptability. However, ASCE 7 and ASCE 41 are tied together through the IBC Performance Code, which lists ASCE 41 performance goals for each Occupancy Category. IT1 Vote: Yes: 5, No: 0, No vote: 1.

Proposal 2-3R4(Y= 27, YR= 1, N= 0, NV= 17--100%)

TS2 found the SEAoC comment to be nonpersuasive and the PUC agreed (Y=20, N=0, NV=0). The remaining comment was found to be nonresponsive and no action was required.

SEAoNC (NV): Since the code cycle has been lenghtened, pehaps more time could be given for the MO vote.

__TS2 Response – Nonresponsive.__ Since this is the last MO ballot, further comments cannot be provided.

SEAoC (YR):
Comment:
Page 2, line 37: Restriction to 40 feet height limit would discourage the use of this procedure.

Page 7, line 41: Restriction to 40 feet height limit would discourage the use of this procedure.

__TS2 Response: Nonpersuasive.__ This is the height limit that research has indicated that reliable member demands and deformations can be expected.

Proposal 2-5R5 (Y= 26, YR= 8, N= 0, NV= 11--100%)

TS2 the ICSSC comments to be nonresponsive. The SEAoCC comment was discussed in great detail and the PUC concluded that an editorial change to the parenthetical phrase adequately addressed the comment. The CMACN comments were similar and the comment about schedule was noted. The SEAs and NCSEA comments are similar to those of CMACN. The PUC accepted the TS2 responses (Y=21, N=0, NV=0).

ICSSC (YR):
Page 2, Line 49: See Proposal 2-111.

TS2 Response – Nonresponsive. *This proposal is independent of the changes associated with SDPRG-1R4. Changes to Proposal 2-5R5 could be made to update the MCE definitions to be consistent with SDPRG-1R4 (assuming the proposal is successful). However, this effort would be part of a future change to the Provisions, such as moving this new Chapter 16 into Part 1, or could be done as part of the process to bring this new material into ASCE 7-10 or some later version.*

Page 2, Lines 49-54: Who approves the selection of simulated or modified ground motions? The approving authority should be stated in the mandatory part.

TS2 Response –Nonresponsive. *Section 16.5 requires that an independent team of registered design professionals in the appropriate disciplines review the work, including the site-specific spectra and ground motion time histories. The Authority Having Jurisdiction has the final say on the approval process and would likely rely on the expert opinion of this independent review team.*

SEAoCC (YR): Sections 16.2.2 and 16.2.3 should also explicitly permit spectral matching of the ground motion acceleration histories to the design response spectra in addition to scaling method given.

The *Provisions* imply that scaling of ground motion acceleration histories to the design response spectra using a scale factor times the time history accelerations for a specific period range is the only approved method of matching the target response spectra. Notable expert seismologists in this field regularly use spectral matching algorithms to match the time histories to the target response spectra across the full period range and should be permitted to do this. We understand there is a PEER ground motion scaling study and a complete re-write of this section to align better with this part and current practice is recommended.

TS2 Response – Persuasive/Editorial. *The parenthetical "(where an identical scale factor is applied to components of a pair")" will be modified to read "(for direct scaling, ~~where~~ an identical scale factor is applied to components of a pair") to help avoid the confusion. The Provisions update schedule does not allow for the completion of the PEER study. Future updates will be able to consider the results from PEER.*

CMACN (YR):

1. Concur with SEAOCC. Although this is step in the right direction, we should wait for PEER study results.

    TS2 Response – Nonpersuasive. *The Provisions update schedule does not allow for the completion of the PEER study. Future updates will be able to consider the results from PEER.*

2. Sections 16.2.2 and 16.2.3 should also explicitly permit spectral matching of the ground motion acceleration histories to the design response spectra in addition to scaling method given.

    TS2 Response – Persuasive/Editorial. *The parenthetical "(where an identical scale factor is applied to components of a pair")" will be modified to read "(for direct scaling, ~~where~~ an identical scale factor is applied to components of a pair") to help avoid the confusion.*

SEAoSC (YR): Same as CMACN.
SEAoNC (YR): Same as CMACN.
SEAoC (YR): Same as CMACN.
SEAoSD (YR): Same as CMACN.

TS2 Response – Nonpersuasive and Partially Persuasive. *See response to CMACN.*

NCSEA (YR):
1. PEER study results may influence the direction of this section and should be included.

2. Sections 16.2.2 and 16.2.3 should also explicitly permit spectral matching of the ground motion acceleration histories to the design response spectra in addition to scaling method given.

TS2 Response – Nonpersuasive and Partially Persuasive. *See response to CMACN.*

Proposal 3-6 (Y= 26, YR= 8, N=0, NV= 12--100%)

TS3 concluded that the DSA(CA) comments were all persuasive-editorial and the required changes have been included in a modified proposal. TS3 discussed its response to the remaining member organization comments, all of which were similar. The PUC accepted the TS3 responses to find all comments persuasive editorial (Y=20, N=0, NV=0).

DSA(CA) (YR):
Page 1, line 26 – replace "1997" with "2008"
Page 1, line 28 – after "Liquefaction: " insert "California Geological Survey (2008), "
Page 1, line 34 – after "Surface fault rupture: " insert "California Geological Survey (2002), "
Page 3, line 28 – after "Jibson, 1993" insert "; Jibson and Jibson, 2003"
Reason – more recent reference, which includes free analytical software
Page 17, line 25 – replace "(last 11,000 years)" with "(about the last 11,000 years)"
Page 19, line 4 – replace "safety" with "safely"
Page 26, line 25 – replace "California Geological Survey, 1997. Special Publication 117" with "California Geological Survey, 2008. Special Publication 117A"
Page 27, line 40 – insert reference "Jibson, R.W., and Jibson, M.W., 2003. Java Programs for Using Newmark's Method and Simplified Decoupled Analysis to Model Slope Performance during Earthquakes: U.S. Geological Survey Open-File Report 03-005, version 1.0."

TS3 Response: Persuasive-editorial. *The text of the proposal has been edited for each of the above remarks.*

Page 18, line 1 – replace "movement" with "displacement"
Reason – careful definition of faulting
Page 18, line 20 – delete "primary and secondary"
Reason – what is the definition of "primary faulting" and "secondary faulting"?
Page 19, line 2 – delete "a minimum of "
Reason – CCR 3603(a) does not state that 50 ft is a "minimum" setback
Page 19, line 2 – delete "a well-defined zone containing"
Reason – CCR 3603 is not limited to "well-defined zones"

Page 19, line 3 – delete "as a minimum"

Reason - CCR 3603(a) does not state that 50 ft is a "minimum" setback

Page 19, line 20 – after "assure that " insert "(a) the structure is not sited across the trace of an active fault, and (b) "

Reason – Most pressure ridges and sags are features of surface-rupturing faults. The recommended language should clarify that these geomorphic features should be investigated for fault rupture hazard, and not assumed to be zones of ductile deformation.

Page 19, line 22 – replace "unacceptable faulting hazard" with "surface faulting hazard"

Reason – how is "unacceptable faulting hazard" defined?

TS3 Response: Persuasive-editorial. *The text of the proposal has been edited to address each of the above language remarks.*

SEAoCC (YR): YR will be changed to affirmative if modifications are made to address the following comments:

1. Page 1, lines 31 through 33: a basic reference for seismic settlement should be provided rather than down-playing the risk of shaking-induced settlement.

2. Page 2, line 15: reference should be made to design-level accelerations rather than geomean PGA's – with the proposed change in computation of design accelerations, this instruction would be inconsistent with evaluation of ground motions.

3. Page 2, line 18. The statement seems to imply that a Factor of Safety of 1.0 is acceptable. Delete sentence.

4. Page 2, lines 22 through 24 – reference should be made that the reduction represents a modification from peak ground acceleration to seismic coefficient.

5. Page 9, lines 34 through 39. If provisions of SDPRG-1R4 are adopted, then a definition of ground motion for use in 3-5R2 should be made. This definition could be MCE, or for use in geotechnical analyses, the ground motion with a 2% probability of exceedence in 50 years. This modification is important because the fragility curves upon which the RTE are developed do not have meaning with geotechnical applications.

6. Page 11, lines 11-22. It is suggested that a special mention be made as to whether the correction for liners in the SPT barrel is used.

7. Page 11, lines 23-25. It is suggested that a mention be made that in the probabilistic analyses, a magnitude correction can be internally applied rather than at this stage.

8. Page 12, lines 12-14. Add "or the PGA used can be adjusted within the probabilistic analysis to correspond to a Magnitude of 7.5.

9. Page 14, lines 12 through 14. It should be mentioned that differential settlement should also be evaluated for these non-critical structures.

10. Page 14, lines 24 through 26. It is considered overly conservative to use a K value of 1.0 for liquefied soils as is implied in the discussion. It should be mentioned that the K value would be expected to be somewhere between the at-rest K value and 1.0

11. Page 14, lines 35 through 46. A mention should be made that when evaluating differential settlement, differential settlement may need to be assumed to be ½ of the total settlement in the absence of sparse data over the footprint of the building.

12. Page 16, lines 16 through 18 should discuss inclusion of downdrag forces on piles or methods whereby the downdrag forces can be reduced, such as in use of concrete forms (sonotubes) left in place for drilled shafts.

13. Page 17, line 2. For the use of ground modification, it should be mentioned that the methodology can be used where excessive strength loss beneath foundations needs to be mitigated.

14. Page 20, lines 26 through 35 and Page 22, lines 14 through 28. The statements made do not incorporate recent research such as Al Atik, L. and N. Sitar, "Dynamic Centrifuge Study of Seismically Induced Lateral Earth Pressures," Proceedings, 4th International Conference on Earthquake Geotechnical Engineering, Thessaloniki, Greece, June 25-28, 2007. These and other studies show that estimating seismic lateral earth pressures utilizing the full peak ground acceleration provides a significant over-estimate of the forces. It is suggested that there not be a recommendation that the peak ground acceleration be reduced to obtaining the seismic coefficient, but instead a range of approaches be recognized.

> ***TS3 Response to Comments 1, 3-4, and 6-14: Persuasive.*** *The text of the proposal has been edited to address each of these comments.*
>
> ***TS3 Response to Comment 2: Partially persuasive****. The text has been modified to make the sentence independent of earthquake definition. To the extent possible, this Proposal 3-6 is intended to serve as a resource document on methodologies for use in evaluations of geohazards and seismic earth pressures. As such the methodologies described should be applicable whether the seismic event is the Maximum Credible Earthquake (MCE), the Design Earthquake (2/3rds of the MCE), or some other event. The selection of the appropriate ground motion is covered in the Provisions.*
>
> ***TS3 Response to Comment 5:*** *As discussed in the previous response, Proposal 3-6 is intended as a resource document for assessing geologic hazards including liquefaction. To the extent possible, it has been written to be independent of the level of ground shaking that will be used by the designer. The selection of the ground motion for design is covered in the Provisions.*
>
> *As discussed in the response to a similar comment in Proposal 3-5R2, the definition of ground motion will be stated on the new PGA maps, which have been completed but not fully labeled. The new PGA maps are for MCE ground motions and not for RTE ground motion. Furthermore, the new PGA maps are based on geomean ground motions as directly incorporated in the ground motion attenuation relations used in making the PGA maps, and not the adjusted ground motion for maximum direction of ground motion. The use of geomean PGA values is consistent with the values used in geotechnical practice and that have been used in developing the empirically-based liquefaction potential evaluation correlations that are widely used in practice.*

CMACN (YR): YR will be changed to affirmative if modifications are made to address the following comments:

1. Concur with SEAOCC. Page 1, lines 31 through 33: a basic reference for seismic settlement should be provided rather than down-playing the risk of shaking-induced settlement.

2. Page 2, line 15: reference should be made to design-level accelerations rather than geomean PGA's – with the proposed change in computation of design accelerations, this instruction would be inconsistent with evaluation of ground motions.

3. Page 2, line 18. The statement seems to imply that a Factor of Safety of 1.0 is acceptable. Delete sentence.

4. Page 2, lines 22 through 24 – reference should be made that the reduction represents a modification from peak ground acceleration to seismic coefficient.

5. Page 9, lines 34 through 39. If provisions of SDPRG-1R4 are adopted, then a definition of ground motion for use in 3-5R2 should be made. This definition could be MCE, or for use in geotechnical analyses, the ground motion with a 2% probability of exceedence in 50 years. This modification is important because the fragility curves upon which the RTE are developed do not have meaning with geotechnical applications.

6. Page 11, lines 11-22. It is suggested that a special mention be made as to whether the correction for liners in the SPT barrel is used.

7. Page 11, lines 23-25. It is suggested that a mention be made that in the probabilistic analyses, a magnitude correction can be internally applied rather than at this stage.

8. Page 12, lines 12-14. Add "or the PGA used can be adjusted within the probabilistic analysis to correspond to a Magnitude of 7.5.

9. Page 14, lines 12 through 14. It should be mentioned that differential settlement should also be evaluated for these non-critical structures.

10. Page 14, lines 24 through 26. It is considered overly conservative to use a K value of 1.0 for liquefied soils as is implied in the discussion. It should be mentioned that the K value would be expected to be somewhere between the at-rest K value and 1.0

11. Page 14, lines 35 through 46. A mention should be made that when evaluating differential settlement, differential settlement may need to be assumed to be ½ of the total settlement in the absence of sparse data over the footprint of the building.

12. Page 16, lines 16 through 18 should discuss inclusion of downdrag forces on piles or methods whereby the downdrag forces can be reduced, such as in use of concrete forms (sonotubes) left in place for drilled shafts.

13. Page 17, line 2. For the use of ground modification, it should be mentioned that the methodology can be used where excessive strength loss beneath foundations needs to be mitigated.

14. Page 20, lines 26 through 35 and Page 22, lines 14 through 28. The statements made do not incorporate recent research such as Al Atik, L. and N. Sitar, "Dynamic Centrifuge Study of Seismically Induced Lateral Earth Pressures," Proceedings, 4th International

Conference on Earthquake Geotechnical Engineering, Thessaloniki, Greece, June 25-28, 2007. These and other studies show that estimating seismic lateral earth pressures utilizing the full peak ground acceleration provides a significant over-estimate of the forces. It is suggested that there not be a recommendation that the peak ground acceleration be reduced to obtaining the seismic coefficient, but instead a range of approaches be recognized.

TS3 Response: *Please see responses to Comments from SEAoCC*

SEAoSC (YR): Same as CMACN.

TS3 Response: *Please see responses to Comments from SEAoCC*

SEAoNC (YR): Same as CMACN.

TS3 Response*:* *Please see responses to Comments from SEAoCC*

SEAoC (YR): Same as CMACN.

TS3 Response: *Please see responses to Comments from SEAoCC*

SEAoSD (YR): YR will be changed to Y if the following comments are addressed:

1. Page 1, lines 31 through 33: a basic reference for seismic settlement should be provided.
2. Page 2, line 15: reference should be made to design-level accelerations rather than geomean PGA's – with the proposed change in computation of design accelerations, this instruction would be inconsistent with evaluation of ground motions.
3. Page 2, line 18. The statement seems to imply that a Factor of Safety of 1.0 is acceptable. Delete sentence.
4. Page 2, lines 22 through 24 – reference should be made that the reduction represents a modification from peak ground acceleration to seismic coefficient.
5. Page 11, lines 11-22. Should the correction for liners in the SPT barrel is used?
6. Page 11, lines 23-25. An internally applied magnitude correction can be made in the probabilistic analyses.
7. Page 12, lines 12-14. Add "or the PGA used can be adjusted within the probabilistic analysis to correspond to a Magnitude of 7.5."
8. Page 14, lines 12 through 14. Differential settlement should also be evaluated for non-critical structures.
9. Page 14, lines 24 through 26. It is considered overly conservative to use a K value of 1.0 for liquefied soils as is implied in the discussion. The K value would be expected to range somewhere between the at-rest K value and 1.0
10. Page 14, lines 35 through 46. When evaluating settlement, differential settlement may need to be assumed to be ½ of the total settlement in the absence of adequate data over the footprint of the building.

11. Page 16, lines 16 through 18 should discuss inclusion of downdrag forces on piles or methods whereby downdrag forces can be reduced, such as in use of concrete forms (sonotubes) left in place for drilled shafts.
12. Page 17, line 2. Ground modification can be used where excessive strength loss beneath foundations requires mitigation.
13. Page 20, lines 26 through 35 and Page 22, lines 14 through 28. The statements made do not incorporate recent research such as Al Atik, L. and N. Sitar, "Dynamic Centrifuge Study of Seismically Induced Lateral Earth Pressures," Proceedings, 4th International Conference on Earthquake Geotechnical Engineering, Thessaloniki, Greece, June 25-28, 2007. These studies show that estimating seismic lateral earth pressures utilizing the full peak ground acceleration provides a significant over-estimate of the forces. Recommend that the peak ground acceleration be reduced to obtain the seismic coefficient, with recognition that a range of methods are acceptable.

    TS3 Response: *Please see responses to Comments from SEAoCC*

NCSEA (YR): YR will be changed to Y if the following comments are addressed:

1. Page 1, lines 31 through 33: a basic reference for seismic settlement should be provided.
2. Page 2, line 15: reference should be made to design-level accelerations rather than geomean PGA's – with the proposed change in computation of design accelerations, this instruction would be inconsistent with evaluation of ground motions.
3. Page 2, line 18. The statement seems to imply that a Factor of Safety of 1.0 is acceptable. Delete sentence.
4. Page 2, lines 22 through 24 – reference should be made that the reduction represents a modification from peak ground acceleration to seismic coefficient.
5. Page 11, lines 11-22. Should the correction for liners in the SPT barrel is used?
6. Page 11, lines 23-25. An internally applied magnitude correction can be made in the probabilistic analyses.
7. Page 12, lines 12-14. Add "or the PGA used can be adjusted within the probabilistic analysis to correspond to a Magnitude of 7.5."
8. Page 14, lines 12 through 14. Differential settlement should also be evaluated for non-critical structures.
9. Page 14, lines 24 through 26. It is considered overly conservative to use a K value of 1.0 for liquefied soils as is implied in the discussion. The K value would be expected to range somewhere between the at-rest K value and 1.0
10. Page 14, lines 35 through 46. When evaluating settlement, differential settlement may need to be assumed to be ½ of the total settlement in the absence of adequate data over the footprint of the building.
11. Page 16, lines 16 through 18 should discuss inclusion of downdrag forces on piles or methods whereby downdrag forces can be reduced, such as in use of concrete forms (sonotubes) left in place for drilled shafts.
12. Page 17, line 2. Ground modification can be used where excessive strength loss beneath foundations requires mitigation.

13. Page 20, lines 26 through 35 and Page 22, lines 14 through 28. The statements made do not incorporate recent research such as Al Atik, L. and N. Sitar, "Dynamic Centrifuge Study of Seismically Induced Lateral Earth Pressures," Proceedings, 4th International Conference on Earthquake Geotechnical Engineering, Thessaloniki, Greece, June 25-28, 2007. These studies show that estimating seismic lateral earth pressures utilizing the full peak ground acceleration provides a significant over-estimate of the forces. Recommend that the peak ground acceleration be reduced to obtain the seismic coefficient, with recognition that a range of methods are acceptable.

    TS3 Response: *Please see responses to Comments from SEAoCC*

Proposal 7-10R1 (Y= 23, YR= 7, N= 0, NV= 16 --100%)

TS7 found AISI coment to be editorial and the change has been made. Each item of CMACN was addressed and where TS7 found them editorial, the changes have been made. The CSSC comment was found to be partially persuasive-editorial and a section, "Sheathing Connectors to wood studs" has been added under "Wood Framing Forces and Connectivity." The comments from the SEAs and NCSEA are the same those from CMACN. The PUC voted to accept the TS7 responses (Y=20, N=0, NV=1).

AISI (YES): On page 7 of 11, line 36 change "punch out" to "punchout" and on line 38 change "punch outs" to "punchouts".

TS7 Response: Persuasive, Editorial. *Revisions made.*

CMACN (YR): YR will be changed to affirmative if modifications are made to address the following comments.

1. Concur with SEAOCC. Pg 1, line 13 Redundancy and exterior finish materials are also indirectly relied upon.

    TS7 Response: Persuasive, Editorial. *That is the point of the paragraph. Mention of redundancy is noted in line 23.*

2. Pg.1, line 25 Lowest stories should not be described as "typically the softest and weakest" because that implies that a code defined soft or weak story is typically present. More correctly, the unaccounted strength and stiffness contributed by the finish materials tend to concentrate damage
at the lowest floor.

    TS7 Response: Persuasive, Editorial. *Clarified as being analysis and testing of structures with finishes included.*

3. pg 2, line 42 the author is not clear if "additional gypsum" means adding a second layer. The main point regardless should be the effect of having a considerable amount of interior full height partitions. In addition to the research cited, I have pictures of houses at the bottom of a landslide that moved 40' that have no shear walls and are essentially supported by stucco and non-structural partitions.

TS7 Response: Persuasive, Editorial. *The word "additional" has been struck to avoid confusion.*

4. pg 6, line 3, a big point should be directly stated: That the narrow wall segment should get a tighter nailing pattern compared to the adjacent longer wall. This is what the intent of the 2w/l provision is but is contrary to the practice and needs to be reiterated.

   TS7 Response: Nonpersuasive. *This discussion is in no way germane to the topic of gypsum wallboard with floating corners.*

5. Pg. 6, line 39, the buckling of the tension post never governs the narrow shear walls and that the provisions are way too restrictive. It is not reasonable to have to design for all three possible mechanisms.

   TS7 Response: Nonpersuasive. *Line 39 discusses tension, and three applicable types of tension limit are discussed. Discussion of compression follows. Current design practice does not allow compression buckling of boundary members to be ignored. Commenter is welcome to submit technical substantiation for eliminating this check for future consideration.*

SEAoSC (YR): Same as CMACN.
SEAoNC (YR): Same as CMACN.
SEAoC (YR): Same as CMACN.
SEAoSD (YR): Same as CMACN.

CASSC (YR): Page 8, lines 37 and 38 mentions that the "behavior of screws in CFS is different than the behavior of sheathing nails in wood framing." For consistency and completeness, a new subsection on page 6 under the header "Wood Framing Forces and Connectivity" should be added describing the principal failure modes of nails that attach sheathing to wood studs. The principal desired mode of response is nail bending and the formation of flexural hinges in the nails just below the face of the studs prior to the onset of other less desirable modes such as pullout or fracture. This sequence of response modes results in relatively less pinched hysteretic curves and more residual stiffness compared to CFS systems. In addition, a section should be added to page 8 that correspondingly describes the expected performance of screws in CFS where the desired modes of response are rocking of the screws as the steel next to the screw holes in the studs yields, as well as tension in the screws, without significant flexural hinges in the screws, while precluding undesirable modes such as the screws ratcheting or pulling out of the holes. This desired sequence of response modes results in considerably more pinched hysteretic curves with lower residual stiffness for CFS systems compared to wood stud systems.

TS7 Response: Partially Persuasive and Editorial. *A parallel section has been added to page 7 discussion sheathing fasteners to wood studs and applicable mode of behavior.*

NCSEA (YR): YR will be changed to Y if the following comments are addressed:

1. Pg 1, line 13 Redundancy and exterior finish materials are also participatory.

2. Pg.1, line 25 Lowest stories should not be described as "typically the softest and weakest" because that implies that a code defined soft or weak story is typically present. More correctly one might write, “the unaccounted strength and stiffness contributed by finish materials tend to concentrate damage at the lowest floor.”

3. pg 2, line 42 the author is not clear if "additional gypsum" means adding a second layer (or more layers than typical). The point should be made to clarify the effect of having a considerable amount of interior full height partitions.

4. pg 6, line 3, add: “narrow wall segments should have closer spaced nailing pattern compared to longer adjacent walls.”

5. Pg. 6, line 39, tension post buckling rarely governs narrow shear wall behavior, indicating that the provisions are too restrictive. It is not reasonable to design for all three possible mechanisms. Eliminate the tension post buckling check.

   TS7 Response*: See response to same questions submitted by NCMAC.*