PUC

COMMENTS AND RESPONSES ON JULY-AUGUST, 2008 PUC BALLOT PROPOSALS

(Reviewed and balloted at PUC meetings
September 9 and December 9, 2008)

PART 1

Proposal 2-4 Edit (Y=18, YR=2, N=1, NV=1 --95%)

TS 2 addressed each comment by preparing a revised proposal 2-4R2. This included Ghosh's comment and therefore was considered editorial. No vote was needed. The Hamburger comment was discussed and a vote was taken to find Hamburger persuasive. This passed (23,0,0,). The comment is included in 2-4R2 as well. The Klingner comment was withdrawn because it primarily addressed the following related proposal. Proposal 2-4R2 passed and will be voted on by the member organizations.

Ghosh (No): Proposal appears to be premature. What is "risk-adjusted MCE spectral acceleration"? I thought this was something still being discussed? Also, I do not know that it is the best of ideas to refer to ASCE 41. NEHRP Provisions used to be a complete document unto itself. Then it was decided to refer to ASCE 7 for most things. Now the document will be dependent on ASCE 41 as well?

> ***TS-2 Response: Editorially Persuasive.***
>
> *The terminology "risk-adjusted MCE spectral acceleration" will be changed to "MCE spectral acceleration". This terminology will be retained unless SDPRG-1R4 passes. It is understood that "RTE spectral acceleration" would be used instead in the event that the SDPRG proposal passes.*
>
> *We can choose to maintain prescriptive provisions on nonlinear static analysis, but to do so would be duplicitous of the work done by ASCE-41.*

Recent attempts to revise the NSP Appendix have concluded with the passage of Proposal 2-3R4, which places our requirements for the use of the NSP in the Part 3 and outsources the details of the procedure to ASCE 41. Given passage of 2-3R4, there are no provisions within the Provisions that can be referred to.

Hamburger (YR): My reservation is only with regard to Design Review. I am concerned that we are increasingly burdening the design process with this requirement. While this may be appropriate for new technologies that are understood by only a few experts, such as was the case with base isolation many years ago, at some point we need to return to an environment where we rely on the competence of the practicing professional to perform routine design and analysis tasks. In my opinion, pushover analysis, which has now been in use for more than a decade, and is codified as a standard option in many analysis software programs is now a routine technology and does not require expert review. I request that as a minimum, the requirement for design review be limited to the pushover analysis / stability check, rather than the design as an entirety.

TS-2 Response: Persuasive.

The text "Design review as prescribed in Chapter 16 is required" will be replaced with "<u>A review of the nonlinear static analysis shall be performed by an independent team having experience in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under earthquake loading, composed of at least two members including at least one registered design professional.</u>"

Klingner (YR): I believe that this proposal is technically reasonable and deserves to proceed forward. I am voting Yes with Reservations because I agree with the YR that it received from TS-2. TS-2 is respectfully requested to address Charney's comment.

TS-2 Response: Nonpersuasive .

Charney's YR comment was on Proposal 2-4C. It will be addressed in the response to that proposal.

Proposal 2-4C (Y=19, YR=2, N=1, NV= 0 --95%)

TS 2 addressed the Saunders' comment and will have an editorial adjustment. The PUC agreed by a vote (22,0,0). During the discussion on Ghosh's comment, part of it was satisfied by the Saunders resolution and a minor change. Ghosh withdrew his comment. Klingner's comment was editorially persuasive and part of it too was tied to the Saunders resolution. Proposal 2-4C will be modified and proceed to member organization ballot.

Saunders (YR): I think the commentary proposed by Charney is a better explanation.

TS-2 Response: Persuasive.

The last sentence: "The occupancy importance factor, I, was inserted into Eq. 5.2-16 to ensure that the permissible axial load level does not increase as the importance factor increases" will be replaced with "The occupancy importance factor, I, was inserted into Eq. 12.8-16 to correct an error in ASCE-7. The stability coefficient is based on the elastic stiffness of the system."

Ghosh (No): 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. The language at the end of the first paragraph: "has not been defined" is not clear to me. I also agree with Charney on the I factor. It has been reinstated because it was left out in error. I think we should just state that.

TS-2 Response: Partially persuasive.

The phenomenon observed for steel moment frames is a systems issue and can be expected to be present in many materials that are subject to "in-cycle" degradation, as reflected in the current ATC-62 project.

The phrase "has not been defined" will be replaced with "has not been specified."

See response to Saunders regarding the proposed text for the I factor.

Klingner (YR): I believe that this proposal is technically reasonable and deserves to proceed forward. I am voting Yes with Reservations because I agree with the YR that it received from TS-2. TS-2 is respectfully requested to address Charney's comment.

TS-2 Response: Persuasive.

See response to Saunders.

Proposal 2-5RA (Y=15, YR=1, N=3, NV=3 --84%)

TS 2 presented their proposal in response to comments. They modified references to Chapter 16 by eliminating specific section numbers since Chapter 16 sections were greatly modified. Once the proposal was presented, a motion was made to pass proposal 2-5RA as modified by persuasive comments and edits and it passed the PUC (18,0,0).

Crouse (No): The No vote is based on statement in Proposal for Change that says revisions where made "to align with changes in Proposal 5-2R (Chapter 16) …" I assume

5-2R is a typo and should be 2-5R. Since I voted No on Proposal 2-5R, then this proposal must receive a no vote also.

TS-2 Response: Persuasive.

Thanks for spotting the typo. Proposal 2-5RA will be withdrawn if Proposal 2-5R3 does not pass.

Bachman (No): I will change my vote to Yes if proposal 2-5R3 passes.

TS-2 Response: Persuasive.

Proposal 2-5RA will be withdrawn if Proposal 2-5R3 doe not pass.

Manley (YR): Just a few comments:
- Modify the title of Section 18.3.1 to read as "Nonlinear Response History Analysis Procedure".

TS-2 Response: Nonpersuasive.

An editorial change to the title of Section 18.3.1can be implemented but then also all other titles should include the word "analysis" in front of "procedure" (Response spectrum analysis procedure, etc.). We believe this to be unnecessary.

- Does the second paragraph of Section 18.3.1.2 need to be better coordinated with Section 16.2, where a "suite of not less than seven appropriate ground motions" is required in the analysis?

TS-2 Response: Nonpersuasive.

Section 18.3.1.2 will require coordination with Chapter 16. However, at the time of development of the proposal, the final form of Chapter 16 was not known and the approach selected was done so to avoid any changes The change will have to be made in the next cycle when the final form of Chapter 16 will be known. There is no harm in keeping the existing form of Section 18.3.1.2.

Holmes (No): My vote would be changed to Y if the following inconsistencies are corrected in this proposal or in 2-5R3.:

1. This proposal references Chapter 16 for use of the Nonlinear Static Procedure. Chapter 16 does not cover the NSP, nor is there instruction for this procedure in Chapter 18.
2. This chapter calls for analysis at the Design Earthquake Level and the MCE. Chapter 16 only covers the MCE.
3. This chapter references Section 17.3.2 (isolation chapter) for Ground Motion Histories. Section 17.3.2 still calls for scaling of Ground Motion Histories by using SRSS and 1.3 times the design spectra (unless I missed a proposal

somewhere). This is inconsistent with Chapter 16 referenced in this proposal (section 18.3.1).

TS-2 Response: Partially Persuasive.

The primary reason the reference to Chapter 16 was made was "to align with the changes in proposal 2-5R (Chapter 16)". Unfortunately, the document received by PUC did not highlight the changes made to the document that would have made clear that the changes made were only to avoid explicit reference to sections of Chapter 16 and to rather generally refer to Chapter 16 as the exact form of the chapter was not known prior to PUC approval. Attached is Chapter 18 with changes highlighted. We hope that this explanation would result in reversal of the NO vote.

In addition, Holmes cites reasons (a) Chapter 16 only covers the DE and not the MCE and (b) Reference is made to ground motion histories in Chapter 17. As seen in the attached highlighted proposal, there is no conflict between Chapter 18 and Chapter 16 in relation to item (a). Furthermore, the reference to Chapter 17 was already in Chapter 18 of ASCE 7-05 and rejection of the proposal will maintain the problem. We believe that the change in both Chapters 17 and 18 can be made as editorial change later in the final editing of the document.

Excerpts from the pertinent sections of Chapter 18 are provided below:

18.3.1 Nonlinear Response History Procedure. A nonlinear response history (time history) analysis shall utilize a mathematical model of the structure and the damping system as provided in Chapter 16~~Section 16.2.2~~ and this section. The model shall directly account for the nonlinear hysteretic behavior of elements of the structure and the damping devices to determine its response, through methods of numerical integration, to suites of ground motions compatible with the design response spectrum for the site.

The analysis shall be performed in accordance with Chapter 16~~Section 16.2~~ together with the requirements of this section. Inherent damping of the structure shall not be taken greater than five percent of critical unless test data consistent with levels of deformation at or just below the effective yield displacement of the seismic force-resisting system support higher values.

18.3.1.2 Response Parameters. **For each ground motion analyzed, individual response parameters consisting of the maximum value of the individual member forces, member inelastic deformations and story drifts at each story shall be determined.** ~~**In addition to the response parameters given in Section 16.2.4,**~~ **Moreover, for each ground motion used for response history analysis, individual response parameters consisting of the maximum value of the discrete damping device forces, displacements, and velocities, in the case of velocity-dependent devices, shall be determined.**

18.3.2 Nonlinear Static Procedure. Nonlinear static procedures may be used to construct the lateral force-displacement curve of the seismic force-resisting system in lieu of the elastoplastic curve assumed in the response spectrum procedure and in the equivalent lateral force procedure. When nonlinear static procedures is used, t~~T~~he nonlinear modeling described in ~~Section 16.2.2 and the lateral loads described in Section 16.2 shall be applied to the seismic force-resisting system~~. Chapter 16 shall be used. The resulting force-displacement curve shall be used in lieu of the assumed effective yield displacement, D_Y, of Eq. 18.6-10 to calculate the effective ductility demand due to the design earthquake, μ_D, and due to the maximum considered earthquake, μ_M, in Equations 18.6-8 and 18.6-9, respectively. The value of (R/C_d) shall be taken as 1.0 in Eq. 18.4-4, 18.4-5, 18.4-8, and 18.4-9 for the response spectrum procedure, and in Eq. 18.5-6, 18.5-7 and 18.5-15 for the equivalent lateral force procedure.

Proposal 3-5 Rev2 (Y=18, YR=3, N=1, NV=0 --95%)

TS 3 discussed each comment. In responding to the comments, TS 3 provided a revised proposal 3-5R2 for consideration believing this satisfied the Holmes, Malley and Bachman comments. During discussion, the PUC determined that paragraph 5 of Sec. 11.8.3 should not be in this section and wanted it deleted. Furthermore, the revised proposal should include more explanation of the terminology of MCE . The Wood comment was primarily a clarification again on MCE. The PUC voted (23,0,0) approving this proposal as modified, 3-5R3.

Between the September 8-9, 2008 PUC meeting that approved this proposal and the December 9, 2008 PUC meeting, we were notified that there was a data error in the preparation of the pga maps. USGS corrected the error and generated new maps for this proposal. These were distributed to the PUC for review and TS 3 explained that the new maps did not affect the technical part of the proposal. TS 3 made a motion for the PUC to substitute the new maps and the PUC approved by a vote (19,0,0).

Holmes (No): I find Tom Hale’s comment persuasive, but it doesn’t go far enough because no acceptability criterion is given. Parts of the superstructure are likely to be inelastic for soil movements from the MCE. Do we want 2/3 of the limiting inelastic force? 2/3 of the force of an elastic analysis performed on the distorted structure? Given this force (whatever it is), is the member designed to normal code limits? If we are requiring geotechnical estimation of soil movements in the MCE, I think we have to deal directly with the consequences. For normal buildings, we are trying to prevent collapse in the MCE. Consistent with this goal, the structure should be shown to remain stable under MCE soil movements, and that should be the requirement. Consideration of movements for 2/3 motions, or design for 2/3 forces will be complicated by trying to combine these actions with seismic actions using R factors.

It is also questionable if this design requirement should be placed in the “Geologic Hazards and Geotechnical Investigation” section.

TS-3 Response – Partially Persuasive.

It was intended that these requirements only apply to foundation elements. We have revised the proposal to clarify and to indicate it is to be combined with the forces determined from the design base shear. While we agree that the structure should be remain stable during the MCE, current design practice for most structures only explicitly considers the DE and most engineers would rather not deal with collapse analysis. Therefore this translation to design forces is necessary. Please see Proposal 3-5 Rev 3 for changes that we believe respond to Mr. Holmes concern.

Malley(YR): I find the comment by Tom Hale somewhat persuasive. I would prefer that the response recognize displacement control in some manner.

TS-3 Response – Partially Persuasive

See response to Holmes.

Wood (YR): This proposal refers to the maximum considered earthquake. Are changes required if the Project 07 recommendations are approved? Will the proposal refer to the risk targeted earthquake?

TS-3 Response – Clarification

No. The proposal fully intends to use the maximum considered earthquake for liquefaction and soil deformation analyses. There is not intent to use the risk targeted earthquake for these analyses.

Bachman YR): I am in general support of the proposal and helped develop it. The proposal got developed late in the cycle and really did not get full TS-3 review. Since the proposal was submitted, it has since been fully vetted by TS-3 which resulted in some recommended changes and a revised proposal. For protocol reasons, I was asked by the TS-3 chair to submit the following revised proposed that has been approved unanimously by TS-3 (at least that is my understanding).

TS-3 Response – Persuasive

We will make the change requested plus the additional changes made to respond to Holmes. See Proposal 3-5Rev 3 which follows.

PROPOSAL 3-5 (2008) – Rev 3 (8-25-08)

SCOPE: 11.8

Revise Provisions Section 11.8 as follows:

11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

11.8.1 Site Limitation for Seismic Design Categories E and F. A structure assigned to Seismic Design Category E or F shall not be located where there is a known potential for an active fault to cause rupture of the ground surface at the structure.

11.8.2 Geotechnical Investigation Report Requirements for Seismic Design Categories C through F. A geotechnical investigation report shall be provided for a structure assigned to Seismic Design Category C, D, E or F in accordance with this section. An investigation shall be conducted and a report shall be submitted that shall include an evaluation of the following potential geologic and seismic hazards:

a. slope instability;
b. liquefaction;
c. total and differential settlement; and
d. surface displacement due to faulting or lateral spreading.

The report shall contain recommendations for appropriate foundation designs or other measures to mitigate the effects of the above hazards.

Exception: Where deemed appropriate by the authority having jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide sufficient direction relative to the proposed construction.

11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F. The geotechnical investigation report for a structure assigned to Seismic Design Category D, E or F shall include:

1. The determination of dynamic seismic lateral earth pressures on basement and retaining walls due to design earthquake ground motions.
2. The potential for liquefaction and soil strength loss, including consequences in item 3, evaluated for site peak ground acceleration~~s~~, earthquake magnitude~~s~~, and source characteristics consistent with the maximum considered~~design~~ earthquake ground motions. Peak ground acceleration shall ~~is permitted to~~ be determined based on either (1) a site-specific study taking into account soil amplification effects as specified in Section 11.4.7, or, ~~in the absence of such a study,~~ (2) the peak ground acceleration~~s shall be assumed equal to $S_s/2.5$~~, PGA_M, from Eq. 11.8-1.

$$PGA_M = F_{PGA}\,PGA \qquad \text{(Eq. 11.8-1)}$$

where

PGA_M = Maximum considered earthquake peak ground acceleration adjusted for Site Class effects.

PGA = Mapped maximum considered earthquake peak ground acceleration shown in Figures 22-_ through 22-_.

F_{PGA} = Site coefficient from Table 11.8-1.

3. Assessment of potential consequences of liquefaction and soil strength loss, including, but not limited to, estimation of total and differential settlement, lateral soil movement, lateral soil loads on foundations, reduction in foundation soil- bearing capacity, soil downdrag and loss in lateral soil reaction for pile foundations, increases in soil lateral pressures on retaining walls, and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, ~~ground stabilization~~selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, ground stabilization, or any combination of these measures and how they shall be considered in the design of the structure.
5. Where ~~structures~~ foundation elements are designed to resist forces resulting from soil liquefaction and associated consequences, the ~~structural~~foundation member design force is permitted to be taken as two-thirds of the force developed from the soil deformation response to maximum considered earthquake ground motions combined with forces determined from the design base shear.

Table 11.8-1 Site Coefficient F_{PGA}

Site Class	**Mapped Maximum Considered Peak Ground Acceleration, PGA**				
	PGA ≤ 0.1	**PGA = 0.2**	**PGA = 0.3**	**PGA = 0.4**	**PGA ≥ 0.5**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				
Note: Use straight-line interpolation for intermediate values of PGA.					

PROPOSAL 3-5 (2008) – Rev 2REB (8-15-08)

SCOPE: 11.8

Revise Provisions Section 11.8 as follows:

11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

11.8.1 Site Limitation for Seismic Design Categories E and F. A structure assigned to Seismic Design Category E or F shall not be located where there is a known potential for an active fault to cause rupture of the ground surface at the structure.

11.8.2 Geotechnical Investigation Report Requirements for Seismic Design Categories C through F. A geotechnical investigation report shall be provided for a structure assigned to Seismic Design Category C, D, E or F in accordance with this section. An investigation shall be conducted and a report shall be submitted that shall include an evaluation of the following potential geologic and seismic hazards:

e. slope instability;
f. liquefaction;
g. total and differential settlement; and
h. surface displacement due to faulting or lateral spreading.

The report shall contain recommendations for appropriate foundation designs or other measures to mitigate the effects of the above hazards.

> *Exception:* Where deemed appropriate by the authority having jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide sufficient direction relative to the proposed construction.

11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F. The geotechnical investigation report for a structure assigned to Seismic Design Category D, E or F shall include:

1. The determination of dynamic seismic lateral earth pressures on basement and retaining walls due to design earthquake ground motions.
2. The potential for liquefaction and soil strength loss, including consequences in item 3, evaluated for site peak ground acceleration~~s~~, earthquake magnitude~~s~~, and source characteristics consistent with the maximum considered~~design~~ earthquake ground motions. Peak ground acceleration shall ~~is permitted to~~ be determined based on either (1) a site-specific study taking into account soil amplification effects as specified in Section 11.4.7, or, ~~in the absence of such a study,~~ (2) the

peak ground acceleration~~s shall be assumed equal to $S_s/2.5$~~, PGA_M, from Eq. 11.8-1.

$$PGA_M = F_{PGA}\, PGA \qquad \text{(Eq. 11.8-1)}$$

where

PGA_M = Maximum considered earthquake peak ground acceleration adjusted for Site Class effects.

PGA = Mapped maximum considered earthquake peak ground acceleration shown in Figures 22-_ through 22-_.

F_{PGA} = Site coefficient from Table 11.8-1.

3. Assessment of potential consequences of liquefaction and soil strength loss, including, but not limited to, estimation of total and differential settlement, lateral soil movement, lateral soil loads on foundations, reduction in foundation soil-bearing capacity, soil downdrag and loss in lateral soil reaction for pile foundations, increases in soil lateral pressures on retaining walls, and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, ~~ground stabilization~~selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, ground stabilization, or any combination of these measures and how they shall be considered in the design of the structure.
5. Where structures are designed to resist forces resulting from soil liquefaction and associated consequences, the structural member design force is permitted to be taken as two-thirds of the force developed from the response to maximum considered earthquake ground motions.

Table 11.8-1 Site Coefficient F_{PGA}

Site Class	Mapped Maximum Considered Peak Ground Acceleration, PGA				
	$PGA \le 0.1$	$PGA = 0.2$	$PGA = 0.3$	$PGA = 0.4$	$PGA \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				
Note: Use straight-line interpolation for intermediate values of PGA.					

REASON FOR PROPOSAL:

No change to the reason statement other than it is my understanding that the above proposal now represents the unanimous opinion of TS-3.

Proposal 6-3R (Y=24, YR=3, N=1, NV=9 --96%)

TS 6 focused on the first AISC comment and found it to be persuasive. Consequently, there is a substantive change of increasing the rotation capacity at a minimum story drift from 2% to 2.5%. The PUC voted (23,0,0) to find AISC persuasive and this requires that Proposal 6-3 be modified and re-balloted to the Member Organizations. The HSEAB and SEAW were found either persuasive or editorial and the PUC voted (23,0,0) approving the responses of TS 6. ASME was found nonresponsive by a vote (23,0,0)

Proposal 6-4R (Y=10, YR=3, N=5, NV=4 --72%)

TS 6 developed this proposal and was questioned about it at a previous PUC meeting. It was taken back and TS 6 developed seismic performance factors by using the ATC 63 methodology that was at the 90% development stage over the spring and summer of 2008. Dr. Uang presented the response to comments based on the ATC 63 analysis and pointed out that it did not necessarily have a full peer review as prescribed in ATC 63 because of time. The PUC listened to Dr. Uang's detailed discussion and reviewed the summary of data of the ATC 63 analysis. The PUC chair stated before the PUC vote was taken that this may be a precedent setting event. Once others see that an ATC 63 methodology was used, they may conclude that if they follow this procedure it could guarantee their new system will be favorably accepted. A PUC member commented that this may not be precedent setting since the ATC 63 process has not been published. Consequently, the PUC could approve this proposal based on the best analysis procedures they had at the time. The PUC voted (16,0,5) approving this proposal to be forwarded to member organizations for ballot.

Hooper (No): Reason: I appreciate the follow-on effort to technically justified the proposed seismic performance factors (SPF) using the ATC-63 Methodology. Significant work was performed and the study prepared by Sato and Uang indicate that the proposed SPF do, indeed, perform at a level consistent with the intent of ATC-63. However, several key aspects of the Methodology do not appear to have been followed as indicated below:

1. An independent peer review was not implemented. This is an integral part of the methodology to ensure that the selected archetypes, the design provisions, the experimental research and the analytical modeling are properly selected and implemented. It's also important for the independent validation of the selected ratings which have a direct effect on whether the system "passes".

TS 6 Response: Nonpersuasive. *The system development team was given a very short time (about a month) to complete the ATC-63 study in order to meet the July 1, 2008 deadline. Although a peer review panel (Ken Wood–Chair, Victor Azzi, Roger*

Brokenbrough, Clarkson Pinkham) was established when the study started, the time was too tight to facilitate the interaction between the team and the panel. The peer review panel received the ATC-63 study report on July 1, 2008. One panelist (Brokenbrough) did respond with no negative comments.

2. The selected ratings haven't been sufficiently justified. Ratings of "Excellent/Superior" for test data and design requirements seem inflated (see item 3, below). A modeling rating of "good" might be accurate but not enough justification was provided.

TS 6 Response: Partially persuasive. *Three papers were included as appendices in the July 1, 2008 submittal in support of the ratings used in the ATC-63 study. One paper ("Cyclic Testing of Cold-Formed Steel Special Bolted Moment Frame Connections," see Appendix 1 in the July 1, 2008 submittal) shows that the hysteresis behavior is highly reproducible in all 9 beam-column subassembly test specimens because slip and especially bearing action in the bolted moment connection dominated the yield mechanism. Considering that the quality of test data for both Reinforced Concrete Moment Frames (both Special and Ordinary) and Wood Light-Frame System reported in ATC-63 90% Draft was rated as "Good," for its less variability the quality of test data for CFS-SBMF was judged to be "Superior" in this study.*

Based on Kircher Item 3 comment, the rating for design requirements has been changed from Superior to Good in the updated ATC-63 study report. A summary of the updated results is provided in the Appendix.

3. AISI S110 was not made available for review. While the appendix material indicates that the specification is based on the outlined methodology, it's not clear what is included in AISI S110 and how it may (or may not) affect the archetypes. Also, it's not clear whether the AISI S110 Specification appropriately limits the size, gauge and shapes of the members. Only two beam gauges and one column thickness were included in the archetypes.

TS 6 Response:
Nonpersuasive. *(a) AISI S110 and its Commentary were provided as part of the original proposal.*
Persuasive. *(b) The range of beam depth (C12 to C20) used in the archetype design is identical to that used in the cyclic testing program. To be consistent with the test database, the limitations on both beam depth and steel grade are added (in italic) in Section D1.2.1 of AISI S110:*

> *"**D1.2.1 Beam Limitations***
> *In addition to the requirements of Section D1.2.3, beams in CFS-SBMF systems shall be ASTM A653 galvanized 55 ksi (374 MPa) yield strength cold-formed steel C-sections members with lips, and designed in accordance with Chapter C of AISI S100. The beam depth shall be not less than 12 in (305 mm) or greater than 20 in (508 mm). The flat depth-to-thickness ratio of the web shall not exceed* $6.18\sqrt{E/F_y}$.

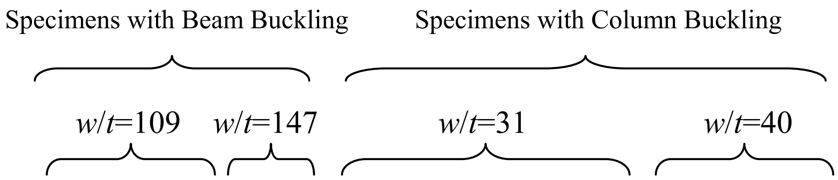
The size of square HSS (HSS 8 to 12) used in archetype design extend somewhat beyond the test database (HSS 8 to 10); a deeper HSS section is sometimes used to increase the lateral stiffness of the frame. The following modification is made to Section D1.2.2:

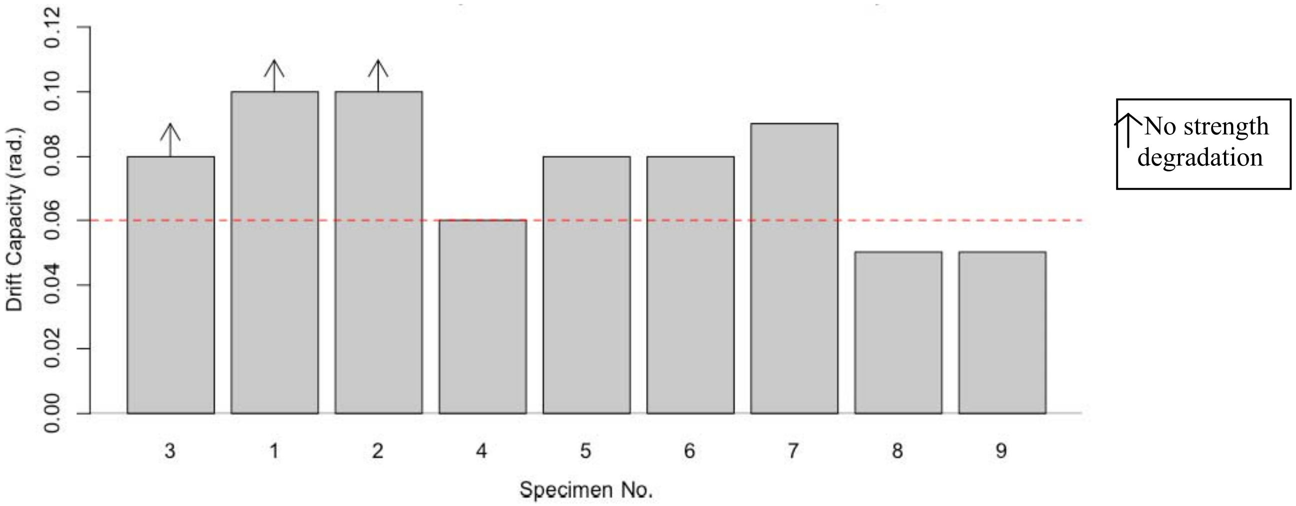
> *"**D1.2.2 Column Limitations***
> *In addition to the requirements of D1.2.3, columns in CFS-SBMF systems shall be ASTM A500 Grade B cold-formed steel hollow structural section (HSS) members painted with a standard industrial finished surface, and designed in accordance with Chapter C of AISI S100. The column depth shall be not less than 8 in (203 mm) or greater than 12 in (305 mm). The flat depth-to-thickness ratio shall not exceed* $1.58\sqrt{E/F_y}$.

Note that surface treatment for both beam and column is included in the modification for consistency with that used in the testing program.

4. The change in selection of the "lower bound solution" from 5% to 6% raises questions regarding what is the right value for this "non-simulated" failure mode, since it is this change that justifies the selected SPF. The original test report indicates that buckling may, indeed, occur at 5% drift, so it seems that going to 6% for this non-simulated failure mode, even with the written justification in the ATC-63 study, needs additional review.

TS 6 Response: Nonpersuasive. *A summary of the drift capacity, conservatively defined as that when the strength degradation is no more than 10% of the peak strength, for all 9 test subassemblies is shown in the following figure. Specimen 3 did not experience any buckling in the beam or column, while the remaining 8 specimens experienced buckling after significant slip/bearing action occurred in the connection region at a story drift beyond 0.04 radians. The mean and standard deviation of the drift capacity are 0.077 and 0.019 radians, respectively. It is judged that a story drift capacity of 0.06 radians is reasonable to define the lower bound collapse limit state. It is too conservative to use 0.05 radian to define collapse limit state.*





5. The "low gravity" archetype used a 50 psf live load. If this beams support a roof, this live load could be as low as 20 psf (or lower with live load reduction). It needs to be verified whether this live load affects the results.

TS 6 Response: Nonpersuasive. *When a 20 psf roof live load is used, wind load combinations usually govern the design and such design will result in a larger system overstrength which is inappropriate for inclusion in the ATC-63 study.*

6. The archetypes do not include a structure with a 35' height, which is being requested in the proposal (the maximum height in the studies was 24'). To justify this height limit, the archetypes should include structures with the maximum height.

TS 6 Response: Partially persuasive. *The proposed height limit is 35 ft. But according to Section 4.2 of the ATC-63 90% Draft, "In a statistical sense, index archetype configurations are not intended to represent 'outliers'." Therefore, a 30-ft tall archetype has been added in the study. The result is included in the updated ATC-63 study report.*

7. Finally, the ATC-63 Methodology suggests that C_d be set equal to R. For this system, C_d should be taken, as a minimum of being equal to R, unless a higher value is required by AISI S110.

TS 6 Response: Persuasive. *Setting Cd equal to R implies that the equal displacement rule should be applied. But this Newmark-type of rule is too conservative when cyclic testing showed that CFS-SBMF exhibits a very significant hardening in strength due to bearing action in the connection region. Based on a statistical evaluation of the R-μ-T relationship, the rationale for using Cd = R/1.2 was presented by Uang to PUC on April 8, 2008. Based on the similar comment from several PUC members, however, the value of Cd has been increased from 3.0 to 3.5 in Table 12.2-1.*

I would change my vote to "Yes" if the above items are addressed.

Holmes (No): My previous vote was contingent on an ATC 63 analysis. The submitted ATC63 analysis does not conform to ATC 63 requirements for the following reasons:

1. No peer review. The issues below may have been resolved by a peer review.

TS 6 Response: *See response to Hooper Item 1.*

2. It is unclear if the tributary lateral loads are ever taken as more than one bay. The pictured structure appears to have only perimeter frames. This could change the relationship of gravity and lateral loads.

TS 6 Response: Persuasive. *SBMF of the same-size beams and same-size columns are expected to be used along each grid line in the direction under consideration. The design of all the archetypes was based on an interior frame configuration with one bay as the tributary width. In response to this comment, Section D1 of S110 is modified as follows:*

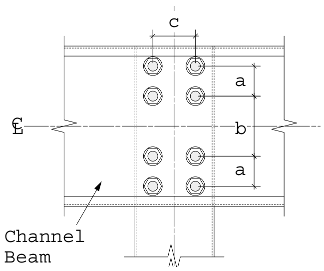
> *"**D1 Cold-Formed Steel Special Bolted Moment Frames (CFS-SBMF)***
> *Cold-formed steel–special bolted moment frames (CFS-SBMF) systems shall withstand significant inelastic deformations through friction and bearing at their bolted connections. Beams, columns, and connections shall satisfy the requirements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

Additional limitation on the bolt group configuration is given in Item 3 below.

3. The answers to Aschheim's questions about friction suggest only one combination of material finishes was tested (for friction values). Do the AISI design rules also limit the finishes in this way? I think the uncertainty of the key parameter has not been properly considered.

TS 6 Response: Persuasive. *To be consistent with those used in the testing program, the grade of steel and the associated surface treatment have been added in Sections D1.2.1 and D1.2.2 (see response to Hooper Item 3.) In addition, Section D1.1.1 is modified as follows:*

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



(figure in Table D1-1)

4. The system is limited to 35’ height, but the design space did not go nearly this tall.

TS 6 Response: *See response to Hooper Item 6.*

Aschheim (No): I do not find responses to several of my comments to the earlier proposal to be persuasive, as follows:

- Response to Comment 3: no assurance has been given that use of R=3.0 (without capacity design, slenderness limits…) leads to acceptable behavior. Absent convincing support, this option should be prohibited.

TS 6 Response: Nonpersuasive. *The exemption from seismic design requirements for steel structures assigned to SDCs B and C is a general clause which is applicable to other types of steel structures. This approach for CFS-SMBF is the same as that for, say, steel Ordinary Moment Frames designed for R=3.5 and satisfying the capacity design requirements outlined in Section 11 of the AISC Seismic Provisions. These seismic design requirements are not required when the designer elects to design an R=3 moment frame. (Without any special seismic detailing, large ductility capacity through slip/bearing is still inherent in an R=3 frame.)*

- Response to Comment 4: the Provisions should contain an easily enforceable maximum value for R associated with slip. A pile of design calculations is no substitute. A ceiling on R is also needed to prevent permanent deformation in minor earthquakes.

TS 6 Response: Nonpersuasive*. The seismic response of SBMF is affected by not only slip but also bearing in bolted moment connections. The design procedure in AISI S110 explicitly requires the designer to compute the slip component of story drift. The story drift and system overstrength calculations also include the effect of bolt slippage. An R value based on the slip strength can be on the order of 5 or higher!*

- Response to Comment 5: “usually does not have architectural attachments” provides no solace where architectural attachments are present. The logic suggests a prohibition of such attachments, or the use of attachments that are demonstrated to withstand the expected drifts. Even so, I still have discomfort over the allowed drift levels.

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.*

- Response to Comment 6: my concern has to do with the modeling for the P-Delta check—given the existence of a nonlinear model, there probably needs to be some clarification that a linear model is to be used for the conventional P-Delta check.

TS 6 Response: Nonpersuasive. *Both the story drift evaluation and P-Delta analysis are consistent with those used in ASCE-7. Therefore, a linear model is to be used for design calculation.*

- Response to Comment 7: since the friction forces developed are dependent on the coating, there should be explicit language relating the friction force evaluation to the coatings being used.

TS 6 Response: *See response to Holmes, Item 3.*

- Response to Comment 8: Once approved, the system may well be used in situations that depart from the original vision. The presence of sloping sites, or large concentrated loads in some locations, may introduce changes in the relative sizes of beam and column members. Formulas that use terms such as “nVs” have to be explicitly restricted to conditions of uniformity, and text should indicate in some way that the engineer is to use principles of mechanics to evaluate system strength, stiffness, torsional response, and so on.

TS 6 Response: *See response to Holmes Item 2 for the modification to AISI S110 Section D1 on additional restrictions on frame configuration.*

Kircher (No): My No vote is based on the following concerns with the 18-page "ATC-63 Study" by Sato and Uang, summarized below:

(1) First, I congratulate the authors on their attempted use of the ATC-63 methodology; the summary report looks pretty good, as far as it goes, but there are some issues (see below).

No response needed.

(2) The ATC-63 Methodology requires independent Peer Review, as described in Chapter 8 of the 90% Draft. Without peer review, the "Methodology" has not been used as intended. If there had been Peer Review, the reviewers might have caught the following issues (and possibly other items related to test data, design requirements and/or nonlinear modeling/analysis, etc.).

TS 6 Response: *See response to Hooper, Item 1.*

(3) Total Uncertainty. Report assumes Good quality modeling (probably okay, so far) and Superior quality test data and design requirements, respectively (which is too optimistic for a new system) - for total beta of 0.55. A total beta of 0.65 (all Good quality ratings), or 0.60 (e.g., Superior quality test data and Good quality design requirements) would be the smallest amount of total uncertainty I can imagine (for a new system). Based on a total beta of 0.60 (and a collapse story drift capacity of 6%), R = 3.5 almost works (close enough), but does not work for a total beta of 0.65.

TS 6 Response: Partially persuasive*. The quality of design requirements has been changed from Superior to Good, which results in a total beta value of 0.60, in the updated ATC-63 study report.*

(4) Report does not show results of pushover calculations of archetype overstrength (as required by the Methodology). In lieu of archetype results, I suggest proposal be modified to include $\Omega_O = 3$ (consistent with maximum value recommended by Methodology), with footnote permitting more precise building-specific calculation.

TS 6 Response: Persuasive. *The suggested value and the associated footnote are included in the modification to Part 1, Table 12.2-1 of the 2009 Provisions as follows:*

Seismic Force Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Structural System Limitations and Building Height (ft) Limit				
					Seismic Design Category				
					B	C	D	E	F
C12. Cold-Formed Steel – Special Bolted Moment Frame[b]	14.1	3.5	~~NA~~3.0[a]	~~2.9~~3.5	35	35	35	35	35

[a] Alternatively, ~~T~~the seismic load effect with overstrength, E_m, ~~shall~~can be based on the expected strength determined in accordance with AISI S110.

[b] Cold-formed steel – special bolted moment frames shall be limited to one-story in height in accordance with AISI S110.

(5) Proposed value of C_d = 3 is not consistent with Methodology (i.e., Methodology defines $C_d = R$, unless system damping is shown to be typically and substantially

greater than nominal 5% value, at or about yield of the system). I suggest proposal be revised to include $C_d = 3.5$ (i.e., same as proposed value of R).

TS 6 Response: *See response to Hooper Item 7.*

I would change my vote to Yes, provided the above suggestions ($\Omega_O = 3$ and $C_d = 3.5$) are incorporated into the proposal, and the PUC concurs with the proponents assumption that test data support system collapse story drift capacity of at least 6%. Note (to the PUC). Although the Sato/Uang report provides a lot of good information, it should not be considered (even with the above tweaks) to represent the level of effort required by the ATC-63 Methodology for a new system (so the PUC will still need to still use some of the old *seat-of-pants* method to decide the fate of this proposal).

Hamburger (YR): I believe the time has come to recognize that except for structures with significant damping, resulting in effective damping much greater than 5% of critical, the value of C_d cannot possibly be less than the value of R. The poor precedent set by systems already in the Provisions, with values of C_d less than R is not an excuse for perpetuating this myth with new systems. My negative will be withdrawn if the value of C_d is taken as 3.5.

TS 6 Response: Persuasive*. In addition to significant damping, significant hardening can also result in a Cd factor less than R. But the Cd value has been increased from 3.0 to 3.5; See response to Hooper Item 7.*

Gilengerten (No): The “NA” entry in the Omega-naught column of the table could mislead. It would be better if instead of “NA”, the table entry was “See footnote a”.

TS 6 Response: *See response to Kircher Item 4.*

The drift limits proposed are excessive. There are no restrictions on the types of cladding and nonstructural elements and systems used in these structures. The high drift limits and flexibility of this system may generate significant damage to nonstructural systems under very nominal seismic demand. While the structural system can sustain these drifts, more consideration of the serviceability is needed.

TS 6 Response: *See response to Aschheim Comment 5.*

The second paragraph of the reason for proposal suggests the R=3.5 be used for preliminary design, and some other approach be used for final design. The proposed change to the Provisions and Commentary do not describe the other approach. I understand that Omega-naught is a product of calculation, but is R also subject to recalculation?

TS 6 Response: Persuasive. *The “procedure” mentioned in the second paragraph of the Reason refers to a procedure in AISI S110 to calculate the seismic load effect with* *overstrength, Em, such that the system ovserstrength factor* Ω_o*, does not have to be listed in Table 12.2-1. Upon the suggestion of Kircher, Item 4, a conservative* Ω_o *value is provided in the table. Alternatively, footnote "a" directs the designer to this procedure to calculate Em.*

I would change my vote to "Yes" if an appropriate drift limit is proposed and the design procedure is clarified.

Klingner (YR): This proposal is generally sound, and the proponents have made a good-faith effort to comply with the spirit and as much of the letter as possible of the ATC-63 90% draft. The purpose of my YR is to place the following comments on the record.

- We should work toward the development of uniform criteria for vetting technical data that are used in support of proposals for seismic design factors. In the case of AAC, the technical data were presented in the form of 3 refereed journal publications, including an EERI Spectra publication defending the proposed approach (since ATC-63 did not yet exist). Those 3 refereed journal publications were supported by many conference proceedings, 2 PhD dissertations, and 3 MS theses. The work had been through review at many levels. In April 2008 with the steel racks tested at UCSD, PUC has been presented evidence consisting of a single research report. PUC and ASCE7 must decide whether the supporting documentation is broad enough to address the behavior that might reasonably be anticipated, and whether the supporting documentation has been subjected to sufficient peer review. I suggest that the required amount of supporting documentation should depend on the extent to which the proposed system departs from what is known. I also suggest that peer review should always be required. Given the current two-year lag times between submittal and publication, it may be unreasonable to ask that the supporting document be published in a peer-reviewed journal. Nevertheless, it is not unreasonable to ask that the supporting document include an article that has been reviewed and accepted for publication in a peer-reviewed journal.

No response needed.

- We should work toward the development of uniform criteria for vetting ATC-63 type evaluations. I note that the final version of the ATC-63 report has not even been published.

No response needed.

Dolan (YR): While I agree with the proposal in principle, I would like to know how the subcommittee voted on this proposal. There is not indication as to whether or not the proposal was unanimously or whether comments were raised.

TS 6 Response: *TS6 voted on first draft of proposal.*

Appendix

Summary of Updated ATC-63 Study

Changes made in the updated study:

(1) Expected material strengths other than nominal strengths were used in the updated analysis.
(2) Rating for design requirements was changed from Superior to Good.
(3) A 30-ft tall archetype (ID No. 13) was added in the most critical group (Maximum Seismic, Low Gravity Design).

Note: Figure or table numbers correspond to those in the original ATC-63 study report submitted on July 1, 2008. For example, Table 1m is the modifed Table 1 based on the updated study.

Table 1m Archetype Structural Design Properties

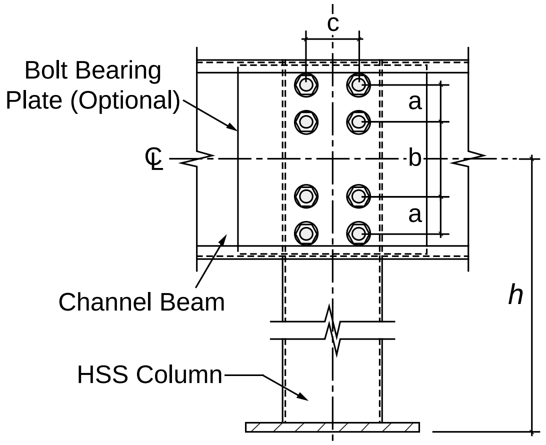
Archetype Design ID Number	Number of Span	Span (ft)	Bay (ft)	Height (ft)	Key Archetype Design Parameters			
					Seismic Design Criteria			S_{MT} [T], g
					SDC	R	T	
Maximum Seismic (D_{max}) and Low Gravity Designs								
3	1	20.0	16	16	D_{max}	3.5	0.87	1.04
5	2	21.4	20	10.25	D_{max}	3.5	0.51	1.50
10	2	23.375	20	16	D_{max}	3.5	0.77	1.16
13	2	21.4	12	30	D_{max}	3.5	1.36	0.66
Maximum Seismic (D_{max}) and High Gravity Designs								
1	2	21.4	16	16	D_{max}	3.5	0.82	1.10
2	1	20.0	16	16	D_{max}	3.5	0.73	1.24
8	2	21.4	14	24	D_{max}	3.5	1.08	0.84
12	1	22.29	16	11	D_{max}	3.5	0.57	1.50
Minimum Seismic (D_{min}) and Low Gravity Designs								
4	1	22.0	20	8.5	D_{min}	3.5	0.43	0.71
7	2	20.0	17	12	D_{min}	3.5	0.95	0.32
11	2	24.0	14	16	D_{min}	3.5	1.10	0.27
Minimum Seismic (D_{min}) and High Gravity Designs								
6	2	24.2	12	18	D_{min}	3.5	0.90	0.33
9	1	22.0	14	9.5	D_{min}	3.5	0.46	0.66

All archetypes are one story height.

Table 2m Archetype Structure Model Size

Archetype Design ID Number	Beam (in)	Column (in)	Live Load *L*, (psf)	Dead Load *D*, (pdf)
1	2C20×3.5×10Ga	HSS10×10×1/4	125	12
2	2C16×3.5×10Ga	HSS10×10×1/4	125	12
3	2C20×3.5×10Ga	HSS10×10×1/4	50	55
4	2C12×3.5×12Ga	HSS8×8×1/4	50	12
5	2C16×3.5×10Ga	HSS8×8×1/4	50	12
6	2C20×3.5×10Ga	HSS10×10×1/4	125	12
7	2C16×3.5×10Ga	HSS8×8×1/4	50	55
8	2C20×3.5×10Ga	HSS12×12×1/4	125	12
9	2C16×3.5×10Ga	HSS8×8×1/4	125	12
10	2C16×3.5×10Ga	HSS10×10×1/4	50	12
11	2C16×3.5×10Ga	HSS10×10×1/4	50	55
12	2C20×3.5×10Ga	HSS8×8×1/4	125	12
13	2C20×3.5×10Ga	HSS12×12×1/4	125	12

Table 3m Bold Configuration

<table>
<tr><th>Archetype Design ID Number</th><th>a (in)</th><th>b (in)</th><th>c (in)</th><td rowspan="14">

Bolted Moment Connection
1-in. snug-tight high-strength bolt
Slip Coefficient $k = 0.33$,
Bolt tension force $T = 10$ (kips)</td></tr>
<tr><td>1</td><td>3</td><td>10</td><td>6¼</td></tr>
<tr><td>2</td><td>3</td><td>6</td><td>6¼</td></tr>
<tr><td>3</td><td>3</td><td>10</td><td>6¼</td></tr>
<tr><td>4</td><td>2½</td><td>3</td><td>4¼</td></tr>
<tr><td>5</td><td>3</td><td>6</td><td>4¼</td></tr>
<tr><td>6</td><td>3</td><td>10</td><td>6¼</td></tr>
<tr><td>7</td><td>3</td><td>6</td><td>4¼</td></tr>
<tr><td>8</td><td>3</td><td>10</td><td>8¼</td></tr>
<tr><td>9</td><td>3</td><td>6</td><td>4¼</td></tr>
<tr><td>10</td><td>3</td><td>6</td><td>6¼</td></tr>
<tr><td>11</td><td>3</td><td>6</td><td>6¼</td></tr>
<tr><td>12</td><td>3</td><td>10</td><td>4¼</td></tr>
<tr><td>13</td><td>3</td><td>10</td><td>8¼</td></tr>
</table>

Table 4m Total System Collapse Uncertainty β_{TOT}

Quality of Test Data	Quality of Design Requirements			
	A- Superior	B-Good	C-Fair	D-Poor
(A) Superior	0.55	**0.60**	0.70	0.85
(B) Good	0.60	0.65	0.75	0.85
(C) Fair	0.70	0.75	0.80	0.95
(D) Poor	0.85	0.85	0.95	1.05

Table 5m Acceptable Value of Adjusted Collapse Margin Ratio (ACMR10% and ACMR20%)

Total System Collapse Uncertainty β_{TOT}	Collapse Probability				
	5%	10%	15%	20%	25%
		ACMR10%		*ACMR20%*	
0.55	2.47	2.02	1.77	1.59	1.45
0.60	2.68	**2.16**	1.86	**1.66**	1.50
0.65	2.91	2.30	1.96	1.73	1.55

Table 10m Summary of Collapse Results for Archetype Designs (Lower Bound, 6% Drift)

Archetype Design ID Number	Design Configuration		Pushover and IDA Results			
	No. of Span.	SDC	μ_C (SDR = 6.0%)	S_{MT} [T], g	$\hat{S}_{CT}$ [T], g	*CMR*
Maximum Seismic (D_{max}) and Low Gravity Designs						
3	1	D_{max}	8.9	1.04	1.35	1.30
5	2	D_{max}	9.6	1.50	2.38	1.59
10	2	D_{max}	10.1	1.16	1.67	1.44
13	2	D_{max}	7.9	0.66	1.17	1.77
Maximum Seismic (D_{max}) and High Gravity Designs						
1	2	D_{max}	8.9	1.10	1.54	1.40
2	1	D_{max}	10.3	1.24	1.95	1.57
8	2	D_{max}	9.4	0.84	1.31	1.56
12	1	D_{max}	7.2	1.50	2.35	1.57
Minimum Seismic (D_{min}) and Low Gravity Designs						
4	1	D_{min}	11.4	0.71	2.29	3.23
7	2	D_{min}	8.5	0.32	0.84	2.63
11	2	D_{min}	10.1	0.27	0.85	3.15
Minimum Seismic (D_{min}) and High Gravity Designs						
6	2	D_{min}	7.9	0.33	1.43	4.33
9	1	D_{min}	9.8	0.66	2.66	4.03

Table 11m Summary of Final Collapse Margins and Comparison to Acceptance Criteria (Lower Bound, 6% Drift)

Archetype Design ID Number	Design Configuration		Computed Collapse Margin				Acceptance Check	
	No. of Span.	*SDC*	*CMR*	μ_C	*SSF*	*ACMR*	Accept. *ACMR*	Pass/ Fail
Maximum Seismic (D_{max}) and Low Gravity Designs								
3	1	D_{max}	1.30	8.9	1.44	1.82	1.66	Pass
5	2	D_{max}	1.59	9.6	1.34	2.13	1.66	Pass
10	2	D_{max}	1.44	10.1	1.41	2.03	1.66	Pass
13	2	D_{max}	1.77	7.9	1.58	2.80	1.66	Pass
Mean						2.20	2.16	Pass
Maximum Seismic (D_{max}) and High Gravity Designs								
1	2	D_{max}	1.40	8.9	1.43	2.00	1.66	Pass
2	1	D_{max}	1.57	10.3	1.40	2.20	1.66	Pass
8	2	D_{max}	1.56	9.4	1.49	2.32	1.66	Pass
12	1	D_{max}	1.57	7.2	1.34	2.10	1.66	Pass
Mean						2.16	2.16	Pass
Minimum Seismic (D_{min}) and Low Gravity Designs								
4	1	D_{min}	3.23	11.4	1.14	3.68	1.66	Pass
7	2	D_{min}	2.63	8.5	1.24	3.26	1.66	Pass
11	2	D_{min}	3.15	10.1	1.28	4.03	1.66	Pass
Mean						3.66	2.16	Pass
Minimum Seismic (D_{min}) and High Gravity Designs								
6	2	D_{min}	4.33	7.9	1.23	5.33	1.66	Pass
9	1	D_{min}	4.03	9.8	1.15	4.63	1.66	Pass
Mean						4.98	2.16	Pass

Proposal 8-43R1 (Y=25 , YR= 4, N=3 , NV= 5 --91%)

TS 8 first addressed PCA's comment and found it persuasive. TS8 will modify the proposal. The PUC agreed by a vote (22,0,1). PPCI's comment was similar and PUC agreed with the same response to PCA. The NAHB comment was found non persuasive and the PUC agreed (23,0,0). Clarification on residential structures was provided in the TS 8 response showing that chimneys on houses is covered by Chapter 13. The SEA (all 4) comments were the same. TS 8 found them persuasive and the change has been made to the proposal. Since this is a substantive change, the proposal must be re balloted to the member organizations. This passed the PUC by a vote (23,0,0). The ASME comment is on a Yes vote and TS8 noted it. This proposal will be re balloted.

General Responses and Discussion by TS-8 on Comment Received to Proposals 8-31R-A, 8-44R-A and 8-50RR-A

General Discussion: TS-8 recognizes that a minority of the full PUC do not support these proposals and that their positions may be supported by enough of the full PUC to not allow the negatives to be overturned (a 2/3rds majority of the meeting quorum is required by PUC rules to sustain a finding of non-persuasive of a negative). But in discussions with the PUC chair, it was agreed that some discussion regarding these proposals should occur at the PUC meeting. The following discussion is intended to cover most if not all the issues raised in these 3 proposals so we don't have to duplicate discussions and expedite resolution of them. It is intended to provide clarity about the positions that appear to have been misconstrued and hopefully will help lead us to understand on how we are going to deal with these issues over the next decade in a time of limited resources. These discussions are intended to clear the air and be constructive and not derisive. The below discussion is organized by topic and will be referred to in response to specific comments on these 3 proposals.

Voting of July 15-Aug 15 Ballot: It very important to note that for some reason, a large block of voters (7 voters which represents 25% of the PUC) did not turn in their PUC ballots, that another 4 voters decided not to vote on Proposals 8-31R and 8-41R, and 3 voters decided not to vote on ballot 8-50RR. During the last PUC ballot we had 26 of 28 voters who cast their ballot and this time we had only 21. On the last PUC ballot for Proposals 8-31, 8-44 and 8-50R we had 7, 7 and 8 negative votes respectively. On this PUC ballot for Proposals 8-31R, 8-44R and 8-50RR we had 9, 9 and 10 negative votes respectively. Because we had such a large group of voters not returning their ballots or deciding not to vote, we cannot characterize the vote an overwhelming negative on these proposals. Rather it appears to characterize a troubling lack of participation and interest on the part of some voters, and a reluctance to address in a workable context an issue of pressing concern to the industry.

Direction of the PUC based on last PUC meeting: There seem to be many who did not understand the vote of the last PUC meeting. All three of these proposals were **PASSED** by the PUC by at least a 2/3rds majority determined by mail ballot. However, even though between 50% and 60% of the voters present at the last PUC meeting agreed to find the negatives non-persuasive 40% to 50% did not. Therefore, we found ourselves in a quandary. After discussions with the PUC Chair, it was agreed that the proposals would be resubmitted with new reason statements with the hope that the revised reasons would be persuasive to the negative commentors or would be persuasive to enough of the other PUC members to permit the negative voters to be found non-persuasive. This is the reason that the revised proposals were prepared and submitted. It is important to keep in mind that a 2/3rds majority of the PUC supported the proposals in the original letter ballot, and that what we are trying to address the concerns of a minority. .

Decisions made by the BSSC Board during last PUC Cycle: There appears to be confusion regarding the decisions made by the BSSC Board on the topics covered by these proposals in the last PUC cycle. There were two proposals regarding these issues during the last PUC associated with nonbuilding structures. One proposal dealt with permitted increases passed the PUC and member ballot, including resolution of negatives. A second proposal dealing with unlimited increases failed to pass the member ballot by one vote. One member voter at the last PUC meeting of the last cycle requested to change his vote to Y rather than NV and was called out of order. So the proposal did not pass. The next day, the BSSC Board decided that it would not object if the second proposal was considered by adoption by ASCE 7. ASCE 7 subsequently decided to adopt the second proposal.

The PUC should note that members of TS-13 (now included in TS-8) did not request this action by the BSSC Board and were not present at the BSSC Board meeting, nor was Jim Malley. It is not understood why the actions of the last cycle PUC and BSSC Board continue to be mischaracterized by Mr. Malley.

Use of 90% Draft of ATC-63 Report and Paper by Helmut Krawinkler and Farzin Zarien in Reason Statement: Mr. Kircher made a point that the 90% Draft Report should not be referenced in the reason statement for these proposals since it is still a draft and the examples were specifically not intended for making any code type decisions or inferences. His point is valid, however, we find ourselves in a Catch 22. The 90% Draft was published since our last PUC meeting so if we did not review and consider in our reason statement we would be faulted for not discussing (in fact it is used for justification for Proposal 6-4, and there are comments by the PUC faulting the proponents for failure to conform to ATC 63 requirements).

Furthermore the concept of trading of lower R values for increasing permitted heights was provided in the 90% draft as noted in quoted statements in the reason statements. Helmut Krawinkler and Zarien's paper referenced that quoted statement in the 90% draft and we felt it was therefore important to include as part of our reason statement. We agree that the factor of 2 reduction recommended Krawinkler and Zarien's paper in the R factors applied to special moment frames and not to flexural shear walls and there are no recommendations regarding the types of structural systems in these proposals. Any extrapolation to other systems was made out of context and should not be used for justification (TS-8 will be providing our apologies for Profs. Krawinkler and Zarien for this over zealous mischaracterization on our part).

That said, there is nothing in either the ATC-63 draft or the Krawinkler paper that would indicate that what is being proposed is not conservative. Rather, both studies suggest that use of the proposed R factors will result in "nonbuilding" structures that are more likely to meet the 10% probability of collapse target than their "building" structures counterparts. All the evidence to date indicates it is reasonable to in general reduce R values for taller structures subject to higher ground motions, for some if not most systems. That is what these proposals do.

The minority voters at the last PUC meeting wanted more technical justification for these proposals. Within the context of the information available, accounting for the fact that the

ATC-63 project is still a work in progress, and based on the direction of current research, this has been done.

Canadian Paper by C. Izvernari, M. Lacerte, and R. Tremblay. TS-8 became aware of this paper in late June of 2008 and did not obtain a copy of this paper for distribution until after July 1st which was the apparent drop dead deadline for submitting PUC proposals. We transmitted the paper to BSSC in early July and asked it be transmitted with our proposals, which they graciously agreed to do. Since the deadline had passed, we were unable to include the Canadian Paper in our reason statement and provided more rationale of its relevance. The reason for including was to provide as much technical justification that we could within the financial limitations of the members of TS-8.

Concept that these proposals should be initiated by TS-6 and TS-2 rather than TS-8. Mr. Hamburger and Mr. Kircher have indicated that proposals such as these should be initiated by TS-6 and TS-2. With regards to proposals 8-31R and 8-44R, it is the judgment of TS-8 that it is in their area of responsibility since if falls within Chapter 15 of ASCE 7.

TS-6 appears to have a primary focus on steel buildings assigned to Seismic Design Categories D, E and F. We welcome their constructive input but not their veto if it does not up line with their philosophy for "normal buildings". On proposal 8-50RR, TS-8 did its best to coordinate and involve TS-2 and TS-6 and attempted to resolve as many of their concerns as possible before submitting the proposal to the PUC for balloting. A significant issue for TS-8 is that neither TS-2 or TS-6 has assigned anyone to be an advocate for the provisions addressing large industrial buildings, especially with long span structures that utilize truss moment frames as their seismic force resisting system. . Neither TS-2 or TS-6 has taken the time to generate needed proposals, even though it would be best if it was done by them. These proposals are urgent matters for the individuals who design industrial buildings but there is no sense of urgency in TS-2 or TS-6 to help the practicing engineers in these areas. Rather, TS8 was essentially directed to go do a comprehensive set of ATC-63 type of analyses and associated tests on the types of structures you want to construct and prove to use what you are doing is safe. These are not "new" structural systems being proposed, we seek only to provide workable guidance to the design community on structures that are and must be built. On a final note, we are not aware of any PUC rule that restricts a TS from making a proposal on any issue in the Recommended Provisions. We did think it is good protocol however to coordinate such proposals with the TS that is responsible for the section under consideration for change.

Need for objective supporting analyses to justify proposal height limit and R values. The primary theme of a majority of the negative voters was they wanted objective supporting analysis to justify the R values and height limits provided in these proposals. They imply they would like ATC-63 type studies be performed (not withstanding that the fact that currently only a draft report for the project has been published). This apparently has become the defacto "gold standard" for some of the PUC members. It should be remembered that all current R values, height limits and restrictions that are currently in the NEHRP Provisions today are based on judgment, including AAC. The PUC has not officially adopted the ATC-63 project report as the basis for establishing R values, height limits and restrictions. In fact no such procedures of any kind currently exist in the provisions for establishing R values, height limits and restrictions. Furthermore, there is absolutely nothing currently in the proposed Commentary of Chapter 12 that will be voted on at Sept 8-9 meeting of the PUC that provides any basis for how the height limits and restrictions for structures in Table 12.2-1 were established.

Who would fund comprehensive ATC-63 type studies for these systems? The other common theme is that if AAC and the steel side bolted connection groups fund ATC-63 type studies, why can't TS-8 do the same? The problem is for the types of industrial structures that we are discussing there are no vested interests (except the general good of the public). Most of the owners and large E&Cs who do this type of design and construction are large publicly held companies. They have been constructing these types of structures for decades.

NIST has agreed to support a new project ATC-76, which will evaluate 4 existing structural systems none of which considers any of the structural systems we have in these proposals at the height limits and design categories we are proposing. The question is who does this effort, how will it be funded, by who, and over what time frame? TS-8 is a technical subcommittee that does not represent any of the building materials interests. We can't wait another 10 years to sort this out.

The following discussion items apply to specific comments associated with Proposal 8-50RR.

Concept that steel OCBFs and OMFs are new systems and the height limits are new. Mr. Dolan makes the point that these should be treated as all new systems. These are not new systems and the height limits are not new. The same height limits and restrictions existed for these systems in the 1994 NEHRP Provisions and the 1997 UBC. However they have morphed over time with no justification other than judgment (See the proposed Chapter 12 commentary). Meanwhile, the detailing for these systems has become much more stringent and R values being proposed for them are extremely low.

Use of 90% Draft ATC-63 Examples for Estimating R values of concrete OMFs at higher SDCs. Mr. Kircher makes the point that the examples provided in Chapter 9 (and we would assume also Chapter 10) of the 90% Draft should not be used for making code decisions. His point is valid. We agree that the values should not be directly used. But if we were to assume that the examples provided in Chapter 9 were valid and deemed acceptable, is the approach described in the reason statement of Proposal 8-50RR reasonable for estimating the R it would take to satisfy the collapse criteria in a preliminary fashion? Obviously when one is doing ATC-63 type studies to establish R factor values for new systems, one is going to do his best to judge what R values to use for a given height and SDC range of motion. This is so that only a minimum number of designs need to be developed and analyzed. Coming up with trial designs is part of the normal design engineering design process and takes judgment. It is suggested that the approach described in the reason statement is a rationale way to do this. The type of approach used to establish trial designs really should be described in the ATC-63 report other than trial and error which is currently the default option. Mr. Leon makes the point that we should do better than crude extrapolations. Frankly, we are of the opinion that the current design parameters for many, if not most, structural systems is based on crude extrapolations and engineering judgment.

Proposal 8-31R (Y=6, YR=3, N=9, NV=4 --50%)

Proposal failed. There is no further action on this proposal this cycle.

Malley (No): This part of comment would not result in a No vote, but YR. That is that the incorporation of BRBF should be consistent with the changes that have been made by the PUC in this cycle to consolidate the BRBF's into one system that has moment connections. We do not need the two lines for "non-moment-resisting beam-column connections" and the corresponding "With permitted height increase".

The remaining comments are the basis for the No vote. What is the basis for cutting the R factor in half, other than to respond to Messrs. Hamburger and Saunders? While using the argument that this has been done before is true, it was by no means the direction that a majority of the PUC agreed to at the end of the last cycle, but we were overruled by the BSSC Board. Perpetuating this approach is not acceptable to me.

While I recognize and appreciate the attempt by TS 8 to provide some technical justification, I believe that what is presented in the Reason for Proposal is not acceptable for a number of reasons. The attempt to use the results of ATC 63 studies on R/C moment frames and extrapolate to the expected performance of steel braced frame systems is surprising, given the vastly different response, failure modes and deformation limits of these systems. The OMF systems studied in ATC 63 were down at Sd1 of 0.2, and we are being asked to use the results for Sd1 of greater than 0.6. The level of nonlinearity, strength ratios and other factors are very different, and it is hard to expect that the results would be at all similar. The reference to Krawinkler and Zarein seems to have been taken very much out of context. The paper is addressing how tall frame structures run into P-Delta problems caused by shear mode story mechanisms more quickly than shorter buildings. To suggest that this paper argues for increasing height limits seems a bit odd. I requested input from Greg Deierlein and Helmut Krawinkler and both believe that the extrapolations used as the primary technical justifications for this proposal (ATC 63 and the Krawinkler/Zarein paper) are not appropriate.

Believe it or not, I am supportive of the concept of trading R factor for other design parameters (like SDC or height limits), but I will not support major changes that do not have technical justification that is specific to the proposal. The PUC has established this precedent with AAC and the bolted light gage moment frames, and we should not allow the “justification bar” to be lowered for major changes to existing lines in the various Design Parameter tables.

I do not have a suggested revision that would satisfy me, since I believe that TS8 did not follow the guidance given at the previous PUC meeting to provide data specific to the type of structures being considered in this proposal, and just giving a suggested revision to the numeric changes is just perpetuating what I believe we can not allow to continue.

> ***TS-8 Response:*** *See above Persuasive on the on BRBF issues and the ATC 63 and Krawinkler/Zarien references and Non-Persuasive on the rest.*

Leon: (No): I have no problem with the inclusion of the special systems but disagree with the lines labeled “with permitted height increase.” As for Proposal 8-50RR, I do not agree with the justification given for the proposed changes in height limitations. In all three cases, ATC-63 research, a paper by Krawinkler and Zarein, and ‘engineering judgment” are cited as the main justification:

- Insofar as the ATC-63 data is concerned, I disagree with the blanket conclusion that braced frames will show the same trends in performance as moment frames. In my experience, performance of braced frames is far more sensitive to uncertainties in member imperfections and connection detailing than moment frames, for which the extensive SAC program has provided a good understanding of the detailing needed.
- I am troubled by the extrapolation that halving the value of the R factors can be used to eliminate height limits in high seismic zones; the citation of the Krawinkler and Zarein to justify this approach for braced frames is not supported by the authors, who limit themselves to remarks on frame and wall structures. I am not saying that the trend (trading R factors for height limitations) is necessarily wrong, but I am saying that the proposal carries no data to justify this extension to braced frames. I may have misread the article, but I found no assumptions on different detailing levels in the studies; I believe that the values of Θ_p , Θ_{pc}/Θ_p and λ used in the Krawinkler and Zarein work are meant to provide a spectrum of performance but are not tied to any particular detailing requirements. Finally, I will note that the authors are careful to point out the limitations of their work, including the fact that the conclusions are valid in a relative but not necessarily in a strict quantitative sense (first paragraph, p. 14 of article).
- The use of ‘engineering judgment” is not sufficient in my view, as we should be striving to provide hard engineering data to support new code provisions wherever possible. In this case, it is true that providing such data will not be a trivial effort; on the other hand, I think that we have enough knowledge to develop that data. Are there no studies that could be used to provide a better justification? I recognize that the committee in the past has made similar decision based on engineering judgment; I just think that we now have the tools to do better and this committee needs to show leadership in that area.

In all three cases, I would consider changing my NO vote if more technical references are provided and/or the “Reason for Proposal” clearly indicates that this is purely based on engineering judgment.

> ***TS-8 Response. See above.*** *Persuasive on the ATC-63 and Krawinkler/Zarien references and Non-Persuasive on the rest.*

Saunders (YR): While I agree with the comments of Jim Malley, I am personally inclined to be a bit more lenient with respect to Non-Building Structures than Buildings (see comments on 8-31R). These structures have many differences from buildings, and many different design constraints. I would change my vote to Yes if the correction for BRBFs noted by Malley is made, and if appropriate reasons and commentary are provided. What I object to in this case is the attempt to justify the changes thru the cited research. The real reason for these proposed changes is expediency and judgment. Since judgment based on past performance experience is the basis for the majority of the current R factors, I would prefer that the reasons and commentary for these proposals focus on that, rather than confusing the issue by quoting non-applicable research. I don't think we can hold out for ATC-63 results for the variety of structures that this proposal covers.

> ***TS-8 Response. Persuasive.*** *We will make the changes requested.*

Hooper (No): Reason: The reason for the "No" vote is associated with the addition of Seismic Performance Factors (SPFs) "With permitted height increase" for the BRBF systems. While this approach was adopted in previous standard development efforts (e.g., ASCE 7-05), there is no real technical justification for the selected values. ATC-63 results are indicated as the primary back-up for the proposed change. It should be noted that the ATC-63 results were for tall concrete moment frame systems, whose performance characteristics are much different from BRBFs and, as such, the ATC-63 results are not germane.

I would support the proposal, and the selected SPFs, if technical justification was provided.

> ***TS-8 Response. Non-Persuasive****. See above opening discussion.*

Wood (YR): There seems to be an inconsistency in the values of C_d in Table 15.4-1. The three primary systems are (a) special steel concentrically braced frames, (b) buckling-restrained braced frames, non-moment-resisting beam-column connections, and (c) buckling-restrained braced frames, moment resisting beam-column connections. With the height limits, the values of C_d are (a) 5, (b) 5 ½ and (c) 5. However, with the permitted height increase, the values of C_d are (a) 3, (b) 3½ and (c) 4. Why is (a) = (c) < (b) for the first case, but (a) < (b) < (c) for the second case?

> ***TS-8. Non-Persuasive.*** *The Cd values for the basic systems are taken equal to values in Table 12.2-1. The Cd values for the systems with permitted height increases and with unlimited height are taken equal to the corresponding R values (the equal displacement rule). It is our opinion that Cd should be taken equal to R unless very specific knowledge is known about the system. The Cd values of the basic systems are the responsibility of TS-2. We do not know how they were established and take no responsibility for justifying them.*

Ghosh (No): While the revised reason statement is a significant improvement over the original reason statement, it does not really justify the proposal.

After repeated attempts, I am not able to see how either the Krawinkler-Zareian paper or the ATC-63 report justifies the proposal. As far as I can tell, there are two primary conclusions from the Krawinkler-Zareian paper. First, "probability of collapse at the MCE level is very different for frame structures than for wall structures." I think most people would agree that braced frames are much closer in their behavior to walls than they are to moment frames. Probability of collapse is found to be period-dependent only for frame structures. Second, "the R-factor approach employed in present codes is inappropriate since it leads to designs with vastly different probabilities of collapse."

The ATC-63 report concluded, as accurately stated in the reason: "Strength requirements could be increased for taller buildings, by using a period dependent R-factor; in a recent paper, Krawinkler and Zareian (2007) illustrated how the R factor would need to change, as a function of period, in order to create uniform collapse probabilities for moment frame buildings of varying height." But then the following reason statement, "While the ATC-63 project studied certain types of systems, it stands to reason that the same types of increases in strength should be required for braced frame structures with longer periods and increased heights." is a stretch. The reason statement needs to explain "why it stands to reason." The first conclusion I cited from the Krawinkler-Zareian paper seems to directly contradict this statement.

The basic concept of this proposal, that of trading height limit for strength, started with ASCE 7-05 Table 15.4-1, as has been stated in the "Reason for Proposal." At that time, the permitted trade-off was limited to certain structural systems on the basis of performance experience (at least that is what I understood). What is the basis of the choice of structural systems for this proposal? There cannot possibly be a record of performance experience for buckling-restrained braced frames. A request from Harold Sprague (or any individual for that matter), I do not believe, can be sufficient justification.

The proposed 400-ft height limit is a dizzying height. The tallest height limit in the entire Table 12.2-1 is 160 ft, except that it is extendable to 240 ft in certain situations. 400 ft is a leap and makes me uneasy at a fundamental level. TS-8 might as well have gone for no limit (NL).

> ***TS-8 Response.*** *See above opening discussion. Persuasive on the ATC-63 and Krawinkler/Zarien references and Non-Persuasive on the rest.*

Manley (No): Unfortunately, I am still not convinced that adequate technical justification has been presented supporting the reduction in R values for a corresponding increase in height of these systems. I recognize that this was the approach that was used in the last cycle for other systems in Table 15.4-1, however I believe it is unfair to permit "business as usual" for these chosen building systems, while requesting thorough ATC-63 analyses for others.

> ***TS-8 Response. Non-Persuasive.*** *See above opening discussion.*

Aschheim ((YR): I would prefer to see changes to beam-to-column strength ratios to ensure better behavior for the taller structures, to better limit sidesway/story mechanisms. Where is the evidence for buckling restrained frames?

> ***TS-8 Response. Non-Persuasive.*** *See above opening discussion. We do not have any detailed studies to defend such change to beam to column ratios but we would fully expect the increase in system strength would provide adequate conservatism with the existing detailing provisions.*

Kircher (No): If TS 8 feels it is appropriate to relax height limits for nonbuilding structures designed for lower values of the *R* factor, then I do not object (even if it doesn't make a lot of sense). However, the Reason for Proposal should not rely on (not refer to) the ATC -63 Draft Report as the technical basis for this proposal (since the example studies in the 90% Draft do not apply - see comments on TS 8-50RR). I would change my vote to Yes, if the proposal is revised (Reason is revised) to not rely on the ATC-63 Draft Report as the technical basis for this proposal (and TS-8 still supports the proposal without relying on the ATC-63 Draft Report).

> ***TS-8 Response: Persuasive.*** *We will delete the reference to the ATC-63 Draft Report.*

Hamburger (No): Now that a rational approach (ATC-63) exists to evaluate the adequacy of framing systems under various design parameters (R, Cd, Ωo) as well as height limits, it makes no sense to be making arbitrary and judgmentally based adjustments to these parameters without appropriate use of the methodology (ATC-63). I do not find that the submitted paper by Krawinkler and Zareian provides sufficient justification to support this proposal. Furthermore, proposals for change to the limits for steel systems should come from TS6, not TS8.

> ***TS-8 Response:*** *See above opening discussion. Persuasive regarding the Krawinkler and Zarien reference but not persuasive regarding the remaining comments.*

Klingner (No): The proponents argue, based on ATC-63 results, that collapse probabilities can be preserved by halving R as heights increase. I am not completely convinced by this argument, because it presumes that the base values for R (before dividing by 2) give reasonable probabilities of failure. Furthermore, there is no justification for extending the table to other types of braced frames for which no ATC-63 data exist. My N can be satisfied by evidence that the proposed seismic design factors are justified by something like the procedure currently under development by ATC-63. I believe that this proposal should be discussed at a PUC meeting.

TS-8 Response. Non-Persuasive. *See above opening discussion.*

Dolan (No): While I understand the desire to trade height for strength, I do not agree that simply adding strength should be sufficient for adding height. With the introduction of ATC-63, I believe that the prudent method would be to complete a numerical investigation of the consequences of adding height. The committee says that it would be expensive, well essentially this is almost equivalent to adding a new system to the R-factor table. We require all new systems to stand up to the more rigorous review, why should steel frames be any different. This proposal increases the allowable height by a factor of 3 with little analytical or other justification. Are we going to simply allow all structural systems raise the allowable heights with the same minimal justification?

TS-8 Response. *Non-Persuasive. See above opening discussion*

Proposal 8-44R (Y=67 YR=1, N=9, NV=5 --47%)

This proposal failed. There is no further action on this proposal this cycle.

Leon: (No): I have no problem with the inclusion of the special systems but disagree with the lines labeled “with permitted height increase.” As for Proposal 8-50RR, I do not agree with the justification given for the proposed changes in height limitations. In all three cases, ATC-63 research, a paper by Krawinkler and Zarein, and ‘engineering judgment” are cited as the main justification:

- Insofar as the ATC-63 data is concerned, I disagree with the blanket conclusion that braced frames will show the same trends in performance as moment frames. In my experience, performance of braced frames is far more sensitive to uncertainties in member imperfections and connection detailing than moment frames, for which the extensive SAC program has provided a good understanding of the detailing needed.
- I am troubled by the extrapolation that halving the value of the R factors can be used to eliminate height limits in high seismic zones; the citation of the Krawinkler and Zarein to justify this approach for braced frames is not supported by the authors, who limit themselves to remarks on frame and wall structures. I am not saying that the trend (trading R factors for height limitations) is necessarily wrong, but I am saying that the proposal carries no data to justify this extension to braced frames. I may have misread the article, but I found no assumptions on different detailing levels in the studies; I believe that the values of Θ_p , Θ_{pc}/Θ_p and λ used in the Krawinkler and Zarein work are meant to provide a spectrum of performance but are not tied to any particular detailing requirements. Finally, I will note that the authors are careful to point out the limitations of their work, including the fact that the conclusions are valid in a relative but not necessarily in a strict quantitative sense (first paragraph, p. 14 of article).
- The use of ‘engineering judgment” is not sufficient in my view, as we should be striving to provide hard engineering data to support new code provisions wherever

> possible. In this case, it is true that providing such data will not be a trivial effort; on the other hand, I think that we have enough knowledge to develop that data. Are there no studies that could be used to provide a better justification? I recognize that the committee in the past has made similar decision based on engineering judgment; I just think that we now have the tools to do better and this committee needs to show leadership in that area.

In all three cases, I would consider changing my NO vote if more technical references are provided and/or the "Reason for Proposal" clearly indicates that this is purely based on engineering judgment.

> ***TS-8 Response.*** *See above opening discussion. Persuasive on the ATC-63 and Krawinkler/Zarien references and Non-Persuasive on the rest.*

Malley (No): Incorporating composite systems into the table with design parameters similar to building structures is acceptable to me.

However, the height limit increase is not. Please see the comments to Proposal 8-31R for the reasons for my negative.

> ***TS-8 Response.*** *See Response to Malley on 8-31R.*

Saunders (YR): See reason for YR vote on. 8-31R

> ***TS-8 Response***. *See Response to Saunders on 8-31R.*

Hooper (No): Reason: The reason for the "No" vote is the same as for Proposal 8-31R having to do "With permitted height increase" for the proposed composite system.

> ***TS-8 Response.*** *See Response to Hooper on 8-31R.*

Ghosh (No): My reasons are essentially the same as written for Proposal 8-31R-A, although some of what I wrote there does not apply directly.

> ***TS-8 Response.*** *See Response to Ghosh on 8-31R.*

Manley (No): See my negative on Proposal 8-31R.

> ***TS-8 Response.*** *See Response to Manley on 8-31R.*

Aschheim (NV with Reservation): As stated in my response to 8-31R, more floors are jeopardized when story mechanisms develop in taller structures—where is the evidence to support the proposed changes?

> ***TS-8 Response.*** *See Response to Aschheim on 8-31R.*

Kircher (No): See comment to No vote on TS 8-31R (2009)

TS-8 Response. *See Response to Kircher on 8-31R.*

Hamburger (No): My reason for this negative is the same as the reason for my negative on Proposal 8-31R.

TS-8 Response. *See Response to Hamburger on 8-31R.*

Klingner (No): This proposal may be technically sound. My Negative can be satisfied by evidence that the proposed seismic design factors and SDC and height limits for concentrically braced steel frames are justified by something like the procedure currently under development by ATC-63. The proponents argue, based on ATC-63 results, that collapse probabilities can be preserved by halving R as heights increase. I am not completely convinced by this argument, because it presumes that the base values for R (before dividing by 2) give reasonable probabilities of failure. I believe that this proposal should be discussed at a PUC meeting. Also, based on working group and PUC discussions of this issue in 2006 and 2007, I do not believe that PUC has clearly articulated the intent of height limitations, so it is difficult for us to compare the consequences of decreases in R combined with increases in height limits.

Dolan (No): While I understand the desire to trade height for strength, I do not agree that simply adding strength should be sufficient for adding height. With the introduction of ATC-63, I believe that the prudent method would be to complete a numerical investigation of the consequences of adding height. The committee says that it would be expensive, well essentially this is almost equivalent to adding a new system to the R-factor table. We require all new systems to stand up to the more rigorous review, why should steel frames be any different. This proposal increases the allowable height by a factor of 3 with little analytical or other justification. Are we going to simply allow all structural systems raise the allowable heights with the same minimal justification?

TS-8 Response. Non-Persuasive. *See above opening discussion*

Proposal 8-50RR (Y=7, YR=2, N=10, NV=3 --47%)

This proposal failed. There is no additional action on this proposal this cycle.

Leon: (No): My rationale for voting No is similar for that in Proposals 8-31 and 8-44R. Moreover, I believe that the more extensive "Reason for Proposal" given for 8-50RR tries to muddy the waters. The basic reason for the proposal is stated in lines 4-17 of p. 6; this ballot is basically a judgment call for each committee member to make based on their experience and intuition. I am troubled by the attempt to provide a technical reason where a sound technical reason does not exist. In my view, the use of terms such as "crudely extrapolated values" (line 21 of page 5), however true, does not belong in the technical justification for a proposal in 2009. I also read the Izvernari, Lacerte and Tremblay paper which is fare more germane to this discussion but found no reference to it in the "Reason for Proposal." The Izvermari et al. paper shows that the Canadian approach of trading increased forces for lower ductilities ($R_{Canada}=2$) works for concentrically braced frames up to about 200 ft. (or 16 stories); however, if my reading of the Canadian code is correct, the base shear values would be higher than those implied by our codes for similar ductilities. Thus extrapolation beyond about 200 ft. is not supported by this paper.

In all three cases, I would consider changing my NO vote if more technical references are provided and/or the "Reason for Proposal" clearly indicates that this is purely based on engineering judgment.

> ***TS-8 Response: See Response to Leon on Proposal 8-31R regarding 8-31R comments.***
> *With regards to the comments above we find them Non-Persuasive. The reason for considering the above paper was to provide additional justification for permitting higher height limits for lower Rs. We are not requesting in this proposal height limits above 160 feet and further our R is 1 rather than 2. It would seem what we are proposing is very conservative relative to the Canadians.*

Malley (No): I do not support any of the changes in this proposal, except that in 12.2.5.4.1, which is a good clarification.

A consensus group of TS6 held a conference call on 8/11/08. None of the members that were able to attend (5 total) support this proposal either.

Please see my comments on 8-31R and 8-44R above related to the discussion on ATC 63 and the Krawlinkler/Zareian paper that are identical in this proposal. In the interest of saving a few trees, I will not re-state any of that discussion on this reason, but it all applies.

I am troubled that TS8 would choose to hold up the non-building structure applications as precedent for this proposal, given the lack of PUC consensus on this change during the previous cycle.

I am also concerned about referring to a previous “extensive justification” that was not accepted by the PUC in our last meeting. The reason that there is an RR version of this proposal is because the justification was not acceptable.

While we (TS6) understand that the intent of this proposal is for industrial buildings, there is nothing in the proposal that limits it to such applications. There is no such definition in any standard that I am aware of that would allow any distinction.

The OCBF and OMF systems are low ductility systems that are highly susceptible to story mechanisms (thus their very limited height applications in the higher SDC’s.). Their design to R=1 provides no certainty that collapse prevention in the MCE will be achieved if inelastic response is required, and the additional allowable height provides more opportunities for story mechanisms to form.

Steel OMF systems and R/C OMF systems have very different design philosophies, detailing requirements and expected levels of ductility in member and connection response. It is hard to understand how one could try to extrapolate the results as has been proposed. I also would like to address lines 33 through 37 of page 5 of the proposal. How the results from studies of R/C OMF’s in low SDC’s can be extrapolated to Steel OMF and OCBF’s in high SDC’s with unlimited height is quite a leap of faith in my mind. I am not swayed by the following statements in the discussion that there has not been time or funding to do the studies to provide more direct technical justification.

> ***TS-8 Response:*** *First it is extremely disappointing that TS-6 decided to have a teleconference regarding this proposal and did not invite a member of TS-8 to participate. This is simply counter-productive Also as indicated above it is very interesting that TS-6 has concerns with this proposal with R = 1 and some level of required seismic detailing, but seems to have no problem with R=3 with no seismic detailing with unlimited height in SDC C. We find the above comments Non-Persuasive. See above opening discussion.*

Saunders (No): I agree with the comments of Jim Malley and participated in the teleconference he has described in his response. In addition to the factors described in Jim's comments, I think that considering that we are now seeking to avoid collapse in the MCE (RTE), the fact that an R=1 building will ostensibly remain elastic at the DBE becomes somewhat irrelevant. The issue is what will happen in larger EQs. By their nature, the systems included in this proposal (particularly OCBF and OMF) will be susceptible to story mechanisms that could occur at forces not much greater than the design forces at R=1. Without significant ductility, these structures, if tall (160 ft.) would very likely collapse well before the MCE due to P-Delta effect.

Additionally, if we begin to allow trading of R factors for other factors in the table, I think we will be opening Pandora's Box, and we will face many more such proposals.

> ***TS-8 Response. Non-Persuasive.*** *We believe that an R = 1 structural does not remain "ostensibly" elastic at the DE. Actually every element is designed to remain elastic at the DE. TS-8 believe that steel OCBFs and OMFs remain "ostensibly" elastic at the MCE. However we do have a concern that our next collapse criteria being proposed by Project 07 translates in to structures being required to have a median collapse capacity at ground motions of 1.5 times or greater than the MCE. Are we continuing to rachet upward our ultimate seismic design criteria? With regards to Pandoras box, please note that the height limits and restrictions being proposed are the same value found in the 1994 NEHRP, but now with much lower R values and much more extensive detailing requirements. We really don't have a problem with criteria with systems existing in the 1994 NEHRP being reopened for consideration by this same approach* **.**

Hooper (No): Reason: The proposed modifications to Section 12.2.5.4.1 do provide need clarification. The reason for the "No" vote is associated with the selection of an R of 1 (and the other SPFs) without appropriate technical justification. While it appears as though an R of 1 will provide an appropriately low probability of collapse given the MCE, truly brittle systems, even with this R value, will not provide the margin that Provisions are intending to provide.

I would support the proposal, with appropriate SPFs, if technical justification was provided.

> ***TS-8 Response. Non-Persuasive.*** *We can not agree with Mr. Hooper's characterization of steel OCBFs and OMFs designed to satisfy AISC 341-05 as "brittle" systems. See response to above response to Malley and Saunders.*

Wood (Yes): p. 1, line 27 and p. 2, line 33 "The height limit in Table 12.2-1" should be "The height limits…"

> ***TS-8 Response. Persuasive.*** *We will make the change requested.*

Ghosh (No): What I said about justification in my comment on Proposal 8-31R-A directly applies to the braced frame parts of this proposal. I have other problems with the justification for the moment frame parts. But I have even bigger problems with this particular proposal.

The basic concept of this proposal, that of trading height limit for strength, started with ASCE 7-05 Table 15.4-1, applicable only to non-building structures similar to buildings. Extending a concept that was discussed and approved for non-building structures to buildings must be approached with abundant caution. By definition, no life safety is at stake in the former case, while life safety is definitely at stake in the latter case.

Malley stated in his negative vote comment on the original Proposal 8-50R: "Since this proposal could be used for any building structure (not just industrial buildings cited in the proposal, since we haven't defined these as a class of buildings), TS2 and TS6 really needed to co-sponsor this proposal ...." I could not agree more. TS2, in particular, needs to discuss and endorse this proposal for it to go forward.

I said in my negative comment on the original Proposal 8-50R: "This proposal needs to address other materials. If the principles on which this proposal is based do not apply to some other material, then it should be explained in the reason. If the principles do apply, then the proposal needs to be expanded. Is past performance experience the basis? Then, this needs to be unequivocally stated."

Here was the response from TS-8 who found me non-persuasive:

"As noted in the reason statement, for whatever reason, special exceptions have always been permitted in structural steel. This proposal was specifically developed at the request of the AISC Ad-Hoc Committee for Industrial Building and Nonbuilding Structures. As noted in the reason statement, TS-8 tried to develop a separate definition for industrial buildings that would the nonbuilding structure permitted structural systems to be used for industrial building applications [this is an exact quote; something is obviously wrong with this sentence]. But we could not get that proposal out of committee. There is no reason that similar concepts could not also be applied to other building structural systems but they would need to deliberated on a case by case by the PUC".

In other words, the proposal really was and is intended for a certain class of buildings, which could not be easily defined. In view of the difficulty, the proposal was simply made generally applicable. I think this proposal should be brought back after that particular class of buildings is properly defined.

As noted in the "Reason for Proposal," the conclusions drawn concerning period-dependent R are really applicable only to concrete moment frame buildings. I don't understand why the system studied at length in the ATC-63 project, namely the ordinary reinforced concrete moment frame system, is left out of the proposal, while many arguments are being made to extrapolate the conclusions to apply to ordinary and intermediate moment frames and braced frames of steel.

According to Reason for Proposal, "A fairly extensive justification (based on judgment) was provided in proposal 8-50R." Here is part of that justification:

"This change is needed to allow the design of certain types of industrial buildings to be constructed in areas of high seismicity where special systems are not feasible. It is fully expected once the ATC-63 project is completed and studies are performed on these ordinary systems, it will be determined that the design coefficients and factors will be shown to be very conservative and will be permitted to be increased. However, in the interim, these conservative values are proposed to permit design and construction of these important types of structures to proceed. It was suggested in discussion, that the alternate means and methods section of the code be used. While this is always a possibility, most firms that design these type facilities do not have the technical horsepower to go this direction and its not really fair to require them to engage high powered consultants to get through a building department. Remember, a few years ago (a few weeks ago in the case of California) these systems were permitted."

The above can be paraphrased to say that this is a "need-based" proposal. The last line in the above paragraph was applicable to braced frames of reinforced concrete and heavy timber as well. Why are those systems left out of the proposal? I personally have received many inquiries about these systems in the little time since California switched from the 1997 UBC to the 2006 IBC.

In summary, this proposal needs to be expanded to include other systems and needs to have proper justification supported by TS2 for me to change my negative vote.

> ***TS-8 Response.*** *We greatly respect the opinions of Dr. Ghosh but we find his comments Non-Persuasive. Please see our responses to Ghosh regarding Proposal 8-31R to reply to his comments on 8-31R. Also see above opening discussion regarding the extrapolation regarding concrete systems associated with the ATC-63 Concrete OMF Example. With regard to his comments regarding why TS-8 did not define a certain class of buildings, TS-8 found that the types of structures are so diverse that definition is nearly impossible, and that in the end the committee determined that the design requirements were stringent enough to provide an adequate level of safety, regardless.*
>
> *With regards to the comment on why this proposal does not also include concrete, the answer is simple. It was not necessary for industrial building applications. Regarding the comment on this proposal being needs based – absolutely. We would hope all proposals are needs based and with regards to why heavy timber and brace frames of concrete have been left out, we cannot answer other than they are not used in industrial applications. Also see response to Saunders above.*

Manley (No): See my negative on Proposal 8-31R.

> ***TS-8 Response.*** *See response to Manley on Proposal 8-31R.*

Holmes (No): The additional arguments since the last vote are not persuasive. Maybe when many, many ATC 63 analyses have been run, we can "fill in" with judgment, but this is not the time. I will still need a more specific use of ATC 63 to justify these changes.

> ***TS-8 Response. Non-Persuasive.*** *See above discussion. ATC-63 is still in draft form, and the PUC has not balloted to make ATC-63 90% draft (or any other version) the defacto standard for acceptance of a structural system. These are systems that are currently in use and have been in the Provisions in the past. ATC-63 studies would be ideal, but unfortunately no sponsor for these studies for*

the types of structural systems being addressed by these proposals has come forward. See also response to Malley and Sauders.

Aschheim (YR): I do not believe the response to Malley as well as to the concerns expressed in the subcommittee ballot is adequate with regard to the concern that nonbuilding structures have not been defined as a class of buildings. Language is needed to restrict applicability.

> ***TS-8 Response. Non-Persuasive.*** *See response to Saunders. The real problem is that the class of buildings we are discussing is real but cannot be designed by the restrictions in Chapter 12. Basically, earlier editions of the Provision permitted structural types such truss moment frames but current provisions don't if the structures are SDC's D, E or F.*

Kircher (No): Procedurally, changes to Chapter 12 should be proposed by (or at least coordinated with) TS-2, rather than brought forward unilaterally by TS-8.

Technically, in brief, the Reason for Proposal attempts, inappropriately, to use results of example studies of the ATC -63 Draft Report as the technical basis for this proposal. Here are some specifics:

(1) Use of Examples. "These examples are intended for illustration only, and are not intended to propose any specific changes to current building codes requirements for any currently approved system" - Section 9.1, ATC-63 90% Draft Report

(2) Generalization of RC OMF and SMF Results to Other Systems. Not appropriate (and possibly very wrong) to assume trends in example results for RC OMF and SMF apply to steel systems (in particular braced frames in high seismic regions).

(3) Use of Methodology. The Methodology is intended as a rational process to establish seismic performance factors (e.g., *R* factor) for new systems (not ad hoc fudging of *R* factors/height limits of current systems). The Methodology could be used to re-evaluate *R* factors (and/or re-evaluate height limits) for existing systems, but would require a dedicated and thorough study (or studies) for this purpose (and results of examples of the 90% Draft Report are not appropriate).

> ***TS-8 Response. Persuasive.*** *We will remove references of the 90% draft in the reason statement. However, we are interested in Mr. Kircher's opinion on the extrapolation discussion in above opening discussion.*

Hamburger (No): My reason for this negative is the same as the reason for my negative on Proposal 8-31R. The paper by Tremblay, et. al, while compelling does not apply to braced frames designed per U.S. criteria. Furthermore, proposals for change to the limits for steel systems should come from TS6, not TS8.

> ***TS-8 Response. Non-Persuasive.*** *See above opening discussion and responses to Leon, Malley and Saunders regarding this proposal.*

Klingner (YR): I believe that the proponents should show evidence that the proposed seismic design factors and SDC and height limits are justified by something like the procedure currently under development by ATC-63. The proponents argue, based on ATC-63 results, that collapse probabilities can be preserved by halving R as heights increase. I am not completely convinced by this argument, because it presumes that the base values for R (before dividing by 2) give reasonable probabilities of failure. Also, based on working group and PUC discussions of this issue in 2006 and 2007, I do not believe that PUC has clearly articulated the intent of height limitations, so it is difficult for us to compare the consequences of decreases in R combined with increases in height limits.

> ***TS-8 Response. Non-Persuasive but Sympathetic.*** *We agree that the PUC has not articulated the intent of height limitations nor provided a technical basis for them. However, as indicated above it does not seem necessary to provide full ATC-63 type evaluations for systems and height limits that previously existed in the NEHRP provisions that have been modified over time without such evaluations or technical justifications.*

Dolan (No): While I understand the desire to trade height for strength, I do not agree that simply adding strength should be sufficient for adding height. With the introduction of ATC-63, I believe that the prudent method would be to complete a numerical investigation of the consequences of adding height. The committee says that it would be expensive, well essentially this is almost equivalent to adding a new system to the R-factor table. We require all new systems to stand up to the more rigorous review, why should steel frames be any different. This proposal increases the allowable height by 60% or more with little analytical or other justification. Are we going to simply allow all structural systems raise the allowable heights with the same minimal justification?

TS-8 Response. Non-Persuasive. *See above opening discussion*

SDPRG 1R4 (Y=13, YR=4, N=3, NV=2 --85%)

SDPRG addressed the Stewart comment and found it nonpersusive. PUC agreed by a vote (22,0,1). Crouse comment was discussed and Crouse withdrew his comment so no vote was needed. The Bachman comment was coupled with a full revised proposal that was included with the comments. This was discussed at length and a motion was made to find Bachman comment nonpesuasive. The PUC voted (12,5,6) and it was thought at first that it failed basically finding it persuasive. However, after a quick review of the voting procedures, it was determined that 12Yes votes divided by the sum of the Yes and No votes (12 + 5 =17) yield 12/17 =70% >2/3. This exceeds the 2/3 requirement and it was passed. The Ghosh comment was found nonpersuasive and the PUC agreed (21,3,0). The Holmes comment was discussed and was withdrawn. The Hamburger comment was found editorial persuasive and those edits will be made. No vote was needed because they were editorial. The Gillengerten comment was discussed and Gillengerten withdrew his comment.

After lengthy discussion and voting on individual comments of this proposal, a PUC member made a motion to show overall acceptance of this proposal. The intent is to demonstrate to others who will review and vote how strong SDPRG 1R4 is accepted by the PUC. The PUC voted and approved (22,0,2).

Once the voting was completed, a comment was made that there should be significant commentary with this proposal describing the development of this proposal and what was considered pertinent and why. The Reason for Proposal was not considered sufficient for this purpose. SDPRG was directed to develop this commentary and to have it prepared by November 15, 2008 to allow an abbreviated PUC ballot and to have any comments reviewed and accepted at another PUC meeting tentatively scheduled for early December, 2008. Although not known at the time, this has served useful since the PUC was unable to complete resolution of comments on other proposals because of time. Another PUC meeting was scheduled for this purpose.

The Commentary was developed and sent to the PUC for review prior to the December 9, 2008 meeting. Since the September 2008 meeting, it was discovered that there was an error in the calculations that formulated the associated maps that were approved in September. USGS prepared new maps and they were submitted to the PUC for review before the December meeting. One of the major changes adjusted the factors in the area of Salt lake City, Utah and the PUC was familiar with the impact of the change because the SDPRG had developed a 32 city comparison of design factors in a table. One of the 32 cities was Salt Lake.

Continued

SDPRG Continued

The fist issue the SDPRG presented was the maps and Mr. Nico Luco of USGS and a member of the SDPRG explained the differences. A motion was made to accept the new maps as presented and the PUC approved (18,0,0).

Then Mr. Luco explained the commentary and pointed out some additional last minute changes. These were shown and described to the PUC. During the discussion, several pointed out several issues that caused concern. Mr. Harris responded with replies to several of these concerns, but not all. The chair pointed out that time does not permit additional review and comment and suggested that if a PUC member wanted to detail his or her concern they would have a week to do so. If the proposal goes forward to the member Organization for ballot, the PUC issues would be included in the background information document (This document). Mr. Harris stated that he would be able to address several issues if given time over lunch to prepare revised text. The motion to approve the commentary was tabled until that time. FEMA representative requested an editorial change that the commentary not refer to the ATC 63 Project. It is now FEMA P695. The change will be made.

Mr. Harris proposed additional text for review. After lengthy discussion, the PUC was asked to vote on the proposed modification and they approved by a vote (19,0,0)

Stewart (YR): This proposal is generally fine, I only disagree with the use of maximum horizontal component (more on that below).
My YR could be changed to Y if average horizontal component is adopted.

> ***SDPRG Response - Non-Persuasive.*** *Defining ground motions in terms of the maximum direction was proposed by the SDPRG (Proposals SDPRG-2 and SDPRG-5) and tentatively adopted by the PUC at the November 12-13, 2007 PUC meeting contingent upon adoption of the new USGS maps (see B. Murphy's PUC meeting notes: "Motion 5 - Proposals 2 and 5 with editorial adjustments, contingent on adoption of 2007 maps (Motion 1A), and parallel charge was voted on by the PUC (19,1,0)"). PUC adoption of maximum direction was confirmed at the April 7-8, 2008 PUC meeting along with new USGS maps and related changes proposed by the SDPRG (subject to PUC approval of final edits reflected in SDPRG-1R4).*

Crouse (YR): The MCE terminology must remain in Sect. 11.2 because it is still relevant for evaluations in Sect. 11.8.3. However, it should be modified to reflect this situation if SDPRG 1R4 passes.

> ***SDPRG Response - Non-Persuasive.*** *Section 11.2 (Definitions) does not require "MCE terminology" unless such is required by other proposed changes (e.g., changes to Section 11.8 proposed by TS 3-5R2 proposal). Please note current*

> *(MCE) and proposed RTE definition applies (only) to Section 11.4 of the Provisions:*
>
> ***RISK-TARGETED (RTE)~~MAXIMUM CONSIDERED EARTHQUAKE (MCE)~~ GROUND MOTIONS:*** *The most severe earthquake effects considered by this standard as defined in Section 11.4.*
>
> *Also please note - above definition does not define ground motions in terms of deterministic, probabilistic or risk-based hazard, or any combination thereof - just the most "severe earthquake" ground motions considered by ASCE/SEI 7-05 (NEHRP Provisions).*

Bachman (No): First, it is not clear whether this revised proposal was ever voted upon formally by the SDPRG since no vote or comments were provided. This is an extremely important proposal and I believe it is paramount that the PUC be provided with the vote and comments of the SDPRG members and their comments, if any, prior to any balloting. Second, in my opinion, while the proposal has come along way it is still not consistent with what I believe was agreed upon at the previous two PUC meetings. In my opinion there were two fundamental directions given at the PUC meetings. The first was transparency and the second was that the target adjusted earthquake would be called the Risk Targeted Earthquake. Nowhere did we agree to eliminate the Maximum Considered Earthquake from the Provisions. I believe elimination of the MCE would cause confusion and mistrust by those using our seismic codes. Furthermore it is needed by those performing liquefaction studies. And retaining the MCE will provide a necessary bridge between our new ground motion determination procedure and the procedures we currently use to determining them. I know there are some who believe that:

$$\text{Transparency} \;\; \alpha \;\; \frac{1}{\text{Simplicity}}$$

However, I don’t think that needs to be case. And furthermore it’s my opinion that

$$\text{Lack of Transparency} \;\; \alpha \;\; \text{Confusion and Mistrust}$$

So while I support the general concepts of the current revised proposal, I still am not in agreement with how it is being implemented. Therefore, I have developed an alternate proposal that retains the MCE. This proposal (identified as SDPRG 1R4 – REB) is provided at the end of this comment. This proposal does not change the technical content or intent of the SDPRG – 1R4 proposal and the resulting design ground motions are identical. It does not add any new maps other than those in those specified in the SDPRG -1R4 proposal. It does add two new terms S_{SR} and S_{1R} (which could be changed to something else by the ASCE 7 SSC if folks think they would cause confusion with S_{RS} and S_{R1} or perhaps S_{RS} and S_{R1} could be changed to something like S_{MSR} and S_{M1R}. I suggest we leave it ASCE 7 SSC to deal with). Simply put my proposal defines at any site location and any period the C_R factor as:

$$C_R = \frac{\text{RTE including deterministic limits}}{\text{MCE including deterministic limits}}$$

This may necessitate a small change in the C_R maps since it would result in defining C_R = 1 where the deterministic limits govern.

I would change my vote to Yes if the following proposal were adopted instead of the proposal SDPRG 1R4. Please note that I have not changed the reason statement and there is no commentary that goes along with SDPRG 1R4. I assume that the commentary would be developed later primarily based on the reason statement. Therefore some tweaking would be required before the reason statement was sent to the BSSC membership if my alternate proposal were to be adopted. I believe what I am proposing is in order and consistent with what the PUC has agreed to in the two previous PUC meetings. I realize that if my alternate proposal is adopted it would cause some problems in the presentations at the Sept 10th BSSC mapping workshop. In my opinion, the occurrence of the mapping workshop and other schedule concerns should have no influence or bearing on the decision making associated with the resolution of comments on this proposal. It is too important.

> ***SDPRG Response - Non-Persuasive.*** *Editorial changes requested by the PUC at the April 7-8 meeting (to address terminology and transparency), incorporated into SDPRG-1R4, were not formally voted on by the SDPRG (since they did not affect technical content of SDPRG proposals). SDPRG votes on proposals (SDPRG-2/5) to use maximum direction ground motions and (SDPRG-3/6) to use 84th percentile (deterministic) ground motions were discussed at the November 12-13, 2007 PUC meeting and voted on affirmatively by the PUC (see minutes to November 12-13, 2007 PUC meeting). The SDPRG vote (December 18, 2007) on SDPRG-1R2 (revised risk targeted ground motions proposal) was 9(Y), 1(YR) Crouse, and 1 (N) Leyendecker. This ballot and resolutions of N/YR votes were discussed with the PUC at the April 7-8 PUC meeting. The PUC tentatively adopted this proposal (and all other SDPRG proposals and new USGS maps) subject to final "terminology and transparency" edits, mentioned earlier. SDPRG-1R4 simply reflects these editorial changes, including combining all SDPRG technical proposals and the new USGS maps into one proposal.*
>
> *The SDPRG was copied on draft responses to "COMMENTS AND RESPONSES ON JULY-AUGUST, 2008 PUC BALLOT PROPOSALS" for SDPRG-1R4 and requested to confirm their support of this proposal (or the REB version thereof) and draft responses to PUC comments, and their preference for inclusion of the SDPRG-1R4 proposal in Part 1 or Part 3, respectively, of the Provisions - see results of vote below:*
>
> *Do you support Proposal SDPRG-1R4?* *9 Y, 1 YR (Holmes), 0 N, 0 NV*

Or would you prefer SDPRG-1R4-REB?	*0 Y, 10 N*
Do you support including SDPRG-1R4 in Part 1 of the Provisions?	*9 Y, 1 N*
Or would you prefer SDPRG-1R4 in Part 3 of the Provisions?	*1 Y (Holmes), 0 N*

In brief, SDPRG-1R4 addresses the terminology and transparency (editorial) changes requested by the PUC at the last meeting. Consistent with current Provisions (and ASCE/SEI 7-05), SDPRG-1R4 is based on a single (if somewhat complex) definition of ground motions, proposing use of risk-targeted (1% in 50-year collapse risk) ground motions, in lieu of current 2% in 50-year hazard-based ground motions, in probabilistic regions. With the possible exception of proposed PGA ground motions (required only for liquefaction studies), hazard-based ground motions would no longer be required by the Provisions and would seem to be unnecessary baggage. Further, multiple definitions of ground motions would be (even) more confusing to most users (especially if the ground motions of one of the definitions are not used anywhere in the Provisions).

PROPOSAL SDPRG-1R4-REB (2009)

SCOPE: **Sec. 11.1 of the *2009 Provisions***
Sec. 11.2 of the *2009 Provisions*
Sec. 11.3 of the *2009 Provision*
Sec. 11.4 of the *2009 Provisions*
Sec. 11.6 of the *2009 Provisions*
Chapters 12 – 22 of the *2009 Provisions*

PROPOSAL FOR CHANGE:

In Sections of the *2009 Provisions:*

Change nomenclature in Chapter 11 as follows:

In Section 11.1.2 Item 1 change S_S to S_{SR}.

Change nomenclature and symbols in Chapters 12 – 20 and Sections 11.6, 21.1, 21.3 and 21.4 as follows wherever they appear:

Change Maximum Considered Earthquake to Risk-Targeted Earthquake

Change MCE to RTE

Change S_S to S_{SR}

Change S_1 to S_{1R}

Change S_{MS} to S_{RS}

Change S_{M1} to S_{R1}

Change S_{aM} to S_{aR}

Revise definitions Sec. 11.2 as follows:

11.2 DEFINITIONS:

MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTIONS: The most severe earthquake effects considered by this standard not adjusted for target risk as defined in Section 11.42.

RISK-TARGETED (RTE): The most severe earthquake effects considered by this standard adjusted for target risk as defined in Section 11.4.

Revise existing and add new symbols to Sec. 11.3 as follows:

11.3 NOTATION:

C_R = risk coefficient; see Section 21.2.1.1

C_{RS} = mapped value of the risk coefficient at a period of 0.2 s as defined by Figure 22-5

C_{R1} = mapped value of the risk coefficient at a period of 1 s as defined by Figure 22-6

S_S = ~~mapped~~ MCE, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.2~~11.4.1~~

S_{SD} = mapped deterministic, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1

S_{SUH} = mapped uniform-hazard, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1

S_1 = ~~mapped~~ MCE, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.2~~11.4.1~~

S_{1D} = mapped deterministic, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.1

S_{1UH} = mapped uniform-hazard, 5 percent damped, spectral response acceleration parameter a period of 1 s as defined in Section 11.4.1

S_{aR} = the site-specific RTE spectral response acceleration at any period adjusted for target risk

S_{SR} = the RTE, 5 percent damped, spectral response acceleration parameter at short periods adjusted for target risk but not adjusted for and site-class effects as defined in Section 11.4.5

S_{1R} = the RTE, 5 percent damped, spectral response acceleration parameter at a period of 1 s adjusted for target risk but not adjusted for site-class effects as defined in Section 11.4.5.

S_{RS} = the RTE, 5 percent damped, spectral response acceleration parameter at short periods adjusted for target risk and site-class effects as defined in Section 11.4.5

S_{R1} = the RTE, 5 percent damped, spectral response acceleration parameter at a period of 1 s adjusted for target risk and site-class effects as defined in Section 11.4.5.

Revise text, equations and tables of Sec. 11.4 as shown below and also revise any cross references to Section Numbers and Equations in Chapters 12-20 as follows:

Any references to Section 11.2 should be changed to 11.3
Any references to Section 11.3 should be changed to 11.5
Any references to Section 11.4 should be changed to 11.6
Any references to Section 11.5 should be changed to 11.7
Any references to Section 11.6 should be changed to 11.8
Any references to Equations 11.4-1 should be changed to 11.4-5
Any references to Equations 11.4-2 should be changed to 11.4-6
Any references to Equations 11.4-3 should be changed to 11.4-7
Any references to Equations 11.4-4 should be changed to 11.4-8

11.4 Seismic Ground Motion Values

11.4.1 Mapped Acceleration Parameters and Risk Coefficients. The parameters S_{SUH} , S_{1UH}, S_{SD} and S_{1D}~~S_S and S_1~~ shall be determined from the 0.2 and 1 s spectral response accelerations shown on Figures 22-1 through 22-4~~22-14~~, respectively, and the risk coefficients C_{RS} and C_{R1} shall be determined from Figures 22-5 and 22-6, respectively.

Move following sentence to Section 11.6 (Seismic Design Category):

Where S_{1R}, is less than or equal to 0.04 and S_{SR} is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.

11.4.2 Maximum Considered Earthquake (MCE) Parameters for Site Class B. The MCE spectral response acceleration for short periods (S_s) and at 1 sec (S_1) for Site Class B shall be determined as follows:

$S_S = S_{SUH}$ except where S_{SUH} is greater than 1.5, S_S shall be taken as the lesser of S_{SUH} and S_{SD} but not less than 1.5.

$S_1 = S_{1UH}$ except where S_{1UH} is greater than 0.6, S_1 shall be taken as the lesser of S_{1UH} and S_{1D} but not less than 0.6.

where

S_{SD} = mapped deterministic, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1

S_{SUH} = mapped uniform-hazard, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1

S_{1D} = mapped deterministic, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.1

S_{1UH} = mapped uniform-hazard, 5 percent damped, spectral response acceleration parameter a period of 1 s as defined in Section 11.4.1

11.4.~~2~~3 Site Class. Based on the site soil properties, the site shall be classified as either Site Class A, B, C, D, E or F in accordance with Section 20. Where the soil properties are not known in sufficient detail to determine the Site Class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site.

11.4.~~3~~4 Site Coefficients and Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters. The MCE spectral response acceleration for short periods (S_{MS}) and at 1 sec (S_{M1}), adjusted for Site Class effects, shall be determined by Eqs. 11.4-1 and 11.4-2, respectively.

$$S_{MS} = F_a S_s \qquad (11.4\text{-}1)$$

$$S_{M1} = F_v S_1 \qquad (11.4\text{-}2)$$

where

S_S = mapped MCE, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.~~1~~2, and

S_1 = mapped MCE, 5 percent damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.~~1~~2

where site coefficients F_a and F_v are defined in Tables 11.4-1 and 11.4-2, respectively based on the values of S_S and S_1.

11.4.5 Site Coefficients, Risk Coefficients and Adjusted Risk-Targeted Earthquake (RTE) Spectral Response Acceleration Parameters. The spectral response acceleration for short periods (S_{SR}) and at 1 sec (S_{1R}), adjusted for the target risk of collapse, shall be determined by Eqs. 11.4-3 and 11.4-4,

$$S_{SR} = C_{RS}\, S_S \qquad (11.4\text{-}3)$$

$$S_{1R} = C_{R1}\, S_1 \qquad (11.4\text{-}4)$$

where

S_S = MCE, 5 percent damped, spectral response acceleration parameter at short periods for Site Class B as defined in Section 11.4.2

C_{RS} = mapped value of the risk coefficient at a period of 0.2 s as defined by Figure 22-5

S_1 = MCE, 5 percent damped, spectral response acceleration parameter at a period of 1 s for Site Class B as defined in Section 11.4.2

C_{R1} = mapped value of the risk coefficient at a period of 1 s as defined by Figure 22-6.

The RTE spectral response acceleration for short periods (S_{RS}) and at 1 s (S_{R1}), adjusted for Site Class effects and the target risk of collapse, shall be determined by Eqs. 11.4-5 and 11.4-6, respectively.

$$S_{RS} = F_a S_{SR} \tag{11.4-5}$$

$$S_{R1} = F_v S_{1R} \tag{11.4-6}$$

where site coefficients F_a and F_v are defined in Tables 11.4-1 and 11.4-2, respectively based on values of S_{SR} or S_{R1}. Where the simplified design procedure of Section 12.14 is used, the value F_a shall be determined in accordance with Section 12.14.8.1, and the values of F_v, S_{1R}, S_{RS}~~S_{MS}~~, and S_{R1}~~S_{M1}~~ need not be determined.

Table 11.4-1 Site Coefficient, F_a

Site Class	~~Mapped Maximum Considered Earthquake~~ Spectral Response Acceleration Parameter at Short Period				
	S_S or S_{SR} ≤ 0.25	S_S or S_{SR} $= 0.5$	S_S or S_{SR} $= 0.75$	S_S or S_{SR} $= 1.0$	S_S or S_{SR} ≥ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				
Note: Use straight-line interpolation for intermediate values of S_S or S_{SR}					

Table 11.4-2 Site Coefficient, F_v

Site Class	~~Mapped Maximum Considered Earthquake~~ Spectral Response Acceleration Parameter at 1-s Period				
	S_1 or S_{1R} ≤ 0.1	S_1 or S_{1R} = 0.2	S_1 or S_{1R} = 0.3	S_1 or S_{1R} = 0.4	S_1 or S_{1R} ≥ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_1 or S_{1R}

11.4.~~4~~6 Design Spectral Acceleration Parameters. Design earthquake spectral response acceleration parameter at short periods, S_{DS}, and a 1 s period, S_{D1}, shall be determined from Eqs. 11.4-7~~Eqs. 11.4-3~~ and 11.4-8~~11.4-4~~, respectively. Where the alternate simplified design procedure of Section 12.14 is used, the value of S_{DS} shall be determined in accordance with Section 12.14.8.1, and the value of S_{D1} need not be determined:

~~$S_{DS} = \frac{2}{3} S_{MS}$ (11.4-3)~~

~~$S_{D1} = \frac{2}{3} S_{M1}$ (11.4-4)~~

$$S_{DS} = \frac{2}{3} S_{RS} \quad (11.4\text{-}7)$$

$$S_{D1} = \frac{2}{3} S_{R1} \quad (11.4\text{-}8)$$

11.4.~~5~~7 Design Response Spectrum. Where a design response spectrum is required by this standard and site-specific ground motion procedures are not used, the design response spectrum curve shall be developed as indicated in Figure 11.4-1 and as follows:

1. For periods less than T_0, the design spectral response acceleration, S_a, shall be taken as given by Eq. 11.4-9~~11.4-5~~:

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right)$$

(Eq. 11.4-9~~11.4-5~~)

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.

3. For periods greater than T_S, and less than or equal to T_L, the design spectral response acceleration, S_a, shall be taken as given by Eq. 11.4-10~~11.4-6~~:

$$S_a = \frac{S_{D1}}{T}$$

(Eq. 11.4-10~~11.4-6~~)

4. For periods greater than T_L, S_a shall be taken as given by Eq. 11.4-11~~11.4-7~~:

$$S_a = \frac{S_{D1}T_L}{T^2}$$

(Eq. 11.4-11~~11.4-7~~)

where

S_{DS} = the design spectral response acceleration parameter at short periods

S_{D1} = the design spectral response acceleration parameter at 1-s period

T = the fundamental period of the structure, s

T_0 = $0.2\dfrac{S_{D1}}{S_{DS}}$

T_S = $\dfrac{S_{D1}}{S_{DS}}$ and

T_L = long-period transition period (sec) shown in Figure 22-7.~~22-15 (Conterminous United States), Figure 22-4222-16 (Region 1), Figure 22-4322-17 (Alaska), Figure 22-4322-18 (Hawaii), Figure 22-4522-19 (Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix), and Figure 22-4622-20 (Guam and Tutuila).~~

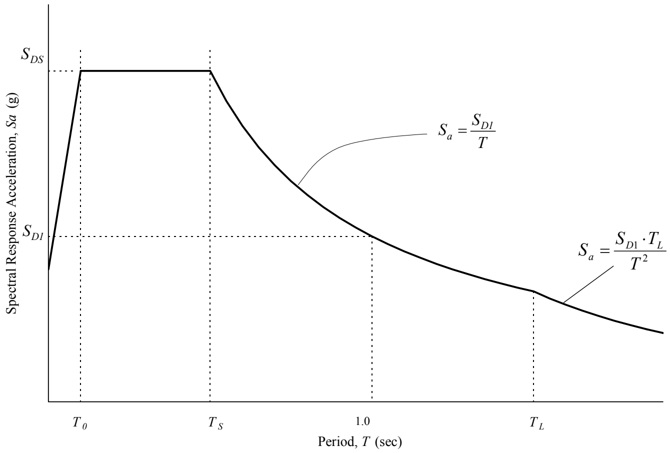


Figure 11.4- 1 Design Response Spectrum

11.4.~~6~~8 RTE~~MCE~~ Response Spectrum. Where a RTE~~MCE~~ response spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5.

11.4.~~7~~9 Site-Specific Ground Motion Procedures. The site-specific ground motion procedures set forth in Section 21 are permitted to be used to determine ground motions for any structure. A site response analysis shall be performed in accordance with Section 21.1 for structures on Site Class F sites, unless the exception to Section 20.3.1 is applicable. For seismically isolated structures and for structures with damping systems on sites with S_{1R} greater than or equal to 0.6, a ground motion hazard analysis shall be performed in accordance with Section 21.2.

Revise text of Sec. 12.14.8.1 as follows:

… In calculating S_{DS}, S_{SR} shall be in accordance with Section 11.4.5~~11.4.1~~, but need not be taken larger than 1.5. …

Revise text of Sec. 21.2 as follows:

21.2.1 Probabilistic MCE Ground Motions. The probabilistic MCE spectral response acceleration shall be taken as the spectral response acceleration in the maximum direction of ground motions represented by a 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance within a 50-yr. period.

21.2.2 Deterministic MCE Ground Motions. The deterministic MCE spectral response acceleration at each period shall be calculated as ~~150 percent of~~ the largest 84th percentile~~median~~ 5 percent damped spectral response acceleration in the maximum direction of ground motions computed at that period for characteristic earthquakes on all known active faults within the region. For the purposes of this standard, the ordinates of the deterministic ground motions response spectrum shall not be taken as lower than the corresponding ordinates of the response spectrum determined in accordance with Fig. 21.2-1, where F_a and F_v are determined using Tables 11.4-1 and 11.4-2, respectively, with the value of S_s taken as 1.5 and the value of S_1 taken as 0.6.

21.2.3 Site-Specific MCE. The site-specific MCE spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations from the probabilistic MCE of Section 21.2.1 and the deterministic MCE of Section 21.2.2.

21.2.4 Probabilistic RTE Ground Motions. The probabilistic RTE spectral response acceleration shall be taken as the spectral response acceleration in the maximum direction of ground motions represented by a 5 percent damped acceleration response spectrum that is expected to achieve a 1 percent probability of collapse within a 50-yr. period. For the purpose of this standard, ordinates of the probabilistic RTE ground-motion response spectrum shall be determined by either Method 1 of Section 21.2.4.1 or Method 2 of Section 21.2.4.2.

21.2.4.1 Method 1. Ordinates of the probabilistic RTE ground-motion response spectrum shall be determined as the product of the risk coefficient at each spectral response period, C_R, and the probabilistic MCE spectral accelerations as determined by Section 21.2.1. The value of the risk coefficient, C_R, shall be determined using values of C_{RS} and C_{R1} from Figs. 22-5 and 22-6, respectively. At spectral response periods less than or equal to 0.2 second, C_R shall be taken as equal to C_{RS}. At spectral response periods greater than or equal to 1.0 second, C_R shall be taken as equal to C_{R1}. At response spectral periods greater than 0.2 second and less than 1.0 second, C_R shall be based on linear interpolation of C_{RS} and C_{R1}.

21.2.4.2 Method 2. Ordinates of the probabilistic RTE ground-motion response spectrum shall be determined from, at each spectral response period, iterative integration of a site-specific hazard curve with a lognormal probability density functions representing the derivative of the collapse fragility (i.e., probability of collapse as a function of spectral response acceleration. At each period, the ordinate of the probabilistic ground-motion response spectrum shall achieve a 1 percent probability of collapse within a 50-yr. period for a collapse fragility having (i) a 10 percent probability of collapse at said ordinate of the probabilistic ground-motion response spectrum and (ii) a logarithmic standard deviation value of 0.8.

21.2.5 Site-Specific RTE. The site-specific RTE spectral response acceleration at any period, S_{aR}, shall be taken as the lesser of the spectral response accelerations from the probabilistic RTE ground motions of Section 21.2.4 and the deterministic MCE ground motions of Section 21.2.2.

Chapter 22 - Revise title, text and maps (figures) as follows:

Chapter 22 SEISMIC GROUND MOTION,~~ AND~~ LONG-PERIOD TRANSITION AND RISK COEFFICIENT MAPS

Contained in this chapter are Figs 22-1 through 22-7~~22-20~~, which provide the mapped uniform-hazard ground motion parameters, S_{SUH} and S_{1UH}, the mapped deterministic ground motion parameters, S_{SD} and S_{1D}, the mapped risk coefficients, C_{RS} and C_{R1}, and mapped long-period transition period, T_L, ~~Maximum Considered Earthquake (MCE) Ground Motion parameters S_S and S_1~~ for use in applying the seismic provisions of this standard. ~~S_S is the mapped MCE ground motion, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1. S_1 is the mapped~~

~~MCE ground motion, 5 percent damped spectral response acceleration parameter at a period of 1 sec as defined in Section 11.4.1.~~

These maps were developed by the United States Geological Survey (USGS) and have been updated for the 2009~~2005~~ edition. Maps for Guam and Tutuila (American Samoa) are not included because uniform-hazard ground motion parameters, deterministic ground motion parameters, and risk coefficients have not yet been developed for those islands. Therefore, similar to the 2005 edition of this standard, the parameters S_{SR} and S_{1R} defined in Section 11.4.5 shall be, respectively, 1.5 and 0.6 for Guam and 1.0 and 0.4 for Tutuila.

The following is a list of figures contained in this chapter.

Replace current list of maps with the following:

Figure 22-1 Uniform-Hazard (2% in 50-Year) Ground Motions of 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B

Figure 22-2 Uniform-Hazard (2% in 50-Year) Ground Motions of 1 s Spectral Response Acceleration (5% of Critical Damping), Site Class B

Figure 22-3 Deterministic Ground Motions of 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B

Figure 22-4 Deterministic Ground Motions of 1 s Spectral Response Acceleration (5% of Critical Damping), Site Class B

Figure 22-5 Risk Coefficient at 0.2 s spectral response period

Figure 22-6 Risk Coefficient at 1 s spectral response period

Figure 22-7 Long-Period Transition Period, T_L (s)

There are no changes made to the maps that are provided in Proposal SDPRG 1R4 so for brevity they have not be repeated here.

REASON FOR PROPOSAL:

No modifications have been made to the reason statement since this was written by the chair of the SDPRG and it would be presumptive of me to do so. If my alternate proposal is adopted PUC, some changes would need to be done before submitting to the BSSC membership although the changes would be relatively minor and could be done in reasonable period of time that would satisfy BSSC schedule goals.

Ghosh (No): I am still trying to grasp the full meaning of all the proposed changes to the ground motion maps. It has not been easy. My understanding, and it may very well be flawed, is that four significant changes are proposed. 1) USGS has updated some source zone models, have used Next Generation Alternation (NGA) relationships, to the exclusion of the old attenuation relationships, in the western United States and have used new attenuation relationships in addition to the old relationships in the central and eastern United States. The new relationships apparently show that eastern earthquakes are much more like western earthquakes than we thought earlier, with ground motion intensity dropping off more steeply with distance from the source than indicated by earlier attenuation curves. As a result of all of this, ground motion – particularly long-period ground motion has decreased significantly (by 50% or more) in many parts of the United States. 2) Uniform-hazard ground motion has now been replaced by risk-targeted ground motion. This switch from a 2% in 50-year hazard level to a 1% in 50-year collapse risk target has resulted in up to 30% decreases in ground motion in high-hazard areas of the central and eastern United States and in coastal Oregon. 3) A switch has been made from "geo-mean" ground motions to maximum direction ground motions. This has resulted in increases in short-period ground motion by a factor of 1.1 and in long-period ground motion by a factor of 1.3. 4) Finally, deterministic ground motions have been changed from 150% of median ground motions to 84th percentile ground motions, which are 180% of median ground motions.

The net result of these four major changes has been that short-period ground motions have gone down rather substantially in the central and eastern states; elsewhere, status quo has by and large been maintained. A skeptical and totally uncharitable view of all these changes might be that a politically expedient decrease of short-period ground motion in parts of the central and eastern United States has been accomplished through impressively fancy technical maneuvering. After many hours of careful listening, some hours of reading, and asking many questions and receiving answers that were mostly forthright, this is not a view I share at this point of time. At the same time, I wonder why put the design profession through all the changes if at the very end they make so little difference. Seismic design of the 2003 NEHRP Provisions is already way more sophisticated than seismic design practiced anywhere else other than Canada. Is all the proposed added sophistication really necessary? I think what SDPRG has done is extremely valuable and commendable. I think this is all very interesting and sheds light on a lot of important aspects of seismic design. I think this is ideal material for inclusion in Part 3 of the NEHRP Provisions. Unless discussion in San Francisco and the Workshop there convince me otherwise, I am at this point opposed to including the changes in Part 1 of the NEHRP Provisions, which would mean automatic consideration for inclusion in ASCE 7-2010.

I understand that a project similar to NGA is now getting started for the central and eastern United States. Let's wait and see what that brings. To make these sweeping changes now, resulting in significant decreases in short-period ground motions for the central and eastern United States, and then making further changes a short few years later as a result of the new study will be a disservice to the design profession and to the public at large.

SDPRG Response - Non-Persuasive. *The Reason for Proposal documents the technical basis for the three topics of proposed change to ground motions (i.e., maximum direction, 84th percentile and risk-targeted ground motions, respectively). They are technically sound proposals (even if some might find them "politically expedient").*

The complexity of the ground motions has not changed appreciably from the previous cycle (still very complex). The addition of "transparency" formulas in Section 11.4 (as requested by the PUC) makes the user more aware of this complexity, than does the current Provisions. Fortunately, Code requirements are moving out of the dark ages, and web-based software (e.g., USGS design ground motions tool) will make implementation relatively simple for the design profession.

If the PUC, et al. does not adopt SDPRG-1R4, then improvements in our understanding of the earth science will not make it into this cycle of the Provisions (ASCE/SEI 7-05). There will always new research in the future (e.g., studies of CEUS ground motions, new earthquake data, etc.). If we wait for research to be complete, we would never update ground motion criteria.

Holmes(YR): My vote would change to Y if the proposal is moved to Part 3. In my opinion, placement in Part 3 does not automatically mean that the proposal or an improved similar one cannot be adopted by ASCE 7. However, approval of the proposal as-is implies complete confidence in all aspects of the current proposal. I think the following issues should be considered before I could give full endorsement:

1. Consideration of the epsilon factor on spectral shape in the near field.
2. As indicated in comments on Proposal 2-5RA, unless I missed a proposal, Chapter 17 still uses the scaling factor of 1.3 and Chapter 18 references Chapter 17 for Ground Motion Histories. These should be made consistent with Proposal 2-8 Rev2.
3. Although NGA is typically smaller than previous attenuation at 1 sec, values of Sa using NGA at larger periods are considerably larger than S1/T. Is this an issue for buildings with periods greater than 1 sec but that are not controlled by minimum base shear?
4. There was previously discussion of "vetting" these proposals at workshops around the country. The only one I know of is being held on Sept 10, after these votes must be submitted.

SDPRG Proposal - Non-Persuasive: *Responses to specific issues follow:*

Issue 1 - Non-Persuasive. The SDPRG considered (Task 3 - Harris) but did not propose changing provisions related to spectral shape. Changes proposed by SDPRG-1R4 are independent of spectral shape (epsilon factor).

Issue 2 - Non-Persuasive. Valid concern, however, not part of SDPRG-1R4. Other proposals (by TS-2) should be revised to address record scaling (to the *maximum direction) in consistent manner in all chapters/sections of the Provisions.*

Issue 3 - Non-Persuasive. The SDPRG considered (Task 3 - Harris) but did not propose changing provisions related to spectral shape. Changes proposed by SDPRG-1R4 are independent of spectral shape (at periods beyond 1 second).

Issue 4 - Non-Persuasive. Technical changes (maximum direction, 84th percentile, and risk-targeted ground motions) were developed by the SDPRG over a year ago and presented to the PUC in 2007, but required integration with the new USGS hazard maps (to fully understand the impact of proposed changes). The USGS (Nico Luco) kept the SDPRG, and the PUC, well informed of USGS progress, but workshops could not be held sooner than finalization of the new USGS maps (mid-2008). Note. The USGS has held several workshops related to their updating of hazard maps.

Hamburger (YR): On page 6, line 17, It appears that reference to Figure 22-7 should be to Figure 11.4-1. In Section 21.2.1.2 – there is a typo: (line 39 – functions should be function). In the same Section 21.2.1.2 I find the terminology “derivative of the collapse fragility” to be confusing. I recommend deleting the words “derivative of” and saying simply: “iterative integration of a site-specific hazard curve with a lognormal probability density function~~s~~ representing the ~~derivative of the~~ collapse fragility (i.e., probability of collapse as a function of spectral response acceleration.”

SDPRG Response - Partially Persuasive (edits).

Page 6, Line 17. Non-Persuasive edit. Figure 22-7 is the correct reference.

Section 21.2.1.2. Line 39 - Persuasive edit, drop the "s" from functions, as shown below:

Section 21.2.1.2. Persuasive edit, change proposed terminology, as shown below:

21.2.1.2 Method 2. Ordinates of the probabilistic ground-motion response spectrum shall be determined from, at each spectral response period, iterative integration of a site-specific hazard curve with a lognormal probability density function~~s~~ representing the ~~derivative of the~~ collapse fragility (i.e., probability of collapse as a function of spectral response acceleration). At each period, the ordinate of the probabilistic ground-motion response spectrum shall achieve a 1 percent probability of collapse within a 50-yr. period for a collapse fragility having (i) a 10 percent probability of collapse at said ordinate of the probabilistic ground-motion response spectrum and (ii) a logarithmic standard deviation value of 0.8.

Gillengerten (No): I think this proposal should be split, the first proposal covering a change to Risk-Targeted Earthquake, and the other covering incorporation of the NGA into the Provisions.

While I support the concept of Risk-Targeted Ground Motions, I believe it should be placed in Part 3, for the following reasons:

1. The Provisions are intended to guard against structural collapse, but up to now, we have not attempted to quantify compliance with the Provisions in terms of collapse. While a 1% in 50 year probability of collapse might be a good target, it is unlikely that we are achieving this using the current design procedures and coefficients. For the majority of the structural systems listed in the Provisions, the probability of collapse is either undefined or defined for a limited subset of structures you <u>could</u> design using the Provisions. For example, there are a number of structural irregularities listed in the design procedures, which if present, trigger various modifications to the design approach. At this time we have not quantified whether the modifications we impose are sufficient to keep the Probability of Collapse at or near the target level. To bring this concept forward into Part 1 at this time, without a concurrent effort to modify the design coefficients and procedures will create inconsistency (or, if you prefer, intensify existing inconsistency) and diminish the validity of the Provisions.
2. It is unclear how the proposed ground motions relate to seismic design issues other than collapse of the structure – liquefaction and nonstructural component design, for example.

In Part 3, the Risk-Targeted ground motions can be introduced to the engineering community, and allow them the opportunity to revise their design procedures. This effort is well underway for primary structural systems, but considerable work is needed for nonstructural components and systems.

I would change my "No" vote to "Yes" if Risk-Targeted ground motions was moved to Part 3.

> ***SDPRG Response - Non-Persuasive.*** *Separation of proposed risk-targeted ground motions from other proposed changes is not feasible (at this time) and contrary to the direction given by the PUC to roll proposed changes into 1 package.*

Proposal 2-8 Rev2 (Y=12, YR=4, N=5, NV=1 --76%)

The Stewart comment was first discussed and TS 2 found it nonpersuasive. The PUC agreed by a vote (19,0,2). The Crouse comment was discussed and Crouse detailed a major concern. TS 2 attempted to find Crouse nonpersuasive and the PUC did not support TS 2 by a vote (9,8,6). In attempting to find an alternative solution, TS 2 offered that the Crouse and Hamburger comments could be resolved if TS 2 attempted to assemble an ad-hoc committee to meet over lunch to form an alternative resolution. TS 2 took their initial responses and modified them to include several comments provided by the PUC members. TS 2 returned with the responses provided below. The PUC found the Hamburger comment persuasive (20,0,1) and the Crouse comment persuasive (23,0,0).
The remaining comments were resolved either by editorial adjustments (Wood) or the PUC member withdrew (Bachman, Ghosh, Holmes, Gillengerten, and Klingner) their respective comment. This proposal, as modified, is approved for voting by the Member Organizations.

Stewart (No): Reason for N vote on 2-8 Rev2 and 2-8C: As explained in my March ballot, I disagree with the stated objective to change the basis for design from an average horizontal component to the strongest direction of shaking. The strongest direction of shaking is highly unlikely to occur in the most critical direction for a given structure, and assuming this worst-case situation as the basis for all seismic design introduces an unnecessary level of conservatism. We should keep our long-standing practice of using the average horizontal component. No evidence has been offered to suggest that this is leading to unsafe seismic designs of buildings. Certainly a much stronger argument is needed to support such a fundamental modification of the seismic design philosophy for buildings.

> ***TS-2 Response: Nonpersuasive.***
>
> *The use of maximum direction of ground motion has already been approved by the PUC.*

Crouse (No): When both horizontal time-history components are input to the analysis, then by definition it is guaranteed that the structure will experience the maximum input shaking. It may not be in one of the principal directions of the building, but if the time history is representative of a near field condition within 3 km of causative fault, then the direction of maximum shaking for long period motions is the fault normal direction. At distances greater than 3 km, the direction of maximum shaking appears to be random. In that case, if the concern is that the maximum component direction may not align with one of the principal building axes, then this problem can be solved by determining this direction for each selected time history at period T. Whether this is necessary is debatable because the maximum direction shaking is determined for a linear oscillator and thus applying it to either principal axis would not necessarily yield the greatest response to a 3-D nonlinear model of the building.

> ***TS-2 Response: Persuasive.***
> *Text will be added to the end of the section as indicated in the response to Hamburger.*

For a 3-D time history analysis, a better approach to scale input time histories would be to remove the 1.1 and 1.3 scale factors from the target spectrum and then scale the time histories (rotated to the maximum shaking direction if deemed necessary) to the reduced spectrum using the usual SRSS method. A check whether the structural dynamic response is sensitive to the rotation could be made by rotating a few of the time histories by say 45° to check whether larger responses are obtained.

An additional argument supporting the approach is as follows. The 1.1 and 1.3 factors are the factors to convert, respectively, the short and intermediate period motions from the geomean value of the two components (definition of ground motion parameter for the MCE) to the maximum value, obtained by component rotation. The representative accelerograms selected for the particular site-specific application will define these factors adequately when seven or more accelerograms are selected, which is newly proposed requirement for RHA. Thus, there is no need to use factors based on statistical analysis of a large database of records, the bulk of which would not be representative for a given site.

> ***TS-2 Response: Partially Persuasive.***
>
> *The idea of rotating each of the ground motion pairs to the maximum direction (and not use the 1.1 and 1.3 factors) was debated at TS-2. One of the challenges of this approach was selecting the appropriate period. Another challenge was the introduction of a new method that would require significant guidance to properly implement. As a compromise, the scaling approach is being simplified as shown in the response to Hamburger, below.*

Wood (YR): This proposal refers to the maximum considered earthquake. Are changes required if the Project 07 recommendations are approved? Will the proposal refer to the risk targeted earthquake?

> ***TS-2 Response: Persuasive.***
>
> *This proposal is an integral part of the Project '07 recommendation and its proposal SDPRG-1R4. The MCE terminology will be editorially modified as specified in SDPRG-1R4 if the proposal passes.*

Bachman (No): This proposal presumes that SDPRG 1R4 will be approved. This is not a done deal. I will change my vote to Y if a version of the SDPRG 1R4 is approved that contains the ground motion maximum direction requirement.

> ***TS-2 Response: Nonpersuasive.***
>
> *See response to Stewart.*

Ghosh (No): This proposal is almost a supplement to Proposal SDPRG 1R4. Belongs in Part 3 of the *Provisions*.

> ***TS-2 Response: Nonpersuasive.***
>
> *This proposal is an integral part of the Proposal SDPRG-1R4. Since SDRPG-1R4 proposes the use of spectral response in the maximum direction of ground motions, then the associated scaling of time histories for use in response history analysis needs to account for this in Part 1 of the Provisions.*

Holmes (No): My vote may change to Y if I understood this. I do not have the Maffei Hashemi study, but probably wouldn't have time to study it if I did. The finding that the SRSS of the components is 1.15 times the max direction is unclear to me. SRSS of what components? As recorded? Rotated somehow? Why is the SRSS bigger in any case? Is the Max direction not determined from the SRSS of rotated pairs? Use of the max direction is an arguable design philosophy to begin with, and now we add another 15%?

> ***TS-2 Response: Partially Persuasive.***
>
> *See response to Hamburger, below.*

Hamburger (No): This is an extremely important companion to the SDPG proposal and I strongly support its concept but deplore its implementation. I believe that the scaling requiring that over the range of periods, the ratio of the average SRSS spectra to the design spectrum "average to 1.15" and not be less than 1.05 at any period is needlessly complex, too confusing and implies accuracy that is just not warranted to an asinine level. I will change my vote to support if this is reworded to more simply require either that the average of the SRSS spectrum not fall below 1.0 times the design spectrum or 1.1 times the design spectrum, with a strong preference for the former.

> ***TS-2 Response: Persuasive.***
>
> *Hamburger makes some valid points. In order to simplify the application of the scaling approach, without sacrificing accuracy, Section 16.1.3.2 will be rewritten as indicated below:*

16.1.3.2 Three-Dimensional Analysis. *Where ~~3-D~~ three-dimensional ~~analysis is~~ analyses are performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distances, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs ~~are~~is not available, appropriate simulated ground motion pairs ~~shall~~ are permitted to be used to make up the total number required. 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 (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between 0.2T and 1.5T, the average of the SRSS spectra from all horizontal component pairs does not fall below~~1.3 times and~~ the corresponding ordinate of the MCE ~~design~~ response spectrum, determined in accordance with Section 11.4.5 or ~~21.2~~11.4.7.~~, is not less than 1.0., by more than 10 percent.~~*

At sites within 5 km of an active fault that controls the hazard, each pair of components shall be rotated to the fault-normal and fault-parallel direction of the causative fault, and shall be scaled so that the average of the fault-normal components is not less than the MCE response spectrum for each period between 0.2T and 1.5T.

Gillengerten (YR): It should be clarified that implementation of this proposal is contingent on the passage of SPDRG 1R4.

TS-2 Response: Persuasive.

See response to Stewart.

Klingner (YR): I believe that the unpublished study by Maffei and Hashemi does corroborate the statement given. I believe that in general data alleged to support a proposal should be presented, so that voters can judge for themselves. I believe that the chances of a successful member organization ballot will be enhanced by inclusion of such a summary in the rationale.

Proposal 2-8C (Y=14, YR=1, N=5, NV=2 --75%)

This proposal is a commentary for Proposal 2-8Rev3 which passed the PUC in September 8-9, 2008 PUC meeting. However, there were several substantive changes to the first proposal. TS2 did not have time to modify this proposal to correctly affect the changes. TS 2 met to modify this proposal to complement 2-8R3 and to incorporate the persuasive comments from the PUC mail ballot. TS 2 presented the proposal and explained the changes. The PUC approved this proposal (18,0,1). Since this is commentary to Proposal 2-8R3 it will be combined with it as one proposal for the member ballot.

Stewart (No): Reason for N vote on 2-8 Rev2 and 2-8C: As explained in my March ballot, I disagree with the stated objective to change the basis for design from an average horizontal component to the strongest direction of shaking. The strongest direction of shaking is highly unlikely to occur in the most critical direction for a given structure, and assuming this worst-case situation as the basis for all seismic design introduces an unnecessary level of conservatism. We should keep our long-standing practice of using the average horizontal component. No evidence has been offered to suggest that this is leading to unsafe seismic designs of buildings. Certainly a much stronger argument is needed to support such a fundamental modification of the seismic design philosophy for buildings.

TS-2 Response: Nonpersuasive.

The use of maximum direction of ground motion has already been approved by the PUC.

Bachman (No): This proposal presumes that SDPRG 1R4 will be approved. This is not a done deal. I will change my vote to Y if a version of the SDPRG 1R4 is approved that contains the ground motion maximum direction requirement.

TS-2 Response: Nonpersuasive.

See response to Stewart.

Ghosh (No): This proposal is almost a supplement to Proposal SDPRG 1R4. Belongs in Part 3 of the *Provisions*.

TS-2 Response: Nonpersuasive.

This proposal is an integral part of the Proposal SDPRG-1R4. Since SDRPG-1R4 *proposes the use of spectral response in the maximum direction of ground motions, then the associated scaling of time histories for use in response history analysis needs to account for this in Part 1 of the Provisions.*

Holmes (No): My vote may change to Y if I understood this. I do not have the Maffei Hashemi study, but probably wouldn't have time to study it if I did. The finding that the SRSS of the components is 1.15 times the max direction is unclear to me. SRSS of what components? As recorded? Rotated somehow? Why is the SRSS bigger in any case? Is the Max direction not determined from the SRSS of rotated pairs? Use of the max direction is an arguable design philosophy to begin with, and now we add another 15%?

TS-2 Response: Partially Persuasive.

See response to Hamburger, below.

Hamburger (No): My negative on this commentary is in parallel with my negative on 2-8R. I will withdraw this negative if my negative to 2.8R is successfully resolved.

TS-2 Response: Persuasive.

Hamburger makes some valid points. Proposal 2-8 was modified to accordingly. The commentary section requires modification as well. The commentary will be rewritten as indicated below:

Add Commentary for Section 16.1.3.2 as follows:

One key change to the ground motion design requirements developed by Project '07's Seismic Design Procedure Reassessment Group is the use of use of maximum direction ground motions. In addition to changing the design values defined in Chapter 11 and used throughout the Provisions, implementing maximum direction ground motions affects the previous ground motion scaling rules specified in Section 16.1.3.2. Studies (Maffei and Hashemi) of 50 ground motions of M6.5-M7.9, for both far-field and near-field records and for periods in the range of 0.1-3.0 seconds, indicate that the maximum direction of ground motion is slightly less than the SRSS of the two components, with the SRSS spectrum tending to be approximately 1.15 times the maximum direction spectrum. The modified scaling new requirements simplifies phrasing of existing language by replacing 10% less than 1.15 times the MCE response spectrum with 1.0 times the MCE response spectrum. The 1.0 comes from (0.9)(1.15) ≈ 1.0.

REFERENCES

Maffei, J., and Hashemi, A., (2008), Personal Communication.

REASON FOR PROPOSAL:

Commentary is needed to Proposal 2-8, which was passed by the PUC on April 8, 2008.

Klingner (YR): I am willing to believe that the unpublished study by Maffei and Hashemi does corroborate the statement given. I believe that in general data alleged to support a proposal should be presented, so that voters can judge for themselves. I believe that the chances of a successful member organization ballot will be enhanced by inclusion of such a summary in the rationale.

TS-2 Response: Persuasive.

The reason statement will be updated to include some of the findings of the study.

PART 2

Proposal 2-111 (Y=13, YR=4, N=3, NV=2 --85%)

TS2 took the lead on responding to the comments from several TSs. The TS positions listed below represent those positions of the PUC at the September 9, 2008 meeting. Most of the comments were editorial and did not require a vote. These changes will be made. Where the PUC shows that text will be added by TS 3, these adjustments will be made. The PUC will review before the next meeting scheduled for December, 2008. The second Bachman comment was found persuasive and the PUC approved by a vote of (19,0,1). Gillengerten's first comment was found nonpesuasive (17,0,1) and his second comment was found nonpersuasive (16,0,0). Most nonpersuasive positions were withdrawn since the PUC believes it will have another chance to review.

December 9, 2008 – The PUC was provided the entire modified chapter for review prior to the meeting. TS 2 has incorporated all persuasive comments and edits from last September. TS 3 prepared revised text to Sec. C11.4.5 in response to the Wood and Ghosh earlier comments and this was reviewed. A motion was made to accept TS 3 response and it was accepted (19,0,0).

Crouse (YR): Some sections will need revision if Proposal SDPRG 1-R4 and/or Proposal 3-5 Rev 2 pass.

Withdrawn

Wood (YR):
p. 5, line 8 – The constant acceleration and constant velocity portions of the response spectra are not discussed.

TS-2 Response: Persuasive.

The text will be added by TS 3.

p. 6, Figure C11.5-1 – The font in the figure needs to be revised to avoid overlap.

TS-2 Response: Editorial.

The figure will be revised accordingly.

Bachman (No): The vote of TS-2 on this proposal should be provided to the PUC along with any comments if not resolved and the TS-2 Response.

I have the following specific comments regarding this proposal.

1. In Section C11.5.1, I would request the following change to the second sentence.

…….rather than to prevent collapse in ~~larger~~ ground motions larger than the MCE.

Withdrawn

2. In Section 11.6 on Page 8 of 12 of the proposal, in lines 5-15 justification is provided on how the lower limit of Seismic Design Category D was established. My interpretation of this section implies that the Seismic Design Category assignments were established in a manner that would make them consistent with how the UBC Seismic Zones were assigned. However this definitely was not the case. UBC Seismic Zones did not include the effects of site amplification in their establishment while the Seismic Design Categories assignment did include site amplification. It seems sometime in the development of the Seismic Design Category assignment tables, it was decided that the effects of site amplification were to be included. I believe a thorough discussion should be added to the commentary explaining why that decision was made (rather than basing on Site Class B) and how the specific cutoff values were determined (considering site amplification effects) between SDC C and D. The commentary should also recognize in its discussion the problems in practice that this decision has caused in jurisdictions that straddle the C and D boundary. The cliff in practice requirements between Seismic Design Category C and D is huge and therefore how these specific decisions were made need to be clearly explained in the commentary.

TS-2 Response: Persuasive.

The following language, originally developed for the commentary to the 1997 NEHRP Provisions, will be added:

"The earlier editions of the Provisions utilized the peak velocity-related acceleration, Av, to determine a building's Seismic Performance Category. However, this coefficient doe not adequately represent the damage potential of earthquakes on sites with soil conditions other than rock. Consequently, the 1997 Provisions adopted the use of response spectral acceleration parameters S_{DS} *and* S_{D1}*, which include site soil effects for this purpose."*

The following language, originally developed for the commentary to the 1997 NEHRP Provisions, will be added:

"Local or regional jurisdictions enforcing building regulations need to consider the effect of the maps, typical soil conditions, and Seismic Design Categories on

the practices in their jurisdictional areas. For reasons of uniformity of practice or reduction of potential errors, adopting ordinances could stipulate particular values of ground motion, particular Site Classes, or particular Seismic Design Categories for all or part of the area of their jurisdiction. For example:

1. *An area with an historical practice of high seismic zone detailing might mandate a minimum Seismic Design Category of D regardless of ground motion or Site Class.*
2. *A jurisdiction with low variation in ground motion across the area might stipulate particular values of the ground motion rather than requiring use of the maps.*
3. *An area with unusual soils might require use of a particular Site Class unless a geotechnical investigation proves a better Site Class.*

3. In Section 11.8.3, I would request that the following statement be added to the end of the first paragraph "It is reasonable for retaining wall forces to be divided by an R_p factor such as 1.5 to account for overstrength of the retaining wall".

Withdrawn

I would change my vote to Y is the above 3 changes were made as requested and the vote of TS-2 on this proposal were provided along with any unresolved comments and the TS-2 Response.

TS-2 Response: Noted.

The commentary section was balloted and all TS-2 comments were incorporated.

Ghosh (No):

Page 2, Line 8 – Suggest changing "Loadings" to "Loads"

TS-2 Response: Editorial.

The change will be made.

Page 2, Line 26 – Why is there no mention of detached one- and two-family dwellings located were $S_S < 0.4g$?

TS-2 Response: Editorially Persuasive.

The following change is will be made:

"1. detached one- and two-family dwellings in Seismic Design Categories A, B, and C, along with those located where $S_s < 0.4g$, are exempt because they represent low seismic risks."

Page 3, Line 26 – I find the paragraph starting on this line to be too cryptic. I am not even sure the first line is accurate. Isn't it a uniform margin against collapse in the maximum considered earthquake? The paragraph is trying to explain a very fundamental aspect of seismic design by ASCE 7-05. More elaboration, I believe, is needed.

Withdrawn

Page 3, Line 44 – I do not understand "The probabilistic motions are greater if these major active faults produce characteristic earthquakes every few hundred years." Is this sentence needed?

Withdrawn

Page 4, Line 12 – "Effective" means exactly what?

Withdrawn

Page 5, Line 8 – How can the commentary be approved with this portion still pending?

TS-2 Response: Persuasive.

The text will be added by TS 3.

Page 5, Line 16 – The sentence starting in the middle of this line is not understandable to me.

Withdrawn

Page 5, Line 19 – There are one or more things obviously not right with this sentence. The easiest fixes would be to add "than" after "confidence" on Line 21 and delete "from" after "using" on Line 21.

TS-2 Response: Editorially Persuasive.

The suggested change will be made.

Page 5, Line 31 – "The consequence … it contains." is not a sentence.

TS-2 Response: Editorially Persuasive.

The following change is will be made:

> *"The consequence to a community of structural damage or failure is unequally distributed across the various structures it contains."*

Page 6, Line 12 – Isn't there more accurate than two – occupancy category, seismic hazard at the site, and soils at the site? Also, is "ingredients" the best word? Components? Elements?

> ***TS-2 Response: Editorially Persuasive.***
>
> *While the site class clearly has an effect on the resulting seismic design category, the table, itself, has two components: Occupancy Category and Response Acceleration Parameter.*
>
> *The following change is will be made:*
>
> *The occupancy category is then used as one of two ~~ingredients~~ components ~~of~~ in determining the seismic design category…*

Page 6, Line 16 – The symbols described in this paragraph do not match those on the figure.

> ***TS-2 Response: Editorially Persuasive.***
>
> *The text and/or figure will be modified to make the two consistent.*

Page 7, Line 14 – Suggest replacing "thereto" with more common language.

> ***TS-2 Response: Editorially Persuasive.***
>
> *The following change is will be made:*
>
> *"Inelastic strain damages a structure, so for a given strength demand, reducing the effective R factor (by mean of the importance factor) increases the required yield strength, thus reducing ductility demand and related damage ~~related thereto~~."*

Page 7, Line 33 – "to group these step functions" means what?

> ***Withdrawn***

Page 7, Line 37 – Suggest replacing "a function" with "one of the functions." It will make the sentence clearer.

> ***TS-2 Response: Editorially Persuasive.***

The proposed change will be made.

Page 7, Line 43 – I found the explanation starting on this line and ending on Line 41 of Page 8 to be rather unclear. I had trouble fully understanding it. I do not understand MMI of "various shaking spectra." There is an editorial note, rather than actual references. How can the Commentary be approved in this form? The sentence starting on Page 8, Line 8 is not at all clear to me. The sentence starting on Page 8, Line 32 is also far from clear to me.

TS-2 Response: Nonpersuasive (except for the editorial note).

It is difficult to make changes based on the above comments without specific suggestions.

Page 9, Line 20 – I think the first sentence of the paragraph starting on this line is misleading, if not inaccurate. I find the whole paragraph to be totally dispensable. It does not tell me anything that I want to or need to know.

TS-2 Response: Nonpersuasive.

The paragraph will be helpful to engineers working in one location to understand that their design and inspection processes will be consistent from project-to-project, regardless of structural type. This paragraph will also assist the building official in the development of their review processes.

Page 10, Line 16 - I think it is large plan aspect ratio, rather than large horizontal dimension.

TS-2 Response: Editorially Persuasive.

The proposed change will be made.

Page 10, Line 17 – Suggest changing "will control" to "may control."

TS-2 Response: Editorially Persuasive.

The proposed change will be made.

Hamburger (YR): Overall, this commentary is excellent. I do have one comment: Section C11.5, first paragraph, second sentence: "The consequence to a community of structural damage or failure unequally distributed across the various structures it contains" appears to be incomplete and makes no sense.

TS-2 Response: Editorially Persuasive.

See response to Ghosh.

Gillengerten (No): Section C11.4. Given the statement of intent to provide a uniform margin against collapse, the commentary needs to discuss how well we believe the Provisions are achieving this. At a minimum, it would be appropriate to reference papers such as Krawinkler and Zazriean, "Prediction of Collapse – How realistic and practical is it, and what we can learn from it", to inform the reader of real limitations that we face when trying to achieve this objective.

TS-2 Response: Nonpersuasive.

The text C11.4 is for the selection of the appropriate ground motion values to achieve this goal. A discussion regarding achieving the intent of a uniform margin against collapse would be out of context.

C11.5.1. This section should mention there is also evidence that increasing the strength of longer period structures can significantly lower the probability of collapse.

TS-2 Response: Nonpersuasive.

The notion of increase the strength of longer period structures to lower the probability of collapse is included in the commentary to Proposal 2-9, which adopted ASCE 7-05 Including Supplement No. 2.

Klingner (YR): This is generally good. Some parts of it will need to be updated if SDPRG 1R4 passes.

TS-2 Response: Non persuasive.

This commentary is developed for Part 2 of the Provisions and is not affected by Part 1 Proposal SDPRG 1R4.

Proposal 2-112 (Y=13, YR=3, N=1, NV=5 --94%)

TS2 addressed each comment and the responses by TS2 are supported by the PUC as listed below. Most were editorial and the chapter has been revised.
The 17th Hamburger comment was found to be technical and persuasive. The PUC approved (15,0,2). This proposal is approved, as modified, to be balloted by the Member Organizations

Hanson (Commenting)
Page 2 of 76, line 17 – insert "and" so it reads "…both ground motion and structural"

TS-2 Response: Editorial.

The change will be made.

Page 2 of 76, line 25 – delete "time" so it reads "…the total ~~time~~ history of response …"

TS-2 Response: Editorial.

The change will be made.

Page 4 of 76, line 5 – "13.??" Have someone from Chapter 13 define "??" or delete this

TS-2 Response: Editorial.

The appropriate section number will be given.

Page 4 of 76, line 43 – the resistance factor symbol did not reproduce in the printed text

TS-2 Response: Editorial.

The change will be made.

Page 11 of 76, lines 45-47 – this would read better by moving the sentence in line 45 to the end of the paragraph: "… values are used. ~~The more restrictive of the system limitations governs.~~ Depending upon the combinations selected, it is possible that one of the two systems will limit the extent of the overall system with regard to use and height. The more restrictive of the system limitations governs. "

TS-2 Response: Editorial.

The change will be made.

Page 19 of 76, Figure C12.3-1 – delete "Figure 4.3-3 Building plan irregularities" from the figure

TS-2 Response: Editorial.

The change will be made.

Page 20 of 76, Figure C12.3-2 – delete "Figure C4.3-4 Building elevation irregularities" from the figure

TS-2 Response: Editorial.

The change will be made.

Page 34 of 76, lines 12 and 13 – revise from "… short period region ~~to account for~~ ignores more inelastic behavior ~~introduces errors in the~~ …" to "… short period region ~~to account for~~ would exaggerate inelastic effects ~~behavior~~ ~~introduces errors in the~~ …"

TS-2 Response: Editorial.

The change will be made.

Page 41 of 76, line 7 – correct spelling of "include"

TS-2 Response: Editorial.

The change will be made.

Page 59 of 76, line 15 – "deflection is the absolute lateral displacement … relative to its base" The displacement cannot be both "absolute" and "relative". Delete the word "absolute".

TS-2 Response: Editorial.

The change will be made.

Page 60 of 76, line 33 – correct spelling of "there" to "their"

TS-2 Response: Editorial.

The change will be made.

Pages 65 and 66, lines 49 – 4 – I do not understand what is being said. There seems to be a disconnect between page 65 and 66. I suggest you get the author to revise this paragraph [two paragraphs?].

TS-2 Response: Persuasive.

The text will be revised to read as follows (without strike-out and underline):

"The standard permits alternate methods of tying foundations together. Lateral soil pressure on pile caps is not a recommended method because motion is imparted from soil to structure during displacement under dynamic conditions."

Wood (YR): p. 3, line 16 – Deviations from the requirements of Chapter 12 are subject to approval *by whom*? How is "rigorously consistent" defined?

TS-2 Response: Editorially Persuasive.

The text will be modified to as follows:

"…must be rigorously consistent as specified in section 11.1.4"

Bachman (No): The vote of TS-2 on this proposal should be provided to the PUC along with any comments if not resolved.

TS-2 Response: Noted.

The commentary section was balloted and all TS-2 comments were incorporated.

I have the follow specific comment regarding this proposal. Of all the things we have in the seismic provisions, by far the most controversial and questioned are the specific height limits and restrictions found in Table 12.2-1 and Section 12.2.1. Yet the commentary is strangely silent on this subject. A simple statement such as the "Height limits and restrictions and restrictions found in Section 12.2.1 and Table 12.2-1 were made by the collective expert judgment of the PUC and the ATC-3 project team (the forerunners of the PUC). They have evolved over the past 30 years based on observations and testing but the specific values and decisions were based on subjective judgment". If TS-2 has alternate wording that's fine with me as long as it's stated.

I would change my vote to Y if a statement addressing my concern was added to Section C12.2.1 and the vote of TS-2 on this proposal were provided to the PUC along with any unresolved comments and the TS-2 Response.

TS-2 Response: Editorially Persuasive.

The following text will be added on page 10, line 4:

"Building height limits have been specified in codes and standards for over 50 years. The Structural system limitations and building height limits specified in Table 12.2-1 evolved from these initial limitations and were further modified by the collective expert judgment of the PUC and the ATC-3 project team (the forerunners of the PUC). They have continued to evolve over the past 30 years based on observations and testing, but the specific values are based on subjective judgment.

Hamburger (YR):

1. The statement in the first paragraph that: "Structures designed in accordance with the standard are likely to suffer serious damage, and possibly could collapse, if subjected to a ground motion comparable to the Maximum Considered Earthquake ground motion (MCE). " while true, in my opinion, creates an overly pessimistic statement as to the performance intent of the provisions. While some

structures designed to the provisions could collapse in MCE motion, the intent is clearly that the likelihood of this is very low (on the order of 10% or less). I would prefer that this be revised to read:

"Structures designed in accordance with the standard are likely to suffer serious damage, and with low probability ~~possibly~~ could collapse, if subjected to ~~a ground motion comparable to the~~ Maximum Considered Earthquake or stronger ground motion (MCE)."

TS-2 Response: Editorially Persuasive.

The text will be changed accordingly.

2. On page 3, line 29 (page 4, line 5) there is question mark after the Section number 13. This needs to be fixed.

   TS-2 Response: Editorial.

   The appropriate section number will be given.

3. On page 3, line 33 (page 4, line 9-11), the paragraph ends with the statement that were Cd is less than R … the system exhibits brittle behavior. This would not seem to make sense Brittle systems should provide relatively little damping and should have Cd values equal to R

   TS-2 Response: Editorially Persuasive.

   The current text states: "Where C_d is substantially less than R, the system is considered to have damping great than the 5 percent of critical damping ~~or the system exhibits brittle behavior~~." This is borne out by a review of Table 12.2-1.

4. One page 3, line 35 (page 4, lines 13-15) – I disagree with the statement that the seismic system is expected to reach significant yield for forces "well in excess" of the design forces. In fact, the design forces are supposed to be the minimum level of force permitted to produce "significant yield." Please remove the term "well in excess" and replace with "in excess".

   TS-2 Response: Editorially Persuasive..

   The text will be changed accordingly.

5. Section C12.2, page 7, line 49 (page 9, lines 40-43). the reference to the Uniform Building Code is no longer relevant and should be deleted.

   TS-2 Response: Editorially Persuasive.

   The sentence will be revised to read (without strike-out and underline):

"These categories are subdivided further for various types of vertical elements used to resist seismic forces."

6. Page 8 (page 9, lines 45 to page 10, line 3)– the discussion of R factors should include some reference to ATC-63 and acknowledgement that while historically, R factors have been set on judgment and precedent, rational procedures for determination of R factors are now available.

   TS-2 Response: Editorially Persuasive.

   The following text will be added to page 10, line 3

   "FEMA P695, Quantification of Building Seismic Performance Factors (ATC-63 Project Report—90% Draft) has been developed with the purpose of establishing and documenting a ~~recommended~~ methodology for ~~reliably~~ quantifying building system performance and response parameters for use in seismic design. While the response modification coefficient (R factor) is a key parameter being addressed, related design parameters such as the system overstrength factor (Ω_0) and deflection amplification factor (C_d) are also addressed. Collectively, these terms are referred to as "Seismic Performance Factors" (SPFs). Future systems will likely derive their SPFs using this methodology and existing system SPFs may also be reviewed in light of this new procedure."

7. Page 8, line 22 (page 10, lines 19-27)– the recommendation to provide some moment-resistance in the gravity frame would seem to be pretty outdated at this point. I don't believe anyone has provided top and bottom clip angles on gravity steel framing, as suggested for more than 40 years. Perhaps this paragraph should be deleted in its entirety and replaced with an acknowledgement that although gravity force resisting systems are not required to provide lateral resistance, many of them do, and to the extent that they provide additional lateral resistance, enhance the building's seismic performance capability , so long as they are capable of surviving the resulting stresses and deformations.

   TS-2 Response: Editorially Persuasive.

   Replace the text starting on line 19 (through the end of the paragraph) with the following:

   "Although gravity load-resisting systems are not required to provide lateral resistance, most of them do. To the extent that the gravity load-resisting system provides additional lateral resistance, they will enhance the building's seismic performance capability, so long as they are capable of resisting the resulting stresses and undergoing the associated deformations."

8. Page 8- lines 30 to 33 (page 10, lines 34-37) – suggests that Special Moment Resisting frames are required, without indicating under what circumstances. This is incorrect as currently edited.

TS-2 Response: Editorially Persuasive.

Replace the text starting on line 34 (through the line 37) with the following:

"Special moment frames designed and detailed for ductile response in accordance with Chapter 14 are required where moment-resisting frame systems are specified in seismic design categories D through F."

9. Page 11 line 48 to page 12 line 4 (page 16, lines 7-13) – this commentary does not make sense. The Commentary describes frame shear wall interactive systems in SDCs A and B. The practice described for Seismic Design categories A and B would seem to be exactly that required for this system – thus the discussion re R factors is quite confusing.

   Withdrawn

10. Page 23, line 16 (page 31, line 26-29) – It would be good to note, at the bottom of this paragraph, that in some cases, the mathematical model can be as simple as a free-body diagram, so that the impression is not created that computer analysis is being required.

    TS-2 Response: Editorially Persuasive.

    The following text will be added to the end of the paragraph:

    "In some cases, the mathematical model can be as simple as a free-body diagram as long that model can appropriately capture the strength and stiffness of the structure."

11. Page 33, lines 30-38 (page 48, lines 17-32)- In addition to pointing out that it is impossible to plot deflected shape of the structure, it would be very good to point out that usually, the forces obtained from modal combinations will not be in equilibrium.

    TS-2 Response: Editorially Persuasive.

    The following text will be added to the end of the paragraph:

    "In addition, the resulting forces obtained from modal combinations will not be in equilibrium."

12. Page 33, line 40 –42 (page 48, lines 34-36). It should be clarified that this more accurate prediction of forces and displacements is only for the elastic range of response.

    TS-2 Response: Editorially Persuasive.

The text will be modified as shown below:

"The key motivation to perform modal response spectrum analysis is to determine how the actual distribution of mass and stiffness of a structure affects the elastic displacements and component forces."

13. Page 34, lines 1 to 3 (page 49, lines 7-9)– the statement that: "**C12.9.2 Modal Response** 1 **Parameters.** The design response spectrum (whether the general spectrum from Section 11.4.5 or a site specific spectrum determined in accordance with Section 21.2) is representative of a linear elastic structure " is incorrect. It is not the representation of any one linear elastic structure, but rather of linear elastic structures. The "s" after structure is required.

    TS-2 Response: Editorial.

    The text will be modified accordingly

14. Page 34, line 27 (page 49, line 46-47) somewhat mischaracterizes the reason that response spectrum analysis often produces lower forces than the ELF. It is not that the ELF response is characterized by a single mode, but rather, that the ELF assumes 100% mass participation in the first mode, which always is an overestimate. Same comment on lines 31-32 (line 51) the clause "due to multimode response" should be deleted.

    TS-2 Response: Editorially Persuasive.

    The text will be modified as shown below and the clause "due to multimode response" will be deleted:

    "…because (1) the calculated fundamental period may be longer that that used in computing V, ~~and~~ (2) the response is not characterized by a single mode, and (3) the ELF base shear assumes 100 percent mass participation in the first mode, which is always an overestimate."

15. Page 34, line 34-35 (page 50, lines 4-8) – this is not the reason that displacements are not scaled. Displacements are not scaled because the use of an overly flexible model will result in conservative estimates of displacement that need not be further scaled.

    TS-2 Response: Editorially Persuasive.

    The text will be modified as shown below (note: existing strike-out and underlining not shown for clarity):

"~~Scaling of d~~ Displacements from the modal response spectrum ~~would produce loads that are inconsistent with the stiffness analysis~~ are not scaled because the use of an overly flexible model will result in conservative estimates of displacement that need not be further scaled."

16. Page 34, lines 43-44 (page 50, lines 16-17)– I disagree that torsional amplification is required for this approach if the structure is torsionally regular.

    Withdrawn

17. Page 41. line 33 (page 61, line 33) – this paragraph recommends that separation be no less than the sum of the anticipated displacements on either side of the joint. This is contrary to typically design practice in the western U.S. that uses the root sum of squares displacements. I suggest this be replaced to show root sum of squares.

    TS-2 Response: Persuasive.

    The text will be revised to read (note: existing strike-out and underlining not shown for clarity):

    It is recommended that the distance be no less than the ~~sum~~ square root of the sum of the squares of the lateral deflections of the two units assumed to deflect toward each other (thus increasing with height)."

18. Page 43, lines 22-28 (page 64, lines 5-11). This paragraph tells readers that the standard is out of date and alternative procedures should be used. This is inappropriate commentary and violates rules given the PUC as to how to formulate commentary. It would be better to say that the ASCE 41 approach is an acceptable alternative to that contained in the standard and may provide more realistic results.

    TS-2 Response: Editorially Persuasive.

    The text will be revised to read:

    "The strain-compatible shear modulus, G, and the associated strain-compatible shear wave velocity, v_s, needed for the evaluation of equivalent linear stiffness ~~should~~ are specified in Chapter 19 of the standard or can be based on a site-specific study. ASCE 41 is an acceptable alternative to that contained in the standard and may provide more realistic results. ~~be determined using the criteria in ASCE 41 or based on a site-specific study; the values provided in Chapter 19 of the standard are out of date. The standard requires parametric variations of the stiffness.~~"

Klingner (YR): This is generally good. Some parts of it will need to be updated if SDPRG 1R4 passes. Please correct the misspelling on Page 30, line 23 (“include”).

> ***TS-2 Response: Non persuasive/Editorial.***
>
> *This commentary is developed for Part 2 of the provisions and is not affected by Part 1 proposal SDPRG 1R4*

Proposal 8-113 (Y=16, YR=2, N=0, NV=4 --100%)

TS 8 addressed the Bachman comment by stating that the TS 8 vote was (8=Yes, 0=Yes with Reservation, 0=No and 5=Not Voting). Hamburger’s comments were found editorial and accepted.

Bachman (YR): The vote of TS-8 on this proposal should be provided to the PUC along with any comments if not resolved. I would change my vote to Y if the vote of TS-8 on this proposal were provided to the PUC along with any unresolved comments and the TS-8 Response.

> ***TS-8 Response: Persuasive.*** *Vote totals will be added. There were no unresolved comments.*

Hamburger (YR):

1. Page 7, line 21 – the statement “the following behavior is anticipated during the design earthquake” is not consistent with the three bullet items that follow. Suggest changing to read” “the following behaviors are anticipated for shaking having different levels of intensity.”

2. P. 19 line 37 “butmay” should be “but may”

> ***TS-8 Response: Persuasive.*** *Suggested changes will be made.*

Proposal 6-114 (Y=15, YR=1, N=2, NV=4 --89%)

The comments and response were all directed to the sections referring to Concrete (14.2). This chapter is a commentary to ASCE 7-05 and several have concern that it should reflect several changes to ASC 318-08. However, this was not published when ASCE 7-05 was published so there are several differences by definition. One change directed by the PUC was to replace the reference to ACI 318-08 with -05, and this will be done. A motion was made to find Wood and Klingner nonpersuasive and this was approved (16,1,2). The Bachman comment was discussed and satisfied. Since it was a YR vote, no PUC vote was required.

Wood (No): Section C14.2 does not reflect the changes adopted in ACI 318-08. Approximately half of the provisions in Section 14.2 of ASCE 7-05 do not apply to ACI 318-08. The PUC decided that the commentary would be based on ACI 318-08, rather than ACI 318-05. This is particularly important because ACI 318-08 has adopted Seismic Design Categories throughout Chapter 21 and this change makes the integration of ASCE 7 and ACI 318 *much easier*. If the PUC decides that we must have a commentary section for each subsection in C14.2, then the issues that have been resolved in ACI 318-08 should be clearly stated. For example:

C14.2.2.3 ACI 318, Section 11.11 This requirement is intended to provide additional toughness to resist shear for columns of frames in SDC B. Otherwise the proportions of those columns make them more susceptible to shear failure under earthquake loading. <u>This requirement has been included in Section 21.2.3 of ACI 318-08.</u>

p. 5, line 12 – The provision affects ordinary moment frames <u>in SDC B</u>.

> ***TS 4 Response: Persuasive.*** *Contrary to what is said in Proposal 6-114 ASCE 7-05 adopts ACI 318-05 and not ACI 318-08. ACI 318-08 uses SDC terminology while ACI 318-05 does not. Further, the use of ACI 318-08 allows the removal of almost one half of the modifications to ACI 318 that are adopted in ASCE/SEI 7. It is highly desirable that this ASCE/SEI 7 commentary note those facts and inform the user as to the location of the best available commentary information.*
>
> *It is moved that the draft C14.2 commentary in Proposal 6-114 be replaced by the following:*

"C14.2 CONCRETE

In C14.2.1 the section adopts ACI 318 by reference for the structural concrete design and detailing and in Chapter 23 the relevant edition of ACI 318 is defined as ACI 318-05. In C14.2.2 the section adopts 18 specific modifications to ACI 318-05 and in C14.2.3 the section adopts additional provisions for deep concrete foundations. There are no provisions for deep concrete foundations in ACI 318.

Part 1 of the 2009 NEHRP Provisions adopts ACI 318-08 as the reference document for structural concrete design and detailing. Changes made in ACI 318 provisions between its 2005 and 2008 Editions permit a reduction from 18 to 9 in the number of specific modifications to ACI 318-08 needed for consistency with ASCE/SEI 7-05 seismic design requirements. Appropriate commentary for the remaining nine modifications, and for C14.2.3, are contained in Part 1 of these 2009 Provisions. Commentary for the nine modifications that are no longer needed is provided by ACI 318-08 Commentary.

In ACI 318-05 low, intermediate and high seismic risk designations are used to delineate detailing requirements. In ACI 318-08 detailing requirements are related to type of structural framing and seismic design categories (SDC) with SDCs adopted directly from ASCE/SEI 7.

The following Table C14.2 lists the ASCE/SEI 7-05 provisions that can be eliminated with the adoption of ACI 318-08 and gives the specific ACI 318 provision that replaces the corresponding ASCE/SEI 7 provision.

Table C14.2
ASCE/SEI 7-05 Provisions Replaced by ACI 318-08 Provisions

ASCE/SEI 7-05 Provision	Provision Scope	ACI 318-08 Provision
14.2.2.2	Requirements for flexural members of SDC B frames	21.2.2
14.2.2.3	Requirements for columns of SDC B frames	21.2.3
14.2.2.6	Use of prestressing steel in members resisting earthquake induced forces	21.1.5.3 & 21.5.2.5
14.2.2.7	Anchorages for unbonded post-tensioning tendons	21.5.2.5
14.2.2.8	Use of prestress in flexural members of special moment frames	21.5.2.5
14.2.2.12	Members not designated as part of LFRS	21.13.3
14.2.2.13	Columns supporting reactions from discontinuous stiff members	21.6.4.6
14.2.2.16	Plain concrete in structures assigned to SDC C, D, E or F	22.10
14.2.2.17	General requirements for anchoring to concrete	D3.3

"

Bachman (YR): The vote of TS-4, 5, 6 and 7 on this proposal should be provided to the PUC along with any comments if not resolved. I would change my vote to Y if the vote of TS-4, 5, 6 and 7 on this proposal were provided along with any unresolved comments and the TS-4, 5, 6 and 7 Responses.

> ***TS 4 Response: Non-Persuasive.*** *TS-4 Chair's email correspondence with Mike Valley shows that this proposal was developed by Mike in response to a directive from the last meeting of the PUC. Because of that directive he did not feel it necessary, nor that there was adequate time, for him to share the proposal with TS-4, 5, 6 and 7 prior to the timeline for submission for the this PUC vote.*

Klingner (No): Many of the references to the 2005 MSJC Code are out of date, and many of the Material-Specific requirements for masonry will have been removed because they are already addressed in the 2008 MSJC Code. Specific provisions that have been address and therefore no longer need appear in ASCE 7 have been detailed in previous TS-5 proposals and proposed commentary in this NEHRP cycle.

Proposal 8-115 (Y=14, YR=2, N=1, NV=5 --94%)

The Bachman's comment was resolved by the TS 8 vote (8=Yes, 0=YR, 0=No, and 5=Not Voting). The Wood comment was withdrawn and the Klingner comment was noted. This proposal is approved for Member Organization ballot.

Wood (No): p. 6, line 4 – I do not agree with the second sentence in this section: "The dynamic response of nonbuilding structures with a fixed base and a relatively uniform distribution of mass and stiffness, such as bottom-supported vertical vessels, stacks, and chimneys, can be represented adequately by a cantilever (shear building) model. *For these structures the equivalent lateral force procedure provided in the standard is suitable*." This is only true if the first mode dominates the response of the nonbuilding structure. If the stack or chimney is very tall, the higher modes will have a significant influence on the dynamic response, and the equivalent lateral force procedure will not be sufficient to represent the influence of these higher modes.

> ***TS-8 Response: Nonpersuasive.*** *While, the higher mode effects do have a significant influence on the dynamic response of the structures listed, the result of these higher mode effects is to reduce (sometimes significantly) the overturning moment and base shear. Therefore, ELF is suitable for the design of these types of structures because ELF results in conservative design loads for practical sizes of these types of structures.*

Bachman (YR): The vote of TS-8 on this proposal should be provided to the PUC along with any comments if not resolved. I would change my vote to Y if the vote of TS-8 on this proposal were provided along with any unresolved comments and the TS-8 Response.

> ***TS-8 Response: Persuasive.*** *Vote totals will be added. There were no unresolved comments.*

Klingner (YR): This is generally good. Some parts of it will need to be updated if SDPRG 1R4 passes.

> ***TS-8 Response:*** *Noted.*

Proposal 2-116 (Y=16, YR=1, N=2, NV=3 --89%)

TS 2 set out to describe this proposal and addressed each comment. The second Bachman comment sparked a lengthy discussion on spectral matching that could not be completely resolved. Consequently, the PUC chair proposed language to provide a warning to the reader on spectral matching. A motion was made to accept the persuasive comments of Bachman and Hamburger and the PUC agreed by a vote of (19,0,0).

Crouse (YR): Will need revision if SDPRG 1-R4 passes.

TS-2 Response: Persuasive.

The text will be modified if Proposal SDPRG-1R4 is adopted.

Bachman (No): The vote of TS-2 on this proposal should be provided to the PUC along with any comments if not resolved.

TS-2 Response: Noted.

The commentary section was balloted and all TS-2 comments were incorporated.

I have the following specific comments regarding this proposal.

1. Equation C16.1-2 should be replaced by the 2 explicit equations for α and β. It is silly to keep it in matrix format and expecting people to solve it to determine.

TS-2 Response: Partially Persuasive.

The explicit equation is quite complex, and more prone to error than the given equation. However, we propose to add the following text to the end of line 36 on page 3:

"If both damping values are the same ($\xi=\xi_a=\xi_b$), which is usually the case, the mass and stiffness proportionality constants may be determined as follows:"

$$\alpha = \xi \frac{2\omega_a\omega_b}{\omega_a+\omega_b}$$
$$\beta = \xi \frac{2}{\omega_a+\omega_b} \qquad \text{C16.1-3}$$

2. In Section 16.1.3, the acceptability of using spectral matching for developing time histories in the western US should be discussed. I personally am opposed to spectral matching but there are many engineers who do request ground motion consultants to provide them in order to minimize design forces on their structures (gaming the system).

TS-2 Response: Persuasive.

The following text will be added to page 4, line 26:

> *“The use of "spectral matching", wherein a ground acceleration history is "reverse engineered" from the design spectrum is not recommended. Such ground motions, when based on uniform hazard spectra, are effectively the sum of several motions (from near and distant earthquakes, for example), and have an unrealistic frequency content.”*

3. In Section C16.1.3.2, explicitly including vertical ground motion time histories in Three-Dimensional Analysis should be discussed. Also it should be stated that without explicitly including vertical ground motion time histories in the analysis and properly modeling for vertical response, we are really only doing 2 ½ Dimensional Analysis.

> ***TS-2 Response: Persuasive.***
>
> *The following text will be added to page 5, line 37:*
>
> *"For certain structures, the response under both horizontal and vertical ground motions should be considered. It is noted, however, that vertical ground motion spectra are not readily available, so the scaling of the vertical components of ground motion would be problematic."*

I would change my vote to Y is the above 3 changes were made as requested and the vote of TS-2 on this proposal were provided along with any unresolved comments and the TS-2 Response.

Hamburger (No): My negative is with regard to statements made under the initial paragraphs of Section C16. regarding nonlinear response history analysis, as follows:

1. The opening sentence states: “Nonlinear response history analysis is not used as part of the normal design process for typical structures” While this statement is true, this technique is seeing increased use for the design of structures other than base isolated and damped structures, e.g. tall buildings.

2. The opening sentence of the second paragraph states: “The principal aim of nonlinear response history analysis is to determine if the computed deformations of the structure are within appropriate limits. Strength requirements do not apply because element strengths are established prior to the analysis” In fact, strength requirements are important, particularly for nonductile elements. While NLTH does predict deformation demands on nonlinear elements, it does and must also be used when applied, to predict strength demands on elements that are designed to remain elastic.

3. Section C16.2.4.1 says strength need not be checked. Strength must be checked for elements that are designed to remain elastic, for example, columns beneath discontinuous walls, connections of braces, etc.

My negative will be withdrawn if this section of commentary is rewritten to acknowledge that:

1. While not commonly used in the past to design typical structures, this technique is seeing increased use in the design of some structures, including structures that are neither damped or base isolated.

   TS-2 Response: Persuasive.

   The above sentence will be added to the end of the first paragraph of section C16.1.

2. That the technique need not be used to supplement the results of elastic design and analysis, and can be used as a direct method of design.

   TS-2 Response: Nonpersuasive.

   While there is some agreement regarding the described approach, adding such a statement does not really provide the reader any real guidance as to how such an analysis method may be use to produced a design that conforms to the various requirements of the provisions.

3. That both forces and deformations are predicted and monitored for use in design.

   TS-2 Response: Persuasive.

   The paragraph starting on line 35 of page 6/11 will be changed (changes in underline below) to read:

   "The principal aim of nonlinear response history analysis is to determine of the computed displacements of the structure are within appropriate limits. Strength requirements <u>for the designated lateral load-resisting elements</u> do not generally apply because element strengths are established prior to the analysis. These initial strengths are typically determined from a preliminary design using linear analysis.

   <u>The nonlinear response history analysis may also provide useful information on the strength requirements of nonstructural components, which are often assumed to remain elastic in the analysis."</u>

PART 3

Proposal IT1-3R (Y= 17, YR=9, N=5, NV=6 --84%)

IT1 attempted to resolve all Member Organization comments. However, it became clear from the vote of the IT members that the comments can be properly resolved if the proposal is moved from Part 1 to Part 3. Since a large number of Member Organization representatives had voted on this proposal, the PUC felt that moving this proposal to Part 3 warranted it to be voted again by the Member Organizations. Therefore, by a vote of Yes=15, No= 3, and Not Voting =5, the PUC approved having the Member Organizations revote this proposal to be in Part 3. It will be re-balloted to the member organizations.

~~Proposal 2-1R5 (Y=17, YR=2, N=2, NV=1 --90%)~~ FAILED

TS 2 has provided a revised proposal that was distributed to the PUC in November. During the discussion, there was grave concern that this proposal lacks sufficient detail even for Part 3. After lengthy discussion a motion was made to accept this proposal as written and it failed the PUC (6,12,1). The proposal failed. Another motion was made to find Ghosh and Hamburger comments persuasive and this was amended to include a statement that this proposal should be moved forward to be a future research need. This motion, as amended, passed (18,0,0).

Saunders (YR): I feel that there should be an explanation of the goal of putting this proposal in Section 3. I assume that the purpose is for engineers to use it in trial designs, test its effects economically, but also, where non-linear analyses are used, compare the performance of otherwise similar designs. Since this proposal was voted down for inclusion in Part 1, I
not anxious to see it move into Part 1, simply because it has been around for awhile.

Ghosh (No): I do not understand: “Where a special moment frame column extends above a level, the flexural strength shall be at least 65% of the column flexural strength below.” (p. 1, Lines 24, 25) Which specific flexural strength are we talking about?

The only justification I see for the proposal is on p. 2, Lines 11-12: “The story approach is shown by plastic analysis studies to be an appropriate provision against the formulation of undesirable story mechanisms.” This is just an assertion and a vague one at that.

The proposal, by its own admission, requires, on average, stronger columns and/or weaker beams compared to current requirements. If reinforced concrete special moment frame columns have to be stronger than they currently are, that is a problem. Larger column sizes are typically unacceptable for obvious reasons. More longitudinal reinforcement causes significant congestion problems and may turn out to be a counterproductive requirement. Weaker beams may very well mean more lateral drift. I think we need to evaluate the consequences a little more before embracing this proposal.

Hamburger (No): My negative is because I believe that a preamble or commentary should be provided that explains why the proponent believes that present strong column – weak beam provisions are inadequate, and also discusses why the PUC did not deem that this set of provisions was ready for placement into Part 1 at this time. Without such explanation, reviewers of Part 3 will not have an understanding of the issues and inappropriate adoption or rejection may occur at a later date. My negative will be removed if an appropriate commentary is provided. Much of the material that accompanied the original submission of this proposal for Part 1 could be used to make up such a commentary.

Klingner (YR): I am willing to believe the rationale's assertion that "The story approach is shown by plastic analysis studies to be an appropriate provision." I believe that a reference or example calculation supporting that statement should be presented, so that voters can judge for themselves. I believe that the chances of a successful member organization ballot will be enhanced by inclusion of such a summary in the rationale.

Proposal 2-3 R4 (Y=17, YR=0, N=2, NV=3 --89%)

TS 2 used the time between the September, 2008 PUC meeting and the December meeting to review this proposal based on the two No comments received. They revised the proposal to incorporate the persuasive comments listed below and presented this proposal. The PUC agreed with the TS revised proposal by a vote (18,0,0).

Bachman (No): The vote of TS-2 on this proposal should be provided to the PUC along with any comments if not resolved. Also, in Section X.4, the selection of structural systems is restricted to those found in Table 12.2-1 and Table 15.4-1 is ignored. I would change my to Y if the vote of TS-3 on this proposal were provided along with any unresolved comments and the TS-3 Response and Table 15.4-1 was added for nonbuilding structure applications (for structural systems similar to buildings) in Section X.4.

> ***TS-2 Response: Persuasive.***
>
> *Revision 3 of this proposal was discussed at the last PUC meeting. The proposal was to move the Appendix, currently to reside in Part 3, to Part 1. During the discussion, TS-2 withdrew this proposal, with the understanding that it would be resubmitted for Part 3. Thus, no comments were received from TS-2 members on Revision 4, which went directly to the PUC.*
>
> *The text for Section X.4 will be changed as follows:*

X.4. Seismic force-resisting system. The seismic force-resisting system shall conform to one of the types indicated in Tables 12.2-1 and 15.4-1, and shall be in accordance with the seismic design category and height limitations indicated in these~~is~~ tables. The appropriate response modification coefficient, *R*, and system overstrength factor, Ω_0, indicated in these tables~~Table 12.2-1~~ shall be used, subject to the following.

Hamburger (No): My negative is for the following:

1. I disagree with the approach that nonlinear analysis be used only as a supplement to elastic analysis. It should be permitted to use nonlinear analysis as the primary means of design, particularly when introduced in Part 3.

TS-2 Response: Partially Persuasive.

The voter may be referring to the Commentary (Page 7 Line 42) where there is mention that the NSP may be used to ensure that systems designed using the ELF achieve strengths comparable to code expectations. However, there is no requirement to do a linear analysis or design, except incidentally for purposes of determining modal properties.

Previous versions of the proposal attempted to move the NSP from Part 3 to Part 1. We have responded to comments received. However, the limitation of the use of this procedure to structures less than 40 feet in height made some PUC members question its placement in Part 1. This is stated in the first part of the Reason statement. We wish to retain the procedure and believe there is potential to extend its use to taller structures when supplemented by additional analysis.

Proposal 2-5R5 (Y=9, YR=5, N=6, NV=3 --74%)

This proposal was mail balloted during July-August of 2008. It was discussed at the September 2008 PUC meeting primarily addressing the PUC comments. Due to the complexity of the discussion, TS 2 never brought this proposal to vote and was deferred to the December, 2008 PUC meeting. During the interim, TS 2 met again to refine this proposal by first incorporating all the persuasive PUC member comments. TS 2 members generated several additional comments and resolved them sufficiently to gain general acceptance. Proposal 2-5R5 was then sent to the PUC members for review. At the December meeting, there was additional discussion on comments provided by Mr. Bachman. Several were editorial and a few substantive changes were made. The PUC had difficulty resolving an issue on Sec. 16.2.1 Duration. TS 2 deferred the discussion to develop acceptable language during the lunch break. TS 2 returned with the changes received from comments earlier in the meeting. The PUC accepted these changes by a vote of (18,0,1).

Since this proposal was finalized late in the cycle, TS 2 has not had a chance to develop appropriate commentary. The PUC decided that since commentary was necessary, they have opted to place this proposal in Part 3 vice Part 1.

Crouse (No): General Comments: There is no indication that any of the comments of TS-2 members have been addressed in this revision. Some of the revisions are unclear and improvements and final revisions depend in part on whether Proposal SDPRG IR4 passes. If it does, then "MCE" will need to be replaced.

Specific Comments:

1. Sect. 16.1.1, line 23, Item 1: Documentation should also be approved by Design Review Team.

TS-2 Response: Persuasive

The text in Section 16.5 was made to clarify this point.

2 Sect 16.1.3: Should analysis for OC's III & IV be based on MCE motions, as stated in 16.1.2? If so, it should be stated in 16.1.3.

TS-2 Response: Persuasive

The text in Section 16.1.2 was editorially revised.

3 Section 16.2.1. The 0.05g requirement is questionable for long period structures, and even if it could be justified in the Western U.S., it probably needs to be lowered for low seismic areas of east, which could experience shaking from distant large earthquakes that would have accelerations lower than 0.05g, but significant long period displacements that would be more critical to the response of long period structures.

TS-2 Response: Persuasive

The text was changed to allow truncating the tail of the record, so long as no values exceeding 10% of peak acceleration of the record are removed.

4. Sect. 16.3.2, line 6: Meaning of "essentially elastic" not clear.

TS-2 Response: Persuasive

The definition of "essentially elastic" is already defined in Section 16.4.3. An editorial reference to Section 16.3.2 has been added.

5. Sect 16.3.5, line 21: meaning of "semi rigid" not clear.

TS-2 Response: Persuasive

The entire section was edited for clarity.

6. Sect 16.3.5, lines 32-34: This sentence refers to Sect 16.3.12 for determination of whether nonlinear force-deformation relations are required, but 16.3.12 does not say.

   TS-2 Response: Editorially Persuasive

   The reference should be 16.3.11.

7. Sect 16.3.7, line 44: RHA should be performed for expected values of D and L. The 1.0 factor on D indicates that is the case for dead loads, but why specify a 0.5 factor for L? Shouldn't the engineer decide the % of L to use? If he cannot decide, then 0.5 is the default. Also, L is defined in Sect. 11.3 (Notation) as a length parameter, but it also needs to be defined as live load.

   TS-2 Response: Partially persuasive

   The use of 0.5 times the reduced L (live load) is used by ASCE 7 to represent the expected value at any arbitrary point in time. An allowance for different values for gravity loads is inserted, subject to design review.

Sect 16.3.11, line 69: Not clear how strength is based on hysteretic strength degradation without actually running dynamic analyses to see how much degradation actually occurs. Shouldn't hysteretic strength degradation be built into the constitutive model used in the dynamic analysis? Presumably such a model would be derived from dynamic load testing of prototypes.

TS-2 Response: Nonpersuasive

The strength degradation "modeled" for use in analysis is to be "consistent with applicable laboratory tests data." Thus it appears to TS2 that the proposed wording already includes what is requested.

Sect. 16.3.12. Wording of this section needs improvement. What exactly is meant by element properties? The title of the section is Stiffness, so presumably stiffness is to be determined, but that's not what the section says.

TS-2 Response: Nonpersuasive

TS2 believes that "element properties" for stiffness are understood to include flexural, axial, and shear stiffness (EI, EA, GA).

Sect. 16.4.2., line 3: Is "average" the correct word? If one out of many RHA collapses the structure, then the "average" could not be calculated. In that case, the median value is more appropriate provided 7 or more RHA are run.

TS-2 Response: Persuasive

The new commentary on the treatment of this issue discusses that collapse is not permitted in any of the 7 RHA.

Sect. 16.4.3, last sentence: “force” should be “stress” to be compatible with “strength.”

TS-2 Response: Nonpersuasive

“Force” here is used in a generic sense.

Sect. 16.4.3.1, 1st sentence: Replace “peak value” with “member’s peak strength.”

TS-2 Response: Partially persuasive

An editorial clarification has been added.

Sect. 16.4.3.2: Is there a precise definition of “nominal capacity,” so that two different engineers would compute or assign similar values for this term for the same structural element? If not, then different word should be used, or sentence added that quantitatively defines nominal capacity.

TS-2 Response: Nonpersuasive

Nominal capacity is consistently defined is the various standards used for determining capacities of elements (the material design standards).

Sect. 16.4.5: Sentence not clear. The way it’s worded, “collapse” refers to the “elements not designated,” not the structure.

TS-2 Response: Editorially Persuasive

Sect. 16.4.6: Can stability criteria be specified, or is the intent to leave this decision of whether the structure is stable to the engineer?

TS-2 Response: Nonpersuasive

Lack of stability is a clear result from an analysis; no interpretation is needed.

Saunders (YR): I am voting YR in support of TS2 members that feel that more refinement of this proposal is needed. I will change my vote to Yes when their comments have been considered and resolved.

TS-2 Response: Nonpersuasive

TS2 was not able to reach unanimity, but our process does not require that high a standard. The response to the PUC comments *has been prepared by a subset of TS2 that is a quorum of TS2 (Hooper, Harris, Aschheim, Johnson, with email input from Charney.).*

Wood (YR):
p. 6, line 9 – This proposal refers to the MCE response in several places. However, Ballot item SDPRG 1R4 has eliminated all references to MCE and refers to RTE. If the Project 07 recommendations are approved, should "MCE" be changed to "RTE" throughout?

TS-2 Response: Persuasive

This will be accomplished once the BSSC membership approves the SDPRG proposal.

p. 6, line 15 – Should this section reference 16.3.11, rather than 16.3.12?

TS-2 Response: Editorially Persuasive

p. 6, line 20 – This section permits "specific exceptions" to be taken to the requirements of the equivalent lateral force procedure and the modal response spectrum analysis procedure. However, the scope of these specific exceptions is not defined.

TS-2 Response: Nonpersuasive

It is assumed that the intent of this comment is to either delete the word "specific" or to clearly define the scope of exceptions. The scope is to be defined by the engineer proposing a design validation according to this new section 16. The word specific is used simply for emphasis to preclude a designer simply stating that a design does not satisfy section 12; the requested identification of exceptions is to be specific.

p. 7, line 45 – The discussion of how to calculate L is rather ambiguous. Is it the intent that the reduced live load is constant throughout the building?

TS-2 Response: Partially Persuasive

The value of L_0 will not necessarily be constant throughout the building, but the area used for computation of the live load reduction factor will be based upon all the area subject to that load within the building. A three-part editorial clarification is added at this point: first to make the redefinition of the tributary area permissive rather than mandatory, to clarify that the "total area" is only that area loaded with a particular live load that is reduceable, and a clarification that K_{LL} (Table 4-2) shall be set at 1.0.

Bachman (No): This proposal got scrambled and there is really no explanation on what we received. It appears that the proposal went out to ballot to TS-2 and there were apparently many excellent TS-2 member comments. But there is no indication that there was any attempt to resolve the comments by TS-2 before forwarding the proposal to the PUC. The proposal should be returned to TS-2 and the comments resolved and then forwarded to the PUC for balloting.

> ***TS-2 Response: Noted***
>
> *The proposal was modified in response to those TS2 comments that were tractable. Some of the comments did not require resolution (for example those that Harris noted as deficiencies in the existing standard).*

Also for scaling of ground motions, the proposal presumes that a version of SDPRG 1R4 proposal will pass that contains the ground motion maximum direction requirement. That is not a done deal. I would suggest that the changes in scaling be stated in the revised proposal that the values would change to these alternate values if a version of the SDPRG 1R4 proposal is approved that contains the ground motion maximum direction requirement.

> ***TS-2 Response: Persuasive***
>
> *If the maximum direction of response is approved, the 1.3 scaling factor will become 1.0 – However, proposal 2-8 will control this particular issue.*

Finally, I would suggest that some mention of vertical ground motion time histories be made in Section 16.2. It's difficult for me to understand how we could do the level of sophisticated analysis suggested in the revised Chapter 16 and totally ignore vertical ground motion time histories as if they didn't exist. It is rationally inconsistent to perform highly sophisticated nonlinear dynamic analysis with only horizontal motion pairs and to then only utilize the $0.2S_{DS}$ factor to account for vertical ground motions instead of including vertical motions directly into the analysis.

> ***TS-2 Response: Partially persuasive***
>
> *Commentary will be written indicating types of structures for which vertical motion should be considered (for example, tanks, very long spans, and long cantilevers). We are not ready to write a general provision for evaluation of response to vertical ground motion.*

Manley (YR): My reservations with this particular proposal continue to be primarily editorial in nature. While these do not necessarily need to be corrected in this venue, they will need to be addressed if and when this proposal is forwarded to ASCE 7 for consideration. Specifically:

- Section 16.1.3: What is meant by “more restrictive” in line 33? Ultimately, how is this requirement going to be interpreted by users and enforced by authorities having jurisdiction (AHJs)? Will there be commentary on these sections?

> ***TS-2 Response: Persuasive***
>
> *See the response to Holmes concerning the acceptance criteria.*

- Section 16.2, 16.2.3 and 16.5: The use of “appropriate” throughout these sections seems unnecessary and vague. It either needs to be better defined or eliminated.

> ***TS-2 Response: Noted***
>
> *Pass this issue on to ASCE 7 to resolve.*

- Section 16.3.1: I wonder if the ‘average’ user will know how to interpret the phrase “… can adversely affect the response of the structure…” in line 98? Is there commentary on this section to better explain this concept?

> ***TS-2 Response: Persuasive***
>
> *Commentary will be written to illustrate examples of nonstructural influence on structural performance.*

- Section 16.3.2: Again, I wonder if the ‘average’ user will know how to interpret the phrase “to remain essentially elastic” in line 6 of 16.3.2. Is there a commentary on this section?

> ***TS-2 Response: Persuasive***
>
> *See response to Crouse.*

- Section 16.3.12: I wonder if the ‘average’ user will know how to interpret the opening phrase “To the extent that such effects are significant”? Is there commentary on this?

> ***TS-2 Response: Nonpersuasive***
>
> *The average engineer will not use this method. Engineers who model nonlinear response will understand that changes in element properties cause changes in the computed response.*

Also, the two subparagraphs need to be numbered, since they are part of a list.

> ***TS-2 Response: Editorially Persuasive***

- Section 16.3.13: Consider deleting “adequately” in line 90 – it doesn’t seem necessary.

Or, consider substituting in the language used in Section 18.3.1 – "Inherent damping of the structure shall not be taken greater than 5 percent of critical unless test data consistent with levels of deformation at or just below the effective yield displacement of the seismic force-resisting system support higher values."

TS-2 Response: Editorially Persuasive

Holmes (No): My vote would change to YR if the proposal is moved to Part 3. In my opinion, placement in Part 3 does not automatically mean that the proposal or an improved similar one cannot be adopted by ASCE 7. However, approval of the proposal as-is does not resolve comments by Harris (or the No from Maffei). Prior to adoption into ASCE 7, I think the following issues need to be considered:

1. Gravity load combinations as noted by Harris

   TS-2 Response:Partially Persuasive

   State of the art is all analyses are done for one gravity load combination for ordinary occupancies.

2. Guidance on combination of actions (P/M, P/deformation, etc) from suite of responses to be used for design. Average P and average M never occurs doesn't make sense.

   TS-2 Response: Persuasive

   The text has been revised to capture parameters consistent with the acceptance criteria.

3. Treatment of dispersion as noted by Harris. Maffei's proposal appears conservative, particularly in the deterministic region where spectral values will be increased by one standard deviation already.

   TS-2 Response: Persuasive

   Dispersion in RHA results is deemed to be captured by the requirement to perform analysis for 7 ground motions and use the average of the peak results, combined with the fact that the acceptance criterion for nonductile response is two-thirds of the expected capacity.

4. Since many of these analyses seem to be needed in the near field where an epsilon of 1 is built in, maybe it is time to deal with the spectrum used to scale the response histories. At least we can acknowledge the issue and reference methods such as the Conditional Mean Spectra.

   TS-2 Response: Nonpersuasive

This is best dealt with in proposal 2-8.

5. ASCE 41 deformation values are referenced but not by performance level, which makes them indeterminate. It is not clear if this is purposeful.

   TS-2 Response: Persuasive

   The specific levels are now referenced.

6. Similar to item 4, consideration of Occupancy Category III and IV suggests acceptance criteria "more restrictive" than the indeterminate ones used for OC I and II, but consistent with the importance factor (which none of us believes is a valid method of providing better performance). If we want to obfuscate this issue, we should at least tie the acceptability criteria to performances suggest by the Occupancy Categories rather than the I factor.

   TS-2 Response: Persuasive

   The specific citation is now made.

7. Use of motions other than MCE (see Proposal 2-5RA)

   TS-2 Response: Nonpersuasive

   This is outside the scope of this proposal.

Aschheim (YR): Since I have voted on this proposal within TS-2, I will try to address only the reservations that were expressed in the TS-2 ballot:

1. I like Maffei's suggestions to replace sections 16.4.2 through 16.4.4 with new 16.4.2 through 16.4.5.

   Withdrawn

2. Section 16.4.5 on story drift is needed. I find the "…unless" clause unsatisfying. I think this would be read as requiring only that the gravity frame not collapse under the mean story drift. This is inconsistent with the views expressed previously on this proposal about the permissible incidence of local or global collapse. It would be preferable to consider dispersion, perhaps as is suggested for demands for low ductility actions (e.g. mean plus one standard deviation).

   TS-2 Response: Partially persuasive

   The text has been clarified to include that the structure supported by elements not part of the SFRS shall not collapse under the MCE level drift.

This brings up a minor but not insignificant point—as I understand it, the term "action" is used generically to refer to forces and moments. It should be defined. But more importantly, "low ductility actions" seems to suggest the actions are low-ductility, whereas it is intended to mean actions in low-ductility members. This distinction is important if we begin to think of "actions" as potentially including other response quantities such as story drifts. Here my point is clearest—the drift affects all types of members, but it is only the action (drift) in the gravity system that we are concerned about here.

TS-2 Response: Persuasive

Exceeding 1.5 times standard drift limits now is treated somewhat like a nonductile action. Action has mostly been editorially removed.

If we are going to exceed 1.5 times the Section 12.12.1 limits, shouldn't we also be concerned about the adequacy of the seismic force resisting system to withstand these drifts? For a "ductile" seismic force resisting system, we are comparing mean demands with member capacities (allowed to degrade to 80% of peak strength based on whatever test protocol) at the element level. Again, use of mean demands here doesn't seem to provide enough confidence against collapse (due to lateral strength degradation and possibly due to gravity failures of lateral systems). Since at a minimum a frame is a series combination of elements, modeling errors (or uncertainty) may make checking each element deformation demand at 20% strength loss optimistic—that is, demands may be higher than computed in some elements. It seems to me we should aim to ensure upper bound interstory drift demands do not cause substantial loss of lateral strength.

TS-2 Response: Nonpersuasive

The analysis of the structural system is intended to satisfy modeling requirements that capture degradation that could lead to collapse.

3. I think it is essential to keep a section that addresses stability (16.4.6). More specific guidance (e.g. distinguishing between numerical issues and large drifts and the acceptable fraction of analysis runs that show instability) could be helpful.

TS-2 Response: Persuasive

The Section has been kept and the text amplified.

4. As Maffei suggests, we should also pick up my comments (on the TS-2 ballot) 1, 3, and 4. I think 2 and 6 should also be addressed—2 is a simple editorial change, and 6 by defining what is meant by "the mean value" or the "mean plus one standard deviation of the values" as stated in 16.4. The new language proposed in 2-5RA Section 18.3.1 may be useful.

TS-2 Response: Persuasive

All 6 comments have been addressed, mostly as requested.

Hamburger (No): Overall, I believe this proposal is greatly improved from the last submittal. I could be convinced to support his proposal with several revisions, as indicated below. However, I am not convinced that the current provisions are that badly broken that failure to adopt this change is a problem. I am not convinced that the code has to be a cookbook that tells the engineer precisely how to do every step of the design process. I am quite happy with provisions that are permissive, within bounds, rather than prescriptive, which the present requirements are.

1. The reason statement should be changed removing the tie of this approach to verification of a linear design. This was a primary concern of my prior negative and remains a concern. I offer as evidence the fact that both the LATBC guidelines for performance-based design of tall buildings, and the presently developing PEER Center criteria do not require an elastic DE analysis as part of the design procedure. If this proposal goes out with this reason it will be behind the times of progressive seismic engineering thinking.

TS-2 Response: Persuasive

The reason statement has been modified.

2. Section 16.2 - Lines 57-58 I do not care for the language “such as velocity pulses ….etc” I would prefer language as follows to replace this: “including directionality of ground motion and velocity pulses, as appropriate. (this is a reservation only)

TS-2 Response: Editorially Persuasive

3. Section 16.3.2 requires identification of all elements that will experience nonlinear response. This is a more restrictive requirement than applies for structures designed using linear analysis procedures, which permit nonlinear behavior of elements that are not part of the lateral force resisting system, if they are detailed to permit such nonlinear behavior. My negative will be withdrawn if this section is revised to apply only to the intended elements of the lateral force resisting system.

TS-2 Response: Persuasive

The text in Section 16.3.2 has been modified to discuss only the elements of the Seismic force-resisting system.

4. Section 16.3.7. The proportion of live load present should be consistent with the proposed requirements for P-delta effects. Also, it is not clear why live load effect on a single element should be made considering the entire floor is of the structure rather than the influence area for the individual element. My negative will be withdrawn is this is fixed to address the area tributary to the individual element.

TS-2 Response: Partially Persuasive

The P-Delta load is consistent, and the influence area has been made permissive, so that a new live load need not be computed.

5. Section 16.3.11 – the statement "at the expected range of deformation" should appear at the end of this paragraph so that it applies to "hysteretic strength degradation as well". Also the "expected range of deformation" should be clarified to include the maximum deformations encountered in response ot any of the ground motions, rather than just the mean, which some may mis-interpret.

TS-2 Response: Editorially Persuasive

6. Section 16.3.13 – the statement that the EOR shall report ' on how viscous damping in the model affects higher mode response" is not clear. I believe the intent is to address the tendency of Rayleigh damping to overdamp higher modes. I believe it is preferable to state that damping can not exceed 5% for any mode required to obtain at least 90% total mass participation.

TS-2 Response: Persuasive

The text is Section 16.3.13 has been modified to clarify the intent.

7. Section 16.4.3.2 – This section is not consistent with Section 16.4.3, which permits element behavior to be considered linear even if demands exceed capacity by a factor of 1.5.

TS-2 Response: Editorially Persuasive

8. Section 16.4.4 – Delete the last sentence. This is confusing and not needed. All elements need to be evaluated – you don't' have to state that Omega zero need not be checked. Omega zero values apply only to linear procedures which are not the subject of this Section.

TS-2 Response: Editorially Persuasive

But also deserves mention in commentary.

Gillengerten (YR): How were the YR votes responded to by the committee?

I support reservations 3a and 3b offered by Harris. The effects of gravity load can significantly alter the demands on structural elements. A single load case will not capture this. Vertical ground motion is also missing. I would change my vote to "Yes" if these issues are addressed in the proposal.

TS-2 Response: Partially persuasive

See response to Holmes on gravity loads and to Bachman on vertical ground motion. The state of the art on these issues is not being moved backwards by this proposal, but we are not able to move as far forward at this time as suggested by the cited comments.

Klingner (No): I agree with the proponents that this proposal is an improvement over the current provisions. I am voting No because I believe that the concerns of Maffei, Charney and Aschheim should be addressed. If TS-2 is really as supportive as they say of this proposal, it would not seem that difficult for them to get a higher percentage of Y votes.

TS-2 Response: Partially persuasive

The comments within TS2 are not all resolvable, because some of them are antithetical. Please see responses to the other comments on the PUC ballot.

Proposal 3-6 (Y=15, YR=2, N=0, NV=5 --100%)

When the proposal was mail balloted to the PUC in July 2008, the PUC responded with 2 YR votes with comments. One of the comments provided a completely edited version of the proposal. It is provided with this text below the Crouse vote. TS 3 took the comments and revised the proposal one more time and this was sent to the PUC in late November, 2008. During the December, 2008 PUC meeting TS 3 used the last revision as the basis to review and discuss. There were additional comments about providing explanation on Geomean and there was agreement to incorporate a comment made by Mr. Robert Hanson. The motion was made to accept the proposal with these revisions and it passed the PUC (19,0,0).

Crouse (YR):

PROPOSAL 3-6 (2009) — Part 3

SCOPE: New Part 3 Proposal for the *2009 Provisions*

[General – Proposal needs editing to make consistent when referencing the specifications and commentary. Headings are also difficult to follow. Can these be made to show level of subheading by changing font size or style.]

PROPOSAL FOR CHANGE:

Add the following new Section in Part 3:

EVALUATION OF GEOLOGIC HAZARDS AND DETERMINATION OF SEISMIC LATERAL EARTH PRESSURES

Summarized below are procedures that are commonly used for evaluating potential site geologic hazards and seismic lateral earth pressures due to earthquakes. The geologic hazards ~~,~~ include~~ing~~ slope instability, liquefaction, ~~differential~~ ground displacement, ~~settlement,~~ and surface fault rupture. ~~Evaluation of the hazard of differential ground settlement is discussed in the section on liquefaction hazard.~~ Geologic hazards evaluations should be carried out by qualified geotechnical professionals and documented in a report. Reporting requirements are given in Provisions Section 11.8. Seismic lateral earth pressure discussions consider both yielding and nonyielding walls.

GEOLOGIC HAZARDS

Screening Evaluation

Evaluation of a seismically-induced geologic hazard may initially consist of a screening evaluation. Although a screening evaluation typically does not require use of detailed analytical procedures, it should be based on detailed site information, including topography, geology, groundwater conditions, subsurface soil and rock stratigraphy and engineering properties, and level of ground shaking. The potential for changes in site conditions over time or as part of site development should be considered. If the findings of a screening evaluation clearly demonstrate the absence of a geologic hazard, then more detailed evaluations, using procedures such as those described in the following sections, need not be conducted. If a screening evaluation cannot demonstrate the absence of a hazard, then the more comprehensive quantitative evaluations described below for the hazards of slope instability, liquefaction, ground displacement~~differential settlement~~, and surface fault rupture should be conducted.

Reference to the following publications is suggested for guidelines on screening evaluations:

- Slope instability: California Geological Survey (1997); Blake et al. (2002); Stewart et al. (2003); U.S. Army Corps of Engineers (2005).
- Liquefaction: Martin and Lew (1999). However, as summarized later in this section under "Recent Updates to the SPT Procedure,", the "Chinese Criteria" for identifying clayey soils susceptible to liquefaction should be abandoned in favor of more recent research.
- Differential Settlement: In the absence of liquefaction, landsliding, or surface fault rupture, differential settlement is generally not a significant hazard except at sites underlain by poorly compacted fills or loose young alluvium.
- Surface fault rupture: U.S. Army Corps of Engineers (2005).

Earthquake Ground Motions for Geologic Hazards Evaluations

The earthquake ground motion parameter generally required for evaluation of the hazards of slope instability, liquefaction, and ~~differential~~ ground displacement ~~settlement~~ is peak ground acceleration, a_{max}. Peak ground acceleration can be determined using the ground motion maps in Chapter 22 and the procedures described in Section~~s 11.4.1 through 11.4.6~~ 11.8.3. ~~Because peak ground acceleration is equal to the zero-period acceleration of a response spectrum, a design~~

~~value of a_{max} is approximated as 0.4 x SDS from Eq. 11.4-5. The value of a_{max} can be determined more accurately for a probability of exceedance of 2% in 50 years by using the USGS national ground motion map website (http://earthquake.usgs.gov/research/hazmaps/) and using the option to obtain values of peak ground acceleration for specified latitudes and longitudes. The value of a_{max} so obtained should be adjusted for site class by multiplying by the site coefficient F_a (Section 11.4.3) and then multiplying by a factor of 2/3 to convert from the maximum considered earthquake (MCE) level to the design earthquake level. The procedure using the USGS web site is not applicable, however, to locations in the U.S. near highly active faults where probabilistic ground motions have been deterministically bounded to lower values as described in Commentary Appendix A of the 2003 NEHRP Provisions.~~Alternatively, peak ground acceleration can be obtained from a site-specific study conforming to the requirements of Section 11.4.7 and Chapter 21.

The evaluation of geologic hazards should be made for site peak ground accelerations, earthquake magnitude, and source characteristics consistent with the maximum considered earthquake (MCE) as required by Provisions Section 11.8.

~~Although the basic evaluations for geologic hazards may be conducted for design earthquake ground motions, it is recommended that evaluations also be conducted for maximum considered earthquake (MCE) ground motions (equal to 1.5 times design ground motions) for use in checking for effects that may result in structure collapse. In addition, Seismic Use Group III structures should be checked for this required post-earthquake condition. If slope instability, liquefaction, or large settlements are predicted for the MCE, the hazard should be brought to the attention of the designer and owner, and a decision made as to the risk of the hazard and the need for mitigation following the methods discussed below.~~

Slope Instability Hazard

When subjected to earthquake-induced ground shaking, sloping ground can pose a hazard to structures located on or in proximity to a slope. The potential severity of the hazard depends on the steepness of the slope, soil and groundwater conditions within the slope, the strength and duration of ground shaking, and the potential consequences of slope movement. In some situations acceptable slope movement can be on the order of feet, whereas in other situations – particularly where buildings are involved – movements of more than a few inches may be unacceptable. A critical first step in the assessment of the slope instability hazard is, therefore, to establish the performance criteria for the structure. Normally this requires detailed discussions between the geotechnical engineer and the structural designer and with the project owner.

Pseudo Static Method of Analysis. The stability of slopes composed of dense (nonliquefiable) or nonsaturated sandy soils or nonsensitive clayey soils can be determined using either pseudo static- or deformation-based procedures. For initial evaluations, the pseudo static analysis may be used, although the deformational analysis described in the next section is now preferred.

In the pseudo static analysis, inertial forces generated by earthquake shaking are represented by an equivalent static horizontal force acting on the slope. The seismic coefficient for this analysis should be the site peak ground acceleration, a_{max}. The vertical component of ground acceleration is normally assumed to be zero during this representation. The factor of safety for a given seismic coefficient can be estimated by using traditional slope stability calculation methods. A factor of safety greater than 1.0 ~~one~~ indicates that the slope is stable for the given lateral force level and further analysis is not required. A factor of safety of less than 1.0 ~~one~~ indicates that the slope will yield and slope deformation can be expected, and a deformational analysis should be made using the techniques discussed below.

A common practice when using the pseudo static method is to reduce the peak ground acceleration by a factor to account for the transitory nature of the ground motions. The factor used in this reduction is often selected as 0.5 but lower reduction factors have also been used. For these analyses the acceptable factor of safety is often taken as 1.1 to 1.3. Implicit within this approach is that deformation of the slope is acceptable. The amount of deformation can range from a few inches to several feet (Blake et al., 2002). Movements of this magnitude are not normally acceptable for building design. For this reason the recommended approach in this guideline is to use the full peak ground acceleration in the pseudo static analysis, and then if the resulting factor of safety is less than 1.0, a deformational analysis is conducted.

When conducting a pseudo static stability analysis, two key assessments must be made by the designer during the set-up of the stability model:

- An accurate characterization of the site must be developed. This characterization needs to consider the final slope geometry, the soil types and layering within the slope, and groundwater conditions likely to exist during the seismic event. The existence of thin soil layers that could serve as slip planes is particularly important in the characterization process.
- The appropriate soil strength to use for the seismic analyses must also be selected. This determination will depend on various factors, including whether the soil is fine- or coarse-grained alluvium, the effective stress conditions, the degree of saturation of the material, and the stress history for the soil. In most situations the undrained strength of the soil is appropriate because of the short duration of seismic loading. Blake et al. (2002) provide important guidance on the use of drained or undrained soil properties, the appropriate type of testing, the use of peak versus residual strengths, and whether reductions in strength are appropriate to account for the effects of loading rate and repeated cycles of load.

For sites where soils could liquefy or where sensitive soils are known to occur, special studies will be required. If liquefaction is predicted under the design seismic event, the ~~residual~~ strength of the soil in a liquefied state should ~~normally~~ be used in the pseudo static stability analyses. Additional discussions of the ~~residual~~ strength of liquefied soils are presented later in the liquefaction hazard section. If sensitive clayey soils exist, special laboratory tests may be required to establish the amount of degradation in soil strength that will occur with cyclic loading.

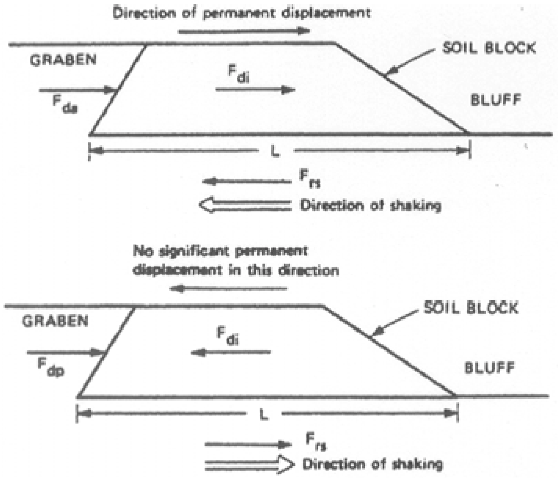
Deformational Methods of Analysis. Deformational analyses resulting in estimates of slope displacement are now accepted practice. The most common analysis, termed a Newmark analysis (Newmark, 1965), uses the concept of a frictional block sliding on a sloping plane or arc. In this analysis, seismic inertial forces are calculated using a time history of horizontal acceleration as the input motion. Slope movement occurs when the driving forces (gravitational plus inertial) exceed the resisting forces. This approach estimates the cumulative displacement of the sliding mass by integrating increments of movement that occur during periods of time when the driving forces exceed the resisting forces. Displacement or yield occurs when the earthquake ground accelerations exceed the acceleration required to initiate slope movement or yield acceleration.

The yield acceleration depends primarily on the strength of the soil and the gradient and height and other geometric attributes of the slope. The same comments on the characterization of the slope and soil strength given above for the pseudo static analysis apply for the deformational analysis, though consideration can be given to the modification of strength with cycles of earthquake loading. See Figure C11.8-1 for forces and equations used in analysis and Figure C11.8-2 for a schematic illustration for a calculation of the displacement of a soil block toward a bluff.

Two methods are commonly used to estimate slope displacements by the Newmark method. The more rigorous approach involves use of earthquake records that will be representative of expected ground shaking at the site during the design seismic event. These records need to be scaled to be consistent with the design response spectra adjusted for site response effects. If more than one characteristic source mechanism contributes to the earthquake hazard, it may be necessary to select sets of records that are characteristic of each source mechanism. In this case multiple potential sources are considered because of the dependency of slope displacement on earthquake magnitude or duration; that is~~i.e.~~, a large distant earthquake may result in lower peak ground acceleration but longer duration of shaking, which potentially could result in more cumulative deformation than a nearby earthquake of higher peak ground acceleration but short duration. Either computer programs (e.g., Jibson, 1993) or spreadsheet methods can be used to determine the cumulative displacement from the earthquake records.

An acceptable alternative method for the determination of displacements on many projects involves the use of charts or simplified equations that show or estimate displacements for different acceleration ratios, where the acceleration ratio is defined as the ratio of yield acceleration to peak ground acceleration. These charts and equations have been developed by calculating the cumulative displacement following the Newmark method for large sets of earthquake records. The charts include those by Franklin and Chang (1977), Makdisi and Seed (1978), Wong and Whitman (1982), Hynes and Franklin (1984), Martin and Qiu (1994); Bray and Rathje (1998), Bray et al. (1998), and Jibson (2007). Simple equations include those by Bray and Travasarou (2007), Jibson (2007), Saygili and Rathje (2008), and Rathje and Saygili. (2008). ~~and Blake et al. (2002).~~ Figure 11.8-3 shows the simplified chart from Bray et al. (1998). ~~Blake et al. (2002).~~ The D_{5-95} term in this figure is the significant duration of shaking – with its relationship differing depending on whether the site is within or greater than 10 km from the earthquake source. The selection between the different charts should be made on the basis of the type of slope and the degree of conservatism necessary for the project. It is important to recognize that when using one of the charts, or the Newmark method in general, a number of simplifying assumptions are made regarding the behavior of the soil during seismic loading. These assumptions limit the accuracy to which the deformations can be estimated. Generally, these methods do~~es~~ not justify estimating deformations to less than a few inches.

Slope deformations can also be estimated by using more rigorous two-dimensional computer modeling methods. The computer code FLAC (Itasca, 1997) is perhaps the most common of the programs being used by practitioners for evaluating the response of slopes to seismic loads. This computer program allows various soil geometries, soil layering, and groundwater conditions to be modeled. Earthquake records representative of the design seismic event are used to conduct the time history analysis. Results provide an understanding of the development of deformations with time, the location of critical surfaces of deformation, and the effects of pore water pressure buildup on slope movement. As with any rigorous model, the accuracy of the deformation estimate is critically dependent on the properties and geometry of the model, as well as the earthquake record selection.



F_{da} = driving force due to active soil pressure

F_{di} = driving force due to earthquake inertia

F_{rs} = resisting force due to soil shear strength

F_{dp} = resisting force due to passive soil pressure

$$F_{di} = K_{\max} W$$

where K_{max} = maximum seismic coefficient and W = weight of soil block

$$F_{rs} = S_u L$$

where S_u = average undrained shear strength of soil and L = length of soil block

Yield seismic coefficient:

$$K_y = \frac{F_{rs} - F_{da}}{W}$$

Figure 11.8-1 Forces and equations used in analysis of translatory landsildes for calculating permanent lateral displacements from earthquake ground motions (National Research Council, 1985; from Idriss, 1985)

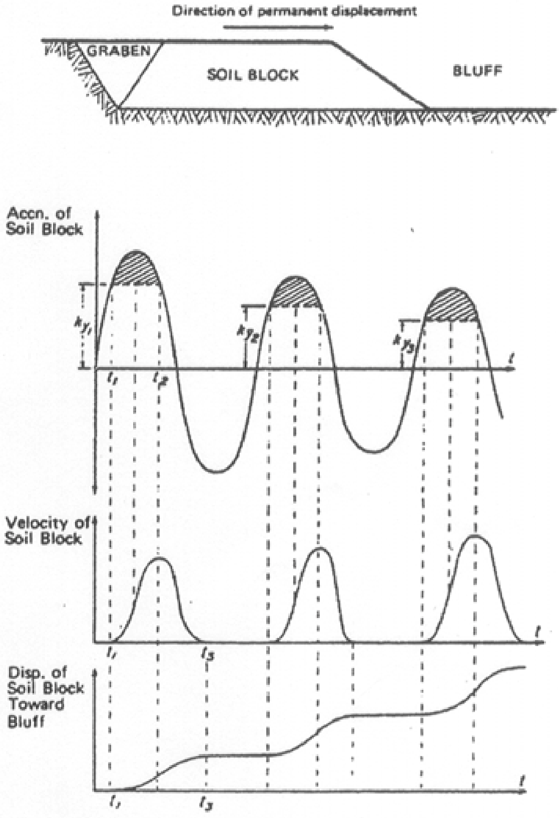


Figure 11.8-2 Schematic illustration for calculating displacement of soil block toward the bluff (National Research Council, 1985; from Idriss, 1985, adapted from Goodman and Seed, 1966)

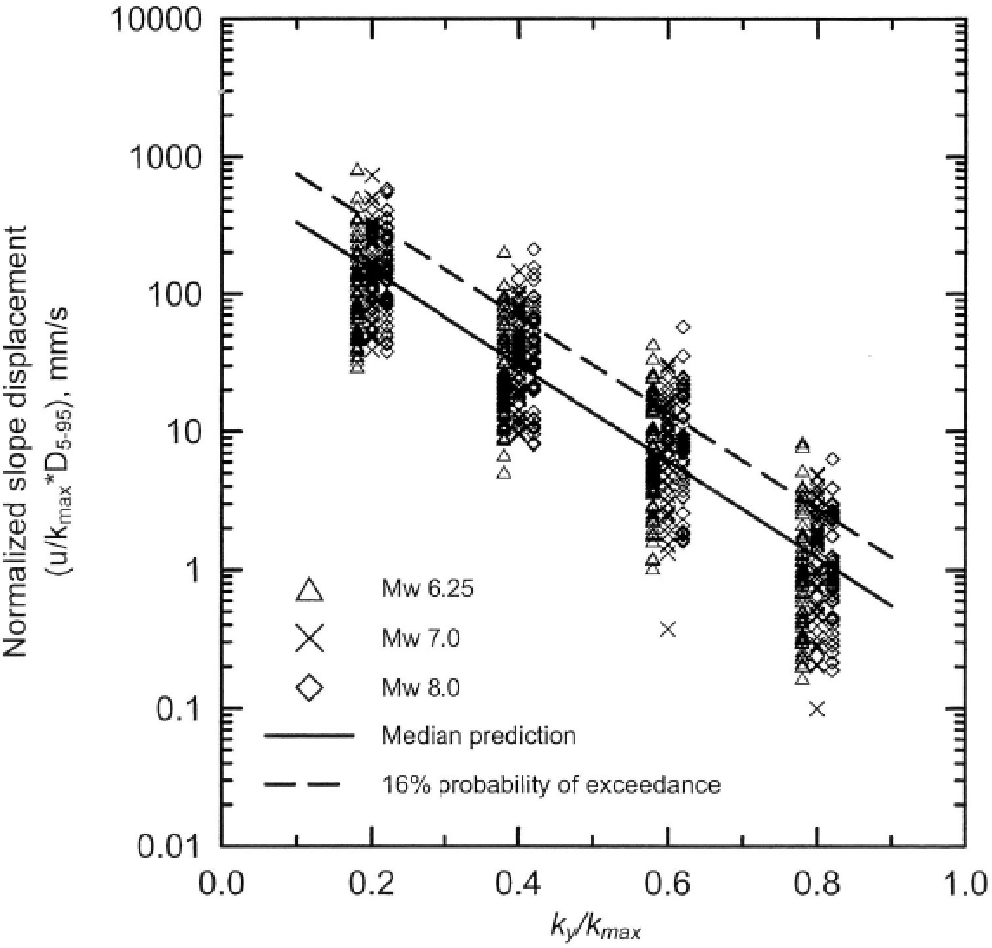


Figure C11.8-3 Normalized Sliding Displacement (Bray et al., 1998~~lake et al., 2002~~).

Mitigation of Slope Instability Hazard. Three general mitigative measures might be considered for locations where slope instability is determined to represent a hazard: (1) design the structure to resist the hazard, (2) stabilize the site to reduce the hazard, or (3) choose an alternative site. Ground displacements generated by slope instability are similar in destructive character to fault displacements generating similar senses of movement: compression, shear, extension or vertical. Thus, the general comments on structural design to prevent damage given under mitigation of fault displacement apply equally to slope displacement.

Techniques to stabilize a site include ~~reducing the driving forces by grading and drainage of slopes and~~ increasing the resistance of the soil to displacement ~~ing forces~~ by subsurface drainage, buttresses, retaining walls, ground anchors, reaction piles or shafts, ground improvement using densification or soil mixing methods, or chemical treatment. Addition details for these mitigation methods can be found in various reports, including Blake et al. (2002).

Liquefaction Hazard

Liquefaction forms the second and, perhaps, the most widely known geologic hazard that must be considered at a building site. This hazard~~s~~ occurs when earthquake-induced ground shaking results in loss of strength within water-saturated, loose granular soils. The consequence of this strength loss relative to a building can be reduction in bearing strength, total and differential settlement, and horizontal ground displacement from lateral spreading or flow failures within the ground. In this section, the hazard of differential settlement, whether due to liquefaction of water-saturated soils or compaction of non-saturated soils, is addressed.

Design to prevent damage due to liquefaction consists of three parts: evaluation of liquefaction hazard, evaluation of potential ground displacement, and where necessary mitigation of the hazard by ~~either~~ designing to resist either ground displacement or strength loss, by reducing the potential for liquefaction, or by choosing an alternative site with less hazard. Before providing guidance in these areas, the following subsections provide a summary of the methods that are used to evaluate the liquefaction hazard and a discussion of recent updates to the most commonly used method of assessing a liquefaction hazard – the empirical Standard Penetration Test (SPT) procedure.

Methods of Liquefaction Hazard Evaluation. Liquefaction hazard at a site is commonly expressed in terms of a factor of safety. This factor is defined as the ratio between the available liquefaction resistance, expressed in terms of the cyclic stresses required to cause liquefaction, and the cyclic stresses generated by the design earthquake. Both of these stress parameters are commonly normalized with respect to the effective overburden stress at the depth in question to define a cyclic resistance ratio (CRR) and a cyclic stress ratio induced by the earthquake (CSR). Three different methods have been proposed and are used to various extents for evaluating liquefaction potential: empirical methods; analytical methods; and physical modeling.

1. Empirical methods are the most widely used methods in practice. These procedures rely on correlations between observed cases of liquefaction and measurements made in the field with conventional exploration methods. Seed and Idriss (1971) first published the widely used "simplified procedure" utilizing the Standard Penetration Test (SPT). Since then, field test methods in addition to the SPT have been utilized in similar simplified procedures. These methods include cone penetrometer tests (CPTs), Becker hammer tests (BHTs), and shear wave velocity tests (SVTs). These empirical procedures are summarized in the proceedings from a workshop (referred to as the Liquefaction Workshop) held in 1996 (NCEER, 1997; Youd et al., 2001). Martin and Lew (1999) provide additional details on the implementation of these procedures relative to engineering practice.

2. Analytical methods are used less frequently to evaluate liquefaction potential – though they may be required for special projects or where soil conditions are not amenable to the empirical method. Analytical methods will also likely continue to gain prominence with time as numerical methods and soil models improve and are increasingly validated. Originally (circa 1970s) the analytical method involved determination of the induced shearing stresses with a program such as SHAKE and comparing these stresses to results of cyclic triaxial or cyclic simple shear tests. Now the analytical method usually refers to a computer code that incorporates a soil model that calculates the buildup in pore water pressure. These more rigorous numerical methods include one-dimensional, nonlinear effective stress codes such as DESRA, DEEPSOIL, DMOD, SUMDES, and TESS and two dimensional, nonlinear effective stress codes such as FLAC, TARA, and DYNAFLOW. This new generation of analytical methods has soil models that are fit to or derived from laboratory data or from liquefaction curves developed from SPT or other field information. These methods are limited by the ability to represent the soil model from either the laboratory or field measurements and by the complexity of the wave propagation mechanisms, including the ability to select appropriate earthquake records to use in the analyses.

3. Physical modeling originally involved the use of centrifuges or relatively small-scale shaking tables to simulate seismic loading under well-defined boundary conditions. Physical model

testing also now includes large laminar boxes mounted on very large shake tables (e.g., Kagawa et al., 2004) and full-scale field blast loading tests (e.g., Ashford et al. 2004; Ashford et al., 2006). This type of modeling is one of the main focus areas of the 2004-2014 Network for Earthquake Engineering Simulation (NEES) supported by the National Science Foundation. Soil used in the small-scale and laminar box models is reconstituted to represent different density and geometrical conditions. Because of difficulties in precisely modeling in situ conditions at liquefiable sites, small-scale and laminar box models have seldom been used in design studies for specific sites. However, physical models are valuable for analyzing and understanding generalized soil behavior and for evaluating the validity of constitutive models under well-defined boundary conditions. Blast loading tests have been conducted to capture the in situ characteristics of the soil for research and design purposes (e.g., Treasure Island, California; Cooper River Bridge in South Carolina, and in Japan). However, the cost and safety issues of blasting methods limit~~s~~ its use to only special design or research projects.

Most liquefaction hazards assessments for buildings will involve use of the SPT empirical method – partly because of the wide acceptance of this approach and also because this approach can be easily integrated into the geotechnical investigations normally performed during building design. The SPT method is based on recommendations developed at the Liquefaction Workshop as described in NCEER (1997) and Youd et al. (2001) or on one of the updates to this methodology as discussed below.

Although the SPT empirical method is the most commonly used of the empirical approaches, it is important to recognize that for certain site conditions alternate empirical methods, such as the CPT, BHT, and SVT, are acceptable and even preferred. This is particularly the case with the CPT method. Advantages of the CPT method compared to the SPT method are the ability of this method to detect thin liquefiable layers that could serve as sliding surfaces and the greater standardization of the method – though this approach has the disadvantage that soil samples are not obtained. Where possible a combination of procedures is recommended to take advantage of the best features of each.

Recent Updates to the SPT Procedure. The methods presented in the Liquefaction Workshop and summarized in the following section represent a consensus-based approach for the onset or triggering of liquefaction; however, the consensus workshop occurred over~~nearly~~ 10 years ago. A number of significant modifications to the methods presented in the Liquefaction Workshop have been recommended over the past 10 years. These modifications include changes to the stress reduction coefficient (r_d), modifications to the magnitude scaling factor (MSF) [also referred to as the duration weighting factor (DWF)], revisions to the overburden correction term (K_σ) and the fines correction (FC), refinements to the overburden correction (C_N), and finally changes to the relationship between the curve relating cyclic stress ratio causing liquefaction and the normalized N value; that is~~i.e.~~, the fundamental liquefaction strength curve such as shown in Figure C11.8-4. These modifications are discussed in detail in papers by Cetin et al. (2004), Idriss and Boulanger (2004, 2006), ~~and Cetin et al. (2004)~~and Moss et al. (2006);~~,~~ ~~and~~ each set of recommended revisions resulted after detailed study of the database of case histories upon which the original blowcount relationships were developed.

Another important observation that has been made over the past 10 years involves the fines criteria used to judge whether or not a soil is liquefiable. Originally, the "Chinese Criteria" was accepted as the method to determine whether or not a cohesionless soil was liquefiable. However, recent work summarized in Boulanger and Idriss (20064) and Bray and Sancio (2006) ~~and Bray et al. (2004)~~ indicate that the Chinese Criteria will be unconservative in some situations, and alternate methods of assessing whether a soil with cohesive fines will be susceptible to liquefaction or cyclic strength reduction need to be considered. The methods recommended by Boulanger and Idriss (2006) and Bray and Sancio (2006) also establish whether the simplified empirical field methods described previously should be used to estimate liquefaction potential or whether other methods, such as laboratory testing, may be more suitable for evaluating the effects of cyclic loading on soil strength.

Methods are also now available for treating the probability of liquefaction, given a certain design ground motion and SPT blowcount. Cetin et al. (2004) present~~s~~ a ~~the latest~~ comprehensive treatment of liquefaction probability. These researchers suggest that following the deterministic approach for estimating liquefaction potential, as discussed above, results in approximately 15 percent probability of liquefaction. The approach presented by Cetin et al. (2004) allows limiting SPT blowcounts to be determined for alternate probabilities, or the probability associated with a given set of blowcounts and ground motions (in terms of CSR) to be defined. Kramer and Mayfield (2007) show how the probability of ground shaking can be combined with the probability of liquefaction in a performance-based approach to evaluating liquefaction potential. This probabilistic framework forms an important basis for performance-based design methods that are currently being developed.

Despite these many important modifications to the general approach for assessing liquefaction hazards over the past 10 years, the profession has not developed a consensus on which of the modifications should be used as a baseline for evaluating liquefaction hazard – similar to the recommendations in NCEER (1997) and Youd et al. (2001) based on the Liquefaction Workshop. Procedures suggested by Idriss and Boulanger (2004), as well as those developed by Cetin et al. (2004) and Moss et al. (2006), present important changes to the liquefaction hazard analysis. However, until a consensus is reached or an adequate period of vetting occurs, it is difficult to recommend between the two. .

In using the more recent methods, it is important that these methods be used consistently. In other words the Idriss and Boulanger method should be used with the various improvements recommended by Idriss and Boulanger, including the revised liquefaction strength plot. Likewise, if the Cetin et al. method is going to be used, it should be used in its entirety. It is also important to use these new methods with some caution, particularly at the limits of the procedure (e.g., at higher blowcounts, deeper depths, and higher CSR values). If the more recent methods are used, the prudent approach will be to check the liquefaction hazard with an alternate method, such as the procedure discussed below. Differences between the hazard estimates resulting from different methods could reflect a real uncertainty in the prediction, and this uncertainty would need to be considered when judging the hazard at a site.

Empirical SPT Method for Evaluating Liquefaction Hazard. Procedures for evaluating the liquefaction hazard using the Liquefaction Workshop methodology are summarized below. As discussed above, the recent changes in the methodology proposed by Idriss and Boulanger (2004) and Cetin et al. (2004) offer an updated alternative to this approach, but need to be used with some caution in the absence of a consensus from the profession that these newer methods are acceptable for generalized use.

1. The first step in the liquefaction hazard evaluation using the empirical SPT approach is ~~usually~~ to define the normalized cyclic shear stress ratio (CSR) from the peak horizontal ground acceleration expected at the site. This evaluation is made using the following equation:

   $$CSR = 0.65\left(a_{\max}/g\right)\left(\sigma_0/\sigma_0'\right)r_d \qquad \text{(C11.8-1)}$$

   Where:

   (a_{max}/g) = peak horizontal acceleration at ground surface expressed as a decimal fraction of gravity, σ_0 = the vertical total stress in the soil at the depth in question, σ_0' = the vertical effective stress at the same depth, and r_d = deformation-related stress reduction factor. The peak ground acceleration, a_{max}, commonly used in liquefaction analysis is that which would occur at the site in the absence of liquefaction. Thus, the a_{max} used in Eq. C11.8-1 is the estimated rock acceleration corrected for soil site response but with neglect of excess pore-water pressures that might develop.

The stress reduction factor, r_d, used in Eq. C11.8-1 was originally determined using a plot developed by Seed and Idriss (1971) showing the reduction factor versus depth. The consensus from the Liquefaction Workshop was to represent r_d by the following equations:

$$r_d = 1.0 - 0.00765z \quad \text{for } z \leq 9.15 \text{ m} \tag{C11.8-2a}$$

$$r_d = 1.174 - 0.267z \quad \text{for } 9.15\ m < z \leq 23 \text{ m} \tag{C11.8-2b}$$

It should be noted that because nearly all the field data used to develop the simplified procedure are for depths less than 50 ft~~15 m~~, there is greater uncertainty in the use of the SPT empirical approach at greater depths. Common practice is to use the SPT method to depths of 75 ft. ~~25 m.~~ In some locations deep deposits of low blowcount or low CPT end resistance values occur, such as in the Puget Sound area and along the Columbia River. It is still prudent to consider these low blowcount materials as susceptible to liquefaction even if they are located at depths greater than 75 ft~~25 m~~. For these sites it may be appropriate to use strain-based procedures (Dobry et al., 1982) or one-dimensional, effective stress modeling methods such as are summarized in Commentary to Chapter 21.

2. The second step in the liquefaction hazard evaluation using the empirical approach ~~usually~~ involves determination of the normalized cyclic resistance ratio (CRR). The most commonly used empirical relationship compares CRR with corrected Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, from sites where liquefaction did or did not develop during past earthquakes. Figure C11.8-4 shows this relationship for Magnitude 7.5 earthquakes, with an adjustment at low values of CRR recommended by the Liquefaction Workshop. Similar relationships have been developed for determining CRR from CPT soundings, from BHT blowcounts, and from shear wave velocity data, as discussed by Youd et al. (2001) and as presented in detail in NCEER (1997).

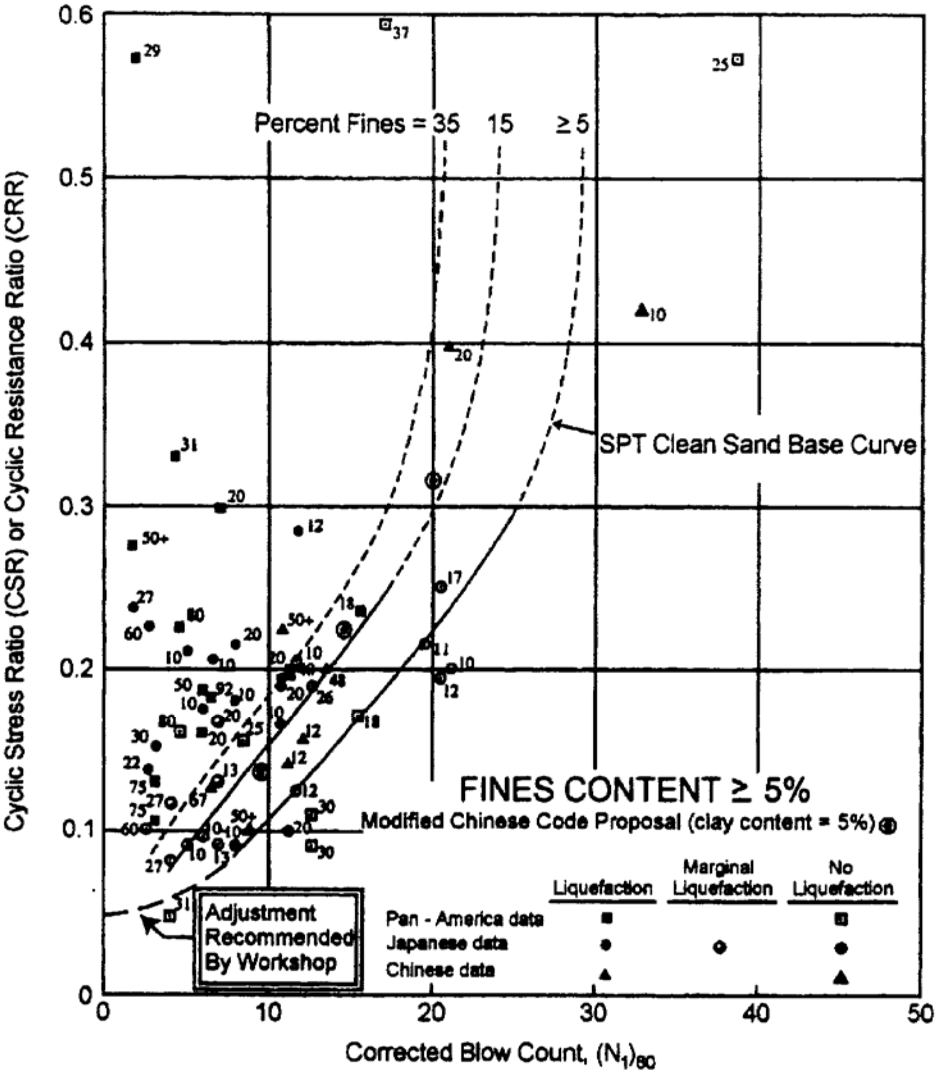


Figure C11.8-4. SPT clean sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories. (Modified from Seed et al., 1985). (NCEER, 1997; Youd et al., 2001).

In Figure C11.8-4, CRRs calculated for various sites are plotted against $(N_1)_{60}$, where $(N_1)_{60}$ is the SPT blowcount normalized for an overburden stress of 100 kPa and for an energy ratio of 60 percent. Solid symbols represent sites where liquefaction occurred and open symbols represent sites where surface evidence of liquefaction was not found. Curves were drawn through the data to separate regions where liquefaction did and did not develop. As shown, curves are given for soils with fines contents (FC) ranging from less than 5 to 35 percent.

The $(N_1)_{60}$ in Figure C11.8-4 is adjusted for various factors before its use, as recommended by the Liquefaction Workshop and discussed by Youd et al. (2001). These include an adjustment for fines, such that only the clean sand curve in Figure C11.8-4 is used, as well as adjustments for a number of other testing related parameters. These adjustments are not repeated in this guideline as they are all in conventional use by the profession and can readily be found in references by Martin and Lew (1999) and by Youd et al. (2001).

It is very important that the engineer consider these correction factors when conducting the liquefaction analyses. Failure to consider these corrections can result in inaccurate liquefaction estimates – leading to either excessive cost to mitigate the liquefaction concern or excessive risk of poor performance during a design event – potentially resulting in unacceptable damage.

Special mention needs to be made of the energy calibration term, C_E. This correction has a very significant effect on the $(N_1)_{60}$ used to compute CRR. The value of this correction factor can vary greatly depending on the SPT hammer system used in the field and on site conditions. The automatic hammer used to conduct SPTs in modern-day explorations avoids much of the uncertainty in energy; however, even it should be periodically calibrated. These calibration measurements are relatively inexpensive and represent a small increase in overall field exploration costs. Many drilling contractors in areas that are seismically active provide calibrated equipment as part of their routine service.

Before computing the factor of safety from liquefaction, the CRR result obtained from Figure C11.8-4 (using the corrected SPT blow count identified in the equation for $(N_1)_{60}$) must be corrected for earthquake magnitude M if the magnitude differs from 7.5. The magnitude correction factor is shown in Figure C11.8-5. This plot was developed during the Liquefaction Workshop on the basis of input from experts attending the workshop. The range shown in Figure C11.8-5 is used because of uncertainties. The user should select a value consistent with the project risk. For M greater than 7.5 the factors recommended by Idriss (second from highest) should be used.

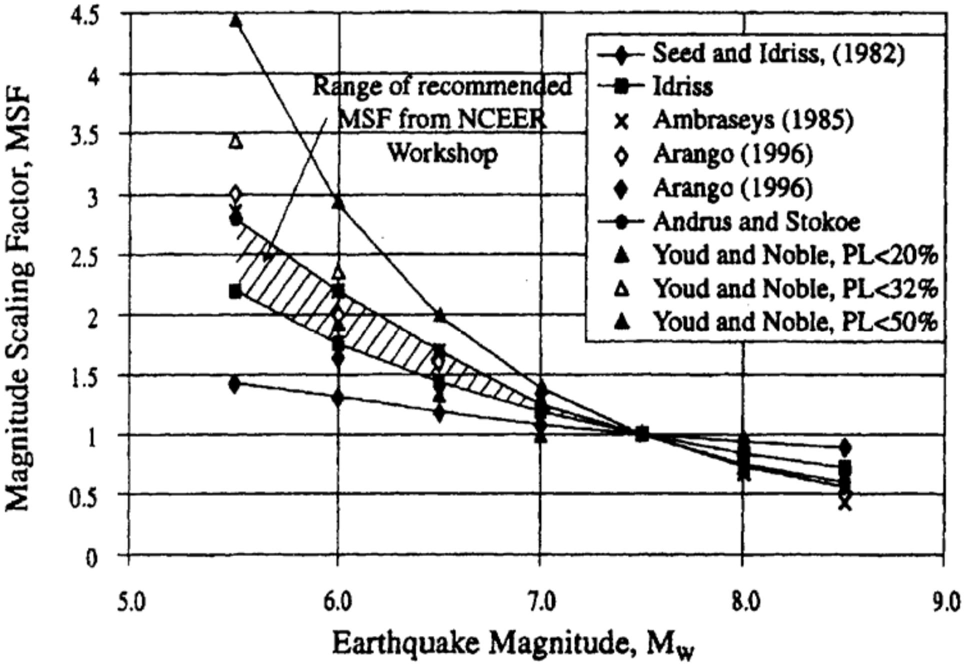


Figure C11.8-5. Magnitude scaling factors derived by various investigators. (NCEER, 1997; Youd et al., 2001).

The magnitude, M, needed to determine a magnitude scaling factor from Figure C11.8-5 should correspond to the Maximum Considered Earthquake (MCE). Where the general procedure for ground motion estimation is used (Sections 11.4.1 - 11.4-6) and the MCE is determined probabilistically, the magnitude used in these evaluations can be obtained as the dominant magnitude(s) determined from deaggregation information available by latitude and longitude from the USGS website (http://earthquake.usgs.gov/research/hazmaps/). Where the general procedure is used and the MCE is bounded deterministically near known active fault sources, the magnitude of the MCE should be the characteristic maximum magnitude assigned to the fault in the construction of the MCE ground motion maps. Where the site-specific procedure for ground motion estimation is used (Sections 11.4.7 and Chapter 21), the magnitude of the MCE should be similarly determined from the site-specific analysis. In all cases, it should be remembered that the likelihood of liquefaction at the site (as defined later by the factor of safety F_L in Eq. C11.8-3) is determined jointly by a_{max} and M and not by a_{max} alone. Because of the longer duration of strong ground-shaking, large distant earthquakes may in some cases generate liquefaction at a site while smaller nearby earthquakes may not generate liquefaction even though a_{max} of the nearer events is larger than that from the more distant events.

3. The final step in the liquefaction hazard evaluation using the empirical SPT approach involves the computation of the factor of safety (F_L) against liquefaction using the equation:

   $$F_L = \text{CRR/CSR} \qquad \text{(C11.8-3)}$$

   If F_L is greater than 1.0~~one~~, then liquefaction should not develop. If at any depth in the sediment profile, F_L is equal to or less than 1.0~~one~~, then there is a liquefaction hazard. Although the curves shown in Figure C11.8-4 envelop the plotted data, it is possible that liquefaction may have occurred beyond the enveloped data and was not detected at ground surface. For this reason a factor of safety of 1.1~~2~~ to 1.3~~5~~ is usually appropriate for building sites – with the actual factor selected on the basis of the importance of the structure and the potential for ground displacement at the site.

   Additional guidance on the selection of the appropriate factor of safety is provided by Martin and Lew (1999). They suggest that the following factors be considered when selecting the factor of safety:

   - The type of structure and its vulnerability to damage.
   - Levels of risk accepted by the owner or governmental regulations with questions related to design for life safety, limited structural damage, or essentially no damage.
   - Damage potential associated with the particular liquefaction hazard. Flow failures or major lateral spreads pose more damage potential than differential settlement. Hence factors of safety could be adjusted accordingly.
   - Damage potential associated with design earthquake magnitude. A magnitude 7.5 event is potentially more damaging than a 6.5 event.
   - Damage potential associated with SPT values; low blow-counts have a greater cyclic strain potential than higher blowcounts.
   - Uncertainty in SPT- or CPT- derived liquefaction strengths used for evaluations. Note that a change in silt content from 5 to 15 percent could change a factor of safety from, say, 1.0 to 1.25.
   - For high levels of design ground motion, factors of safety may be indeterminate. For example, if $(N_1)_{60})$ = 20, M = 7.5, and fines content = 35 percent, liquefaction strengths cannot be accurately defined due to the vertical asymptote on the empirical strength curve.

Martin and Lew (1999) indicate that the final choice of an appropriate factor of safety must reflect the particular conditions associated with the specific site and the vulnerability of site-related structures. Table C11.8-1 summarizes factors of safety suggested by Martin and Lew.

Table C11.8-1. Factors of safety for liquefaction hazard assessment (from Martin and Lew, 1999).

Consequences of Liquefaction	$(N_1)_{60})_{cs}$	Factor of Safety
Settlement	≤ 15	1.1
	≥ 30	1.0
Surface Manifestations	≤ 15	1.2
	≥ 30	1.0
Lateral Spread	≤ 15	1.3
	≥ 30	1.0

As a final comment on the assessment of liquefaction hazards, it is important to note that soils composed of sands, silts, and gravels are most susceptible to liquefaction while clay soils generally are not susceptible to this phenomenon. The curves in Figure C11.8-4 are valid for soils composed primarily of sand. The curves should be used with caution for soils with substantial amounts of gravel. Verified corrections for gravel content have not been developed; a geotechnical engineer, experienced in liquefaction hazard evaluation, should be consulted when gravelly soils are encountered.

Evaluation of Potential for Loss of Ground Support, Increased Loads, and Ground Displacements. Liquefaction by itself may or may not be of engineering significance. Only when liquefaction is accompanied by loss of ground support, increased soil loads, and/or ground deformation does this phenomenon become important to structural design. Surface manifestations, loss of bearing strength, increased lateral earth pressures, ground settlement, flow failure, and lateral spread are ground failure mechanisms that have caused structural damage during past earthquakes. These types of ground failure are described in Martin and Lew (1999), U.S. Army Corps of Engineers (2005), and National Research Council (1985) and are discussed below. The type of failure and amount of ground displacement are a function of several parameters including the looseness of the liquefied soil layer, the thickness and extent of the liquefied layer, the thickness and permeability of unliquefied material overlying the liquefied layer, the ground slope, and the nearness of a free face.

*Surface Manifestations***.** Surface manifestations refer to sand boils and ground fissures on level ground sites. For structures supported on shallow foundations, the effects of surface manifestations on the structure could be tilting or cracking. Criteria are given by Ishihara (1985) for evaluating the influence of thickness of layers on surface manifestation of liquefaction effects for level sites. These criteria may be used for noncritical or nonessential structures on level sites not subject to lateral spreads (see later in this section). Additional analysis should be performed for critical or essential structures.

Loss of bearing strength. Loss of bearing strength can occur if the foundation is located within or above the liquefiable layer. The consequence of bearing failure could be settlement or tilting of the structure. Usually, loss of bearing strength is not likely for light structures with shallow footings founded on stable, nonliquefiable materials overlying deeply buried liquefiable layers, particularly if the liquefiable layers are relatively thin. Simple guidance for how deep or how thin the layers must be has not yet been developed. Martin and Lew (1999) provide some preliminary guidance based on the Ishihara (1985) method. Final evaluation of the potential for loss of bearing strength should be made by a geotechnical engineer experienced in liquefaction hazard assessment.

Increased lateral earth pressure. Another possible consequence of liquefaction ~~to structures~~ is increased lateral pressures against basement and retaining walls.~~walls as discussed in Section 11.8-3~~. A common procedure used in design for such increased pressures is to assume that the liquefied material acts as a dense fluid having a unit weight of the liquefied soil. The wall then is designed assuming that hydrostatic pressure for the dense fluid acts against~~along the total subsurface height of~~ the wall. If unsaturated soil is present above the liquefied soil, it is treated as a surcharge that increases the fluid pressure within the underlying liquefied soil by an amount equal to the thickness times the total unit weight of the surcharge soil. The procedure applies equivalent horizontal earth pressures that are greater than typical at-rest earth pressures but less than passive earth pressures. As a final consideration, to prevent buoyant rise of a structure as a consequence of liquefaction, the total weight of the structure should be greater than the volume of the basement or other cavity times the unit weight of liquefied soil. (Note that structures with insufficient weight to counterbalance buoyant effects could differentially rise during an earthquake.)

Ground settlement. For saturated or dry granular soils in a loose condition, the amount of ground settlement can approach 3 to 4 percent of the thickness of the loose soil layer in some cases. This amount of settlement could cause tilting or cracking of a building, and therefore, it is usually important to evaluate the potential for ground settlement during earthquakes.

Tokimatsu and Seed (1987) published an empirical procedure for estimating ground settlement. It is beyond the scope of this discussion ~~commentary~~ to outline that procedure which, although explicit, has several rather complex steps. The Tokimatsu and Seed procedure can be applied whether liquefaction does or does not occur. For dry cohesionless soils, the settlement estimate from Tokimatsu and Seed should be multiplied by a factor of 2 to account for multi-directional shaking effects as discussed by Martin and Lew (1999). An alternate approach is that proposed by Ishihara and Yoshimine (1992). It also can be used for saturated and unsaturated soils.

*Flow failures***.** Flow failures or flow slides are the most catastrophic form of ground failure that may be triggered when liquefaction occurs. They may displace large masses of soils tens of feet~~meters~~. Flow slides occur when the average static shearing stresses on potential failure surfaces are less than the average shear strengths of liquefied soil on these surfaces. Standard limit equilibrium static slope stability analyses may be used to assess flow failure potential with the residual strength of liquefied soil used as the strength parameter in the analyses.

The determination of residual strengths is very inexact, and consensus as to the most appropriate approach has not been reached to date. ~~Two r~~Relationships for residual strength of liquefied soil that are often used in practice are those of Seed and Harder (1990), Olson and Stark (2002), and Idriss and Boulanger (2007). These strengths have been empirically determined from back analyses of flow failures. ~~and Stark and Mesri (1992). A more complete discussion and references on this topic may be found in Martin and Lew (1999).~~

Lateral spreads. Lateral spreads are ground-failure phenomena that can occur on gently sloping ground underlain by liquefied soil. They may result in lateral movements in the range of a few inches ~~centimeters~~ to several feet~~meters~~. Earthquake ground-shaking affects the stability of gently sloping ground containing liquefiable materials by seismic inertial forces combined with static gravity forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Temporary instability due to seismic inertial forces is~~are~~ manifested by lateral "downslope" movement. For the duration of ground shaking associated with moderate- to large-magnitude earthquakes, there could be many such occurrences of temporary instability during earthquake shaking, producing an accumulation of "downslope" movement.

Various analytical and empirical techniques have been developed to date to estimate lateral spread ground displacement; however, no single technique has been widely accepted or verified for engineering design. Three approaches are used depending on the requirements of the project: empirical procedures; simplified analytical methods; and more rigorous computer modeling.

1. Empirical procedures use correlations between past ground displacement and site conditions under which those displacements occurred. Youd et al. (2002) present an empirical method that provides an estimate of lateral spread displacements as a function of earthquake magnitude, distance, topographic conditions, and soil deposit characteristics. As shown in Figure C11.8-6, the displacements estimated by the Youd et al. (2002) method are generally within a factor of two of the observed displacements. Bardet et al. (2002) present an empirical method having a formulation similar to that of Youd et al. (2002) but using fewer parameters to describe the soil deposit. The Bardet et al. (2002) model was developed to assess lateral spread displacements at a regional scale rather than for site-specific applications. Various other empirical methods are also available, including an alternate SPT method by Rausch and Martin (2000) and both ~~a~~ SPT and CPT-based methods by Zhang et al. (2004). These methods can result in large differences in predicted displacement and therefore, it is usually best to use several methods when estimating displacement. Because of the uncertainty in results, these methods are normally ~~best~~ used for preliminary screening or comparative evaluations.

2 Simplified analytical techniques generally apply some form of Newmark's analysis of a rigid body sliding on an infinite or circular failure surface with ultimate shear resistance estimated from the ~~residual~~ strength of the liquefied ~~deforming~~ soil. Additional discussion of the simplified Newmark method is provided in the discussion of slope instability hazard. A key question for this approach is the method of defining the strength of the liquefied soil. The same residual strength as used for flow failure assessments has often been used for the spreading analyses. However, many researchers will argue that lateral spreads do not involve the same boundary conditions as occur for lateral flows, and specifically that the ratcheting mechanism of loading with dilation at larger strains is not properly considered. No consensus currently exists on the most suitable method for obtaining the liquefied strength for lateral spreading analyses, though the use of the residual strength from flow failures is thought to be conservative for most lateral spreading analyses. Work by Olson and Johnson (2008) appear to support the acceptability of use of the residual strength. In view of the current uncertainties, a cautious approach must be taken when estimating deformations for cases involving liquefaction.

3. More rigorous computer modeling typically involves use of nonlinear finite element or finite difference methods to predict deformations, such as with the computer code FLAC. As noted previously, the accuracy of this approach is critically dependent on the properties and geometry of the model, as well as the earthquake record selection. Of particular importance for the liquefaction problem is the completeness of the pore pressure model, and its ability to handle various soil conditions. ~~Both the simplified Newmark method and the rigorous computer codes require considerable experience to obtain meaningful results.~~ For example, the soil model within the nonlinear computer codes is often calibrated for only specific conditions. If the site is not characterized by these conditions, errors in estimating the displacement by a factor of two or more can easily occur.

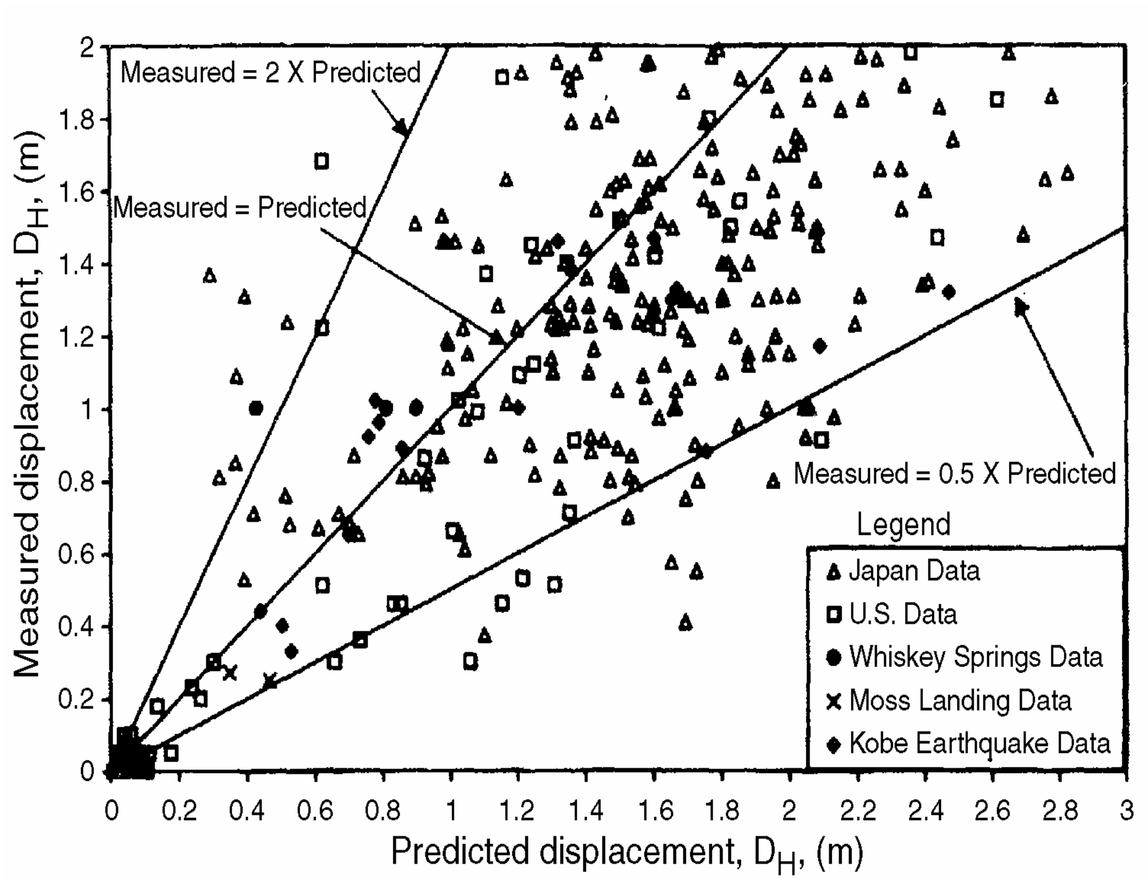


Figure C11.8-6. Measured versus predicted displacements for displacements up to 2 meters. (Youd et al., 2002).

Mitigation of Liquefaction Hazard. Three general measures might be considered for mitigation of liquefaction hazards: (1) design the structure to resist the hazard, (2) stabilize the site to reduce the hazard, or (3) choose an alternative site. Structural measures that are used to reduce the hazard include deep foundations, mat foundations, or footings interconnected with ties. Deep foundations have performed well at level sites of liquefaction where effects were limited to ground settlement and ground oscillation with no more than a few inches of lateral displacement. Deep foundations, such as piles, must consider the potential for ~~may receive~~ reduced soil support through the liquefied layer and may be subjected to ~~transient~~ lateral displacements across the layer. Well reinforced mat foundations also have performed well at localities where ground displacements were less than 1 foot, although re-leveling of the structure has been required in some instances (Youd, 1989). Strong ties between footings also should provide increased resistance to damage where differential ground displacements are less than a foot.

Evaluations of structural performance following two Japanese earthquakes, 1993 Hokkaido Nansei-Oki and 1995 (Kobe) Hyogo-Ken Nanbu, indicate that small structures on shallow foundations performed well in liquefaction areas where ground displacements were small. Sand boil eruptions and open ground fissures in these areas indicate minor effects of liquefaction, including ground oscillation and up to a foot ~~several tenths of a meter~~ of lateral spread displacement. Many small structures (mostly houses, shops, schools, etc.) were structurally undamaged although a few tilted slightly. Foundations for these structures consist of reinforced concrete perimeter wall footings with reinforced concrete interior wall footings tied into the perimeter walls at intersections. These foundations acted as diaphragms causing the soil to yield beneath the foundation which prevented fracture of foundations and propagation of differential displacements into the superstructure.

~~Similarly, well reinforced foundations that would not fail could be used in U.S. practice as a mitigative measure to reduce structural damage in areas subject to liquefaction but with limited potential for lateral (< 0.3 m) or vertical (< 0.05 m) ground displacements. Such strengthening also would serve as an effective mitigation measure against damage from other sources of limited ground displacement including fault zones, landslides, and cut fill boundaries. Where slab-on-grade or basement slabs are used as foundation elements, these slabs should be reinforced and tied to the foundation walls to give the structure adequate strength to resist ground displacement. Although strengthening of foundations, as noted above, would largely mitigate damage to the structure, utility connections may be adversely affected unless special flexibility is built into these nonstructural components.~~

~~Another possible consequence of liquefaction to structures is increased lateral pressures against basement walls as discussed in Section 11.8-3. A common procedure used in design for such increased pressures is to assume that the liquefied material acts as a dense fluid having a unit weight of the liquefied soil. The wall then is designed assuming that hydrostatic pressure for the dense fluid acts along the total subsurface height of the wall. The procedure applies equivalent horizontal earth pressures that are greater than typical at-rest earth pressures but less than passive earth pressures. As a final consideration, to prevent buoyant rise as a consequence of liquefaction, the total weight of the structure should be greater than the volume of the basement or other cavity times the unit weight of liquefied soil. (Note that structures with insufficient weight to counterbalance buoyant effects could differentially rise during an earthquake.)~~

At sites where expected ground displacements are unacceptably large, ground modification to lessen the liquefaction or ground failure hazard or selection of an alternative site may be required. Techniques for ground stabilization to prevent liquefaction of potentially unstable soils include removal and replacement of soil; compaction of soil in place using vibrations, heavy tamping, compaction piles, or compaction grouting; buttressing; chemical stabilization with grout; and installation of drains. Further discussion of mitigation methods is given by the National Research Council (1985) and Martin and Lew (1999).

Surface Fault Rupture Hazard

Fault ruptures during past earthquakes have led to large surface displacements that are potentially destructive to engineered construction. Displacements, which range from a fraction of an inch to tens of feet, may occur along traces of active faults. The sense of displacement ranges from horizontal strike-slip to vertical dip-slip to many combinations of these components. The following paragraphs ~~commentary~~ summarize~~s~~ procedures to follow or consider when assessing the hazard of surface fault rupture. Sources of detailed information for evaluating the hazard of surface fault rupture include Slemmons and dePolo (1986), the Utah Section of the Association of Engineering Geologists (1987), Swan et al. (1991), Hart and Bryant (1997), Hanson et al. (1999), and California Geological Survey (2002). Other beneficial references are given in the bibliographies of these publications.

Assessment of Surface Faulting Hazard. The evaluation of surface fault rupture hazard at a given site is based extensively on the concepts of recency and recurrence of faulting along existing faults. The magnitude, sense, and frequency of fault rupture vary for different faults or even along different segments of the same fault. Even so, future faulting generally is expected to recur along pre-existing active faults. The development of a new fault or reactivation of a long inactive fault is relatively uncommon and generally need not be a concern. For most engineering applications related to foundation design, a sufficient definition of an active fault is given in

CDMG Special Publication 42 (Hart and Bryant, 1997): “An active fault has had displacement in Holocene time (last 11,000 years).”

As a practical matter, fault investigations should be conducted by qualified geologists and directed at the problem of locating faults and evaluating recency of activity, fault length, the amount and character of past displacements, and the expected amount and potential of future displacement. Identification and characterization studies should incorporate evaluation of regional fault patterns as well as detailed study of fault features at and in the near vicinity (within a few hundred yards to a mile) of the site. Detailed studies ~~often~~ can include trenching to accurately locate, document, and date fault features.

Suggested approach for assessing surface faulting hazard. The following approach should be used, or at least considered, in fault hazard assessment. Some of the investigative methods outlined below should be carried out beyond the site being investigated. ~~However, it is not expected that all of the following methods would be used in a single investigation:~~

1. A review should be made of the published and unpublished geologic literature from the region along with records concerning geologic units, faults, ground-water barriers, etc.

2. A stereoscopic study of aerial photographs and other remotely sensed images should be made to detect fault-related topography/geomorphic features, vegetation and soil contrasts, and other lineaments of possible fault origin. The study of predevelopment aerial photographs is often essential to the detection of fault features. Recently, the use of LiDAR (LIght Detection And Ranging) has been found to provide excellent identification of fault traces in areas where tree growth and vegetation would normally obscure evidence of faulting from the air.

3. A field reconnaissance study generally is required and should include observation and mapping of features such as bedrock and soil units and structures, geomorphic surfaces, fault-related geomorphic features, springs, and deformation of man-made structures due to fault creep. Field study should be detailed within the site with less detailed reconnaissance of an area within a mile or so of the site. Evidence from prehistoric liquefaction (paleoliquefaction) can also provide important information regarding the magnitude and timing of fault movement in the site area or region.

4. Subsurface investigations may be necessary ~~commonly are needed~~ to evaluate location and activity of fault traces, where uncertainty exists about the location or activity of a fault. These investigations may include trenches, test pits, and/or boreholes to permit detailed and direct observation of geologic units and faults.

5. The geometry of faults may be further defined by geophysical investigations including seismic refraction, seismic reflection, gravity, magnetic intensity, resistivity, ground penetrating radar, etc. These indirect methods require ~~a~~ knowledge of specific geologic conditions for reliable interpretation. Geophysical methods alone never prove the absence of a fault and they typically do not identify the recency of activity.

6. More sophisticated and more costly studies may provide valuable data where geological special conditions exist or where requirements for critical structures demand a more intensive investigation. These methods might involve repeated geodetic surveys, strain measurements, or monitoring of microseismicity and radiometric analysis (C^{14}, K-Ar), stratigraphic correlation (fossils, mineralogy) soil profile development, paleomagnetism (magnetostratigraphy), or other dating techniques (thermoluminescense, cosmogenic isotopes) to evaluate the age of faulted or unfaulted units or surfaces.

The following information should be developed to provide documented support for conclusions relative to location and magnitude of faulting hazards:

1. Maps should be prepared showing the existence (or absence) and location of active ~~hazardous~~ faults on or near the site. The distribution of primary and secondary faulting (fault zone width) and fault-related surface deformation should be shown.
2. The type, amount, and sense of displacement of past surface faulting episodes should be documented, if possible.
3. From this documentation, estimates of location, magnitude, and likelihood or relative potential for future fault displacement can be made, preferably from measurements of past surface faulting events at the site, using the premise that the general pattern of past activity will repeat in the future. Estimates also may be made from published empirical correlations between fault displacement and fault length or earthquake magnitude (e.g., Wells and Coppersmith, 1994). Where fault segment length and sense of displacement are defined, these correlations may provide an estimate of future fault displacement (either the maximum or the average to be expected). Probabilistic studies may be considered to evaluate the probability of fault displacement (e.g., Youngs et al., 2003).
4. The degree of confidence and limitations of the data should be addressed.

There are no codified procedures for estimating the amount or probability of future fault displacements. Estimates may be made, however, by qualified earth scientists using techniques described above. Because techniques for making these estimates are not standardized, peer review of reports is useful to verify the adequacy of the methods used and the estimated amount or frequency of movement~~s reports~~, to aid the evaluation by the permitting agency, and to facilitate discussion between specialists that could lead to the development of standards.

The following guidelines are given for safe siting of engineered construction in areas crossed by active faults:

1. Where ordinances have been developed that specify safe setback distances from traces of active faults or active fault zones, those distances must be complied with and accepted as the minimum for safe siting of buildings. For example, Provisions Section 11.8 precludes structures in Seismic Design Category E or F from being sited where there is a known potential for an active fault to cause rupture of the ground surface at the structure. Some states have adopted more definitive requirements. For example, the general setback requirement in California is a minimum of 50 ft from a well-defined zone containing the traces of an active fault. That setback distance is mandated as a minimum for structures near faults unless a site-specific special geologic investigation shows that a lesser distance could be safety applied (*California Code of Regulations*, Title 14, Division 2, Sec. 3603(a)).
2. In general, safe setback distances may be determined from geologic studies and analyses as noted above. Setback requirements for a site should be developed by the site engineers and geologists in consultation with professionals from the building and planning departments of the jurisdiction involved.

Where sufficient geologic data have been developed to accurately locate the zone containing active fault traces and the zone is not complex, a smaller ~~50 ft~~ setback distance may be specified. For complex fault zones, greater setback distances may be required. Dip-slip faults, with either normal or reverse motion, typically produce multiple fractures within rather wide and irregular fault zones. These zones generally are confined to the hanging-wall side of the fault leaving the footwall side little disturbed. Setback requirements for such faults may be rather narrow on the footwall side, depending on the quality of the data available, and larger on the hanging wall side of the zone. Some fault zones may contain broad deformational features such as pressure ridges and sags rather than clearly defined fault scarps or shear zones. Nonessential structures may be sited in these zones provided structural mitigative measures are applied as noted below. Studies by qualified geologists and engineers are required for such zones to assure that building foundations can withstand probable ground deformations in such zones.

Mitigation of Surface Faulting Hazards. There is no mitigative technology that can be used to prevent fault rupture from occurring. Thus, sites with unacceptable faulting hazard must either be avoided or structures designed to withstand ground deformation or surface fault rupture.

In general practice, it is economically impractical to design a structure to withstand more than a few inches of fault displacement. Some buildings with strong foundations, however, have successfully withstood or diverted a few inches of surface fault rupture without damage to the structure (Youd, 1989; Kelson et al., 2001). Well reinforced mat foundations and strongly inter-tied footings have been most effective. In general, less damage has been inflicted by compressional or shear displacement than by vertical or extensional displacements.

SEISMIC LATERAL EARTH PRESSURES

Determination of Lateral Pressures on Basement and Retaining Walls Due to Earthquake Motions. Paragraph 1 of Section 11.8.3 requires that seismic lateral pressures on basement walls and ~~free-standing~~ retaining walls be determined for structures on SDC D through F, but does not specify the methods for calculating these pressures. Discussion and guidance regarding different approaches for determining seismic lateral pressures is given below.

Waterfront structures often have performed poorly in major earthquake due to excess pore water pressure and liquefaction conditions developing in relatively loose, saturated granular soils. However, damage reports for structures away from waterfronts are generally limited with only a few cases of stability failures or large permanent movements (Whitman, 1991). Due to the apparent conservatism or overstrength in static design of most walls, the complexity of nonlinear dynamic soil-structure interaction, and the poor understanding of the behavior of retaining structures with cohesive or dense granular soils, Whitman (1991) recommends that “engineers must rely primarily on a sound understanding of fundamental principles and of general patterns of behavior.”

Seismic earth pressures on retaining walls are ~~is~~ discussed below for two categories of walls: “yielding” walls that can move sufficiently to develop minimum active earth pressures and “nonyielding” walls that do not satisfy this movement condition. The amount of movement to develop minimum active pressure is very small. A displacement at the top of the wall of 0.002 times the wall height is typically sufficient to develop the minimum active pressure state. Generally, free-standing gravity or cantilever walls are considered to be yielding walls (except massive gravity walls founded on rock), whereas building basement walls restrained at the top and bottom are often considered to be nonyielding.

Yielding Walls

Limit Equilibrium Force Approach. At the 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Seed and Whitman (1970) made a significant contribution by reintroducing and reformulating the Mononobe-Okabe (M-O) seismic coefficient analysis (Mononobe and Matsuo, 1929; Okabe, 1926), the earliest method for assessing the dynamic lateral pressures on a retaining wall. The M-O method is a limit-equilibrium approach based on a Coulomb failure wedge with the ~~based on the key~~ assumption that the wall displaces or rotates outward sufficiently to produce the minimum active earth pressure state.

The M-O formulation is expressed as:

$$P_{AE} = (1/2)\gamma H^2 (1 - k_v) K_{AE} \qquad \text{(C11.8-4)}$$

where:

P_{AE} is the total (static + dynamic) lateral thrust, γ is unit weight of backfill soil, H is height of backfill behind the wall, k_V is vertical ground acceleration divided by gravitational acceleration, and K_{AE} is the static plus dynamic lateral earth pressure coefficient which is dependent on (in its most general form) angle of friction of backfill, angle of wall friction, slope of backfill surface, and slope of back face of wall, as well as horizontal and vertical ground acceleration. The formulation for K_{AE} is given in textbooks on soil dynamics (Prakash, 1981; Das, 1983; Kramer, 1996) and discussed in detail by Ebeling and Morrison (1992).

The value of a_{max} used in the K_{AE} determination is for the design earthquake ground motion (that is, 2/3rd of the MCE ground motion), as required by Section 11.8.3. The instantaneous peak acceleration (that is, a_{max} for the design earthquake ground motion) and not an average of the ground motion is used in this determination. In the past it has been common practice for geotechnical engineers to reduce the instantaneous peak by a factor from 0.5 to 0.7 to represent an average seismic coefficient for determining the seismic earth pressure on a wall. The reduction factor was introduced in a manner similar to the method used in a simplified liquefaction analyses to convert a random acceleration record to an equivalent average series of cyclic loads. This approach can result in confusion on the magnitude of the seismic active earth pressure and, therefore, is not recommended. The use of the design earthquake rather than the MCE in the determination of a_{max} already considers that a reserve of 1.5 exists within the structural design. Any further reduction to represent average rather than instantaneous peak loads is a structural decision and must be an informed decision made by the structural designer. As discussed within the section **Displacement-based Approach,** a reduction in a_{max} is, however, permitted if the wall can undergo permanent displacements.

The M-O equation makes several other very important assumptions, including that the soil behind the retaining wall is a uniform, cohesionless soil and that the groundwater elevation is below the base of the retaining wall. The implications of these assumptions are discussed later in this section.

Simplified M-O formulation. Seed and Whitman (1970), as a convenience in design analysis, proposed to evaluate the total lateral thrust, P_{AE}, in terms of its static component (P_A) and dynamic incremental component (ΔP_{AE}):

$$P_{AE} = P_A + \Delta P_{AE} \qquad \text{(C11.8-5a)}$$

or

$$K_{AE} = K_A + \Delta K_{AE} \qquad \text{(C11.8-5b)}$$

or

$$\Delta P_{AE} = (1/2)\gamma H^2 \Delta K_{AE} \qquad \text{(C11.8-5c)}$$

Seed and Whitman (1970), based on a parametric sensitivity analysis, further proposed that for practical purposes:

$$\Delta K_{AE} = (3/4)K_h \qquad \text{(C11.8-6)}$$

$$\Delta P_{AE} = (1/2)\gamma H^2 (3/4) k_h = (3/8) k_h \gamma H^2 \qquad \text{(C11.8-7)}$$

~~W~~where:

k_h is horizontal ground acceleration divided by gravitational acceleration. Unless permanent displacement of the wall is acceptable, k_h should be taken equal to the site peak ground acceleration, a_{max}, that is consistent with design earthquake ground motions. For the distribution of the dynamic thrust, ΔP_{AE}, Seed and Whitman (1970) recommended that the resultant dynamic thrust act at 0.6H above the base of the wall (that is, inverted trapezoidal pressure distribution). Note that this approach assumes dry, cohesionless backfill material. If soil conditions behind the wall have a cohesive soil component (that is, a c-ϕ soil), this simplified approach is no longer appropriate. Additional discussion of this issue is included below in the subsection *Limitations of M-O approach*. ~~as determined in Section 11.8.2, subsection entitled earthquake ground motions for geologic hazards evaluations.~~

Equation 11.8-7 generally is referred to as the simplified M-O formulation and is not applicable for sloping ground above the wall. For walls that are in excess of 15 f~~ee~~t in height, special studies can also be conducted to evaluate the coherency of ground motions behind the wall from which an average seismic coefficient can be developed. These special studies require consideration of the frequency characteristics of ground motion, as well as the stiffness of the soil and the wall height, and usually require use of a finite element or difference computer model.

Since its introduction, there has been a consensus in geotechnical engineering practice that the simplified M-O formulation reasonably represents the dynamic (seismic) lateral earth pressure increment for yielding retaining walls. However, there are limitations associated with the M-O approach, and these limitations can have a significant effect on the magnitude of estimated seismic active earth pressure. ~~For the distribution of the dynamic thrust, ΔP_{AE}, Seed and Whitman (1970) recommended that the resultant dynamic thrust act at 0.6H above the base of the wall (that is, inverted trapezoidal pressure distribution).~~

Limitations of M-O approach. Although the M-O approach is simple to use, certain designs become very difficult to solve with the standard M-O equations. These designs involve high ground accelerations, combinations of moderate-to-high ground accelerations and steep backslopes, and where mixed backfill conditions exist (that is, either where c-ϕ soils occur or where only a thin zone of granular backfill is placed between the wall and a cohesive or rock condition). For these cases the M-O approach does not provide realistic answers.

An acceptable alternative approach for these cases is to use a generalized limit equilibrium (slope stability) computer program. With this alternate approach appropriate soil properties and geometry can be modeled, and the seismic coefficient can be defined on the basis of the peak ground acceleration or a reduced seismic coefficient if displacement is acceptable. For most seismic loading cases the total stress (undrained) c-ϕ parameters will be appropriate for design because of the rate of seismic loading. With this generalized limit equilibrium method, the external force required for stability is computed. This force represents the dynamic earth pressure on the wall. The total force can be distributed as a uniform seismic pressure or the seismic increment can be determined and applied as an inverted triangle. Note that when subtracting the static force from the total seismic earth pressure, it is necessary to determine the static earth pressure under the same conditions as used during the pseudo-static seismic analysis. This could mean determining the static earth pressure for the same c-ϕ combination as used for the seismic analysis.

Displacement-based Approach. The alternate approach for the design of yielding walls is to evaluate the movement of the wall during seismic loading. Various methods are available for conducting displacement-based analyses – ranging from extensions of the M-O formulations to two- and three-dimensional computer modeling. One of the key requirements for the displacement-based approach is the determination of the level of acceptable displacements. This determination will depend both on the wall type and on the nature of facilities next to the wall. These nearby facilities can range from buildings to buried utilities.

Careful attention needs to be given to the characterization of soil conditions behind the wall when using a displacement-based approach. Both the geometry of fill and native deposits, as well as the strength of the soil under cyclic loading, must be considered. Initially, the peak strength of the soil can be used for the analysis; however, if significant deformations are predicted, it may be necessary to repeat the analysis using the residual strength of the soil. See discussions on site characterization within the seismic slope stability section for additional guidance on the selection of soil strengths.

Simplified M-O Approach. ~~Using the simplified M-O formulation, a yielding wall may be designed using either a limit-equilibrium force approach (conventional retaining wall design) or an approach that permits movement of the wall up to tolerable amounts.~~ Richards and Elms (1979) introduced a method for seismic design analysis of yielding walls considering translational sliding as a failure mode and based on tolerable permanent displacements for the wall. Elms and Martin (1979) showed that the $k_h = a_{max}/2$ is adequate for design if the wall is allowed to slide up to $10a_{max}$, where a_{max} is the design ground motion and displacement is in “inches.” For seismically active areas, the displacement associated with 10 a_{max} can be 4 to 6 inches.

The use of $k_h = a_{max}/2$ is often used ~~has now become a common design practice~~ without regard for the displacement that is associated with this assumption. Clearly, several inches of movement can be tolerable for some types of yielding walls, but not all. For example, a semi-gravity cantilever wall could be designed to slide several inches; however, the anchors for a tieback wall would likely restrict this level of movement from occurring for a well designed anchored wall. Use of $k_h = a_{max}/2$ requires that the designer check to confirm that deformations will develop without damaging the wall or other nearby facilities. Consideration should also be given to other factors that could limit displacements, such as physical obstructions or uncertainties in the amount of strength that will be mobilized. If movement cannot be tolerated or if the wall may not move enough to mobilize the yield condition, then either the full a_{max} should be used for determination of seismic earth pressure or more rigorous procedures such as described below should be used.

~~Similar to evaluations of seismic slope stability described in Section 11.8.2, careful attention needs to be given to the characterization of soil conditions behind the wall when using the displacement-based approach. Both the geometry of fill and native deposits, as well as the strength of the soil under cyclic loading, must be considered. Initially, the peak strength of the soil can be used for the analysis; however, if significant deformations are predicted, it may be necessary to repeat the analysis using the residual strength of the soil. See discussions on site characterization within the seismic slope stability section for additional guidance on the selection of soil strengths.~~

Other simplified displacement methods. There are a number of other empirical formulations for estimating permanent displacements under a translation mode of failure; these have been reviewed by Whitman and Liao (1985). Nadim (1980) and Nadim and Whitman (1984) incorporated the failure mode of wall tilting as well as sliding by employing coupled equations of motion, which were further formulated by Siddharthan et al. (1992) as a design method to predict the seismic performance of retaining walls taking into account both sliding and tilting. Alternatively, Prakash et al. (1995) described design procedures and presented design charts for estimating both sliding and rocking displacements of rigid retaining walls. These design charts are the results of analyses for which the backfill and foundation soils were modeled as nonlinear viscoelastic materials. A simplified method that considers rocking of a wall on a rigid foundation about the toe was described by Steedman and Zeng (1996) and allows the determination of the threshold acceleration beyond which the wall will rotate. A simplified procedure for evaluating the critical threshold accelerations for sliding and tilting was described by Richards et al. (1996).

Application of methods for evaluating tilting of yielding walls has been limited to a few case studies and back-calculation of laboratory test results. Evaluation of wall tilting requires considerable engineering judgment. Because the tilting mode of failure can lead to instability of a yielding retaining wall, it is suggested that this mode of failure be avoided in the design of new walls by proportioning the walls to prevent rotation in order to displace only in the sliding mode.

Computer Modeling. An alternative displacement-based approach is the use of two-dimensional computer codes such as FLAC and PLAXIS. These methods allow a more detailed evaluation of soil-structure interaction for different wall geometries and external loads, different soil and structural properties, and different earthquake loading conditions. Results from these analyses can be particularly helpful in understanding the deformations that occur within and near the retaining wall, including the soil in front of and behind the wall. As noted previously, these methods require considerable expertise in terms of soil and structural modeling and selection of earthquake records and should be used with particular care. While results may appear very reasonable, small changes in model setup or input parameters selection can significantly affect the quality of results, potentially leading to unconservative design decisions.

~~*Limitations of M-O approach.* Although the M-O approach is simple to use, certain designs become very difficult to solve with the standard M-O equations. These designs involve high ground accelerations, combinations of moderate-to-high ground accelerations and steep backslopes, and where mixed backfill conditions exist (i.e., either where c-ϕ soils occur or where only a thin zone of granular backfill is placed between the wall and a cohesive or rock condition). For these cases the M-O approach does not provide realistic answers. An acceptable alternative approach for these cases is to use a generalized limit equilibrium (slope stability) computer program. With this alternate approach appropriate soil properties and geometry can be modeled, and the seismic coefficient can be defined on the basis of the peak ground acceleration or a reduced seismic coefficient if displacement is acceptable. With this generalized limit equilibrium method, the external force required for stability is computed. This force represents the dynamic earth pressure on the wall.~~

Nonyielding Walls

By definition nonyielding walls do not deform when subjected to seismic earth pressures. This type of response requires a very stiff wall in combination with a rigid base condition. Most nonyielding walls will be located on rock or very stiff soil. Even in this condition, wall flexibility can be sufficient to develop active seismic earth pressures, significantly reducing the loading on basement walls. Where the basement wall is located on rock or very stiff soil and where structural analyses determine that the wall flexibility is such that deformations will not develop seismic active earth pressures (that is, deformations < 0.002H where H is the wall height), the wall should be designed as a nonyielding wall. The following discussions provide guidance on two methods for dealing with cases where rigid wall conditions occur. Also included is a discussion of soil-structure interaction methods for evaluating earth pressures on basement walls where uncertainties on the flexibility of the wall occur.

Simplified Wood Approach. Wood (1973) analyzed the response of a rigid nonyielding wall retaining a homogeneous linear elastic soil and connected to a rigid base. For such conditions, Wood established that the dynamic amplification was insignificant for relatively low-frequency ground motions (that is, motions at less than half of the natural frequency of the unconstrained backfill), which would include many earthquake problems.

For uniform, constant k_h applied throughout the elastic backfill, Wood (1973) developed the dynamic thrust, ΔP_E, acting on smooth rigid nonyielding walls as:

$$\Delta P_E = F k_h \gamma H^2 \tag{C11.8-8}$$

The value of F is approximately equal to unity (Whitman, 1991) leading to the following approximate formulation for a rigid nonyielding wall on a rigid base:

$$\Delta P_E = k_h \gamma H^2 \tag{C11.8-9}$$

As for yielding walls, the point of application of the dynamic thrust is taken typically at a height of $0.6H$ above the base of the wall.

It should be noted that the model used by Wood (1973) does not incorporate any effect on the pressures of the inertial response of a superstructure connected to the top of the wall. This effect may modify the interaction between the soil and the wall and thus modify the pressures from those calculated assuming a rigid wall on a rigid base.

Although the study performed by Wood included dynamic analysis of a rigid wall with fixed base condition, the solution commonly used and presented in Equations C11.8-8 and C11.8-9 are based on static "1 g" loading of the soil and wall and does not include the effects of the wave propagation in the soil. The subject of soil-wall interaction is addressed in the following sections. ~~This section also provides further discussion on the applicability of the Wood and the M-O formulations.~~

~~**Soil-structure-interaction Approach and Modeling for Partially Embedded Structures.**~~ ~~Lam and Martin (1986), Soydemir and Celebi (1992), Veletsos and Younan (1994a and 1994b), and Ostadan (2005), among others, argue that the earth pressures acting on the walls of partially embedded structures (e.g., basement walls) during earthquakes are primarily governed by soil-structure interaction (SSI) and, thus, should be treated differently from the concept of limiting equilibrium (that is, M-O method). Soil-structure interaction includes both a kinematic component—the interaction of a massless rigid wall with the adjacent soil as modeled by Wood (1973) but including the wave propagation in the soil—and an inertial component—the interaction of the wall, connected to a responding superstructure, with the adjacent soil. Detailed SSI analyses incorporating kinematic and inertial interaction may be considered for the estimation of seismic earth pressures on critical walls. Computer programs that may be utilized for such analyses include FLUSH (Lysmer et. al, 1975) and SASSI2000 (Lysmer et al., 1999).~~

Ostadan Rigid Wall Approach. Ostadan (2005) observed that for partially embedded structures subjected to ground shaking, the characteristics of the dynamic earth pressure amplitudes versus frequency of the ground motion were those of a single-degree-of-freedom (SDOF) system and proposed a simplified method to estimate the magnitude and distribution of dynamic thrust. Results provided by Ostadan (2005) utilizing this simplified method, which were also confirmed by dynamic finite element analyses, indicate that, depending on the dynamic properties of the backfill as well as the frequency characteristics of the input ground motion, a range of dynamic earth pressure solutions would be obtained for which the M-O solution and the Wood (1973) solution represent a "lower" and an "upper" bound, respectively.

The solution by Ostadan considers the kinematic soil-structure interaction effects and is based on the dynamic soil properties and the design ground motion characteristics. This solution assumes a rigid wall on rigid foundation and does not include the effect of the superstructure and its inertia on seismic soil pressure. The 5-step method to compute the seismic soil pressure following Ostadan's method is as follows:

1. Perform free-field soil column analysis and obtain the ground response motion at the depth corresponding to the base of the wall in the free-field. The response motion in terms of acceleration response spectrum at 30 percent damping should be obtained. The free-field soil column analysis may be performed using the computer program SHAKE with input motion specified either at the ground surface or at the depth of the foundation basemat. The choice for location of control motion should be consistent with the development of the design motion.
2. Use Eq. (C11.8-10) to compute the total soil mass (m) using the Poisson's ratio (ν) and mass density of the soil.

   $$m = 0.50\,(\rho)\,H^2\,(\Psi\nu) \qquad \text{(C11.8-10)}$$

   where:

   ρ = mass density of the soil (total weight density divided by acceleration of gravity)
   H = height of the wall
   $\Psi\nu$ = factor to account for the Poisson's ratio as defined by the following equation.

   $$\Psi\nu = 2 / [(1-\nu)(2-\nu)]^{0.5} \qquad \text{(C11.8-11)}$$

3. Obtain the total lateral seismic force from the product of the total mass obtained in Step 2 and the acceleration spectral value of the free-field response at the soil column frequency obtained at the depth of the bottom of the wall (Step 1). The soil column frequency (f_s) is an output provided by SHAKE.

   $$f_s = v_s/(4H) \qquad \text{(C11.8-12)}$$

   where:

   v_s = average strain-compatible shear wave velocity of the soil column over the height of the wall.

4. Obtain the maximum lateral seismic soil pressure at the ground surface level by dividing the lateral force obtained in Step 3 by the area under the normalized seismic soil pressure, 0.744 H.

5. Obtain the pressure profile by multiplying the peak pressure from Step 4 by the pressure distribution relationship given by Equation (C11.8-13 below).

   $$p(y) = -0.0015+5.05y-15.84y^2+28.25y^3-24.59y^4+8.14y^5 \qquad \text{(C11.8-13)}$$

   where:

   y = normalized height ratio (Y/H) measured from the bottom of the wall and ranging from 0 at the bottom of the wall to 1 at the top of the wall.

   Y = the distance of the point under consideration from the bottom of the wall.

The area under the seismic soil pressure curve can be obtained from integration of the pressure distribution over the height of the wall. The total area is 0.744H x $p_{\max}$ for a wall with the height of H and maximum pressure of $p_{\max}$ at the top.

With this method, the site specific dynamic soil properties, soil nonlinear effects and the characteristics of the design motion are considered in the computation of the seismic soil pressure. A complete verification of the 5-step method against finite element solutions and comparison with the Wood solution and the M-O method is presented by Ostadan (2005).

Soil-structure-interaction Approach and Modeling for Partially Embedded Structures. Lam and Martin (1986), Soydemir and Celebi (1992), Veletsos and Younan (1994a and 1994b), and Ostadan (2005), among others, argue that the earth pressures acting on the walls of partially embedded structures (e.g., basement walls) during earthquakes are primarily governed by soil-structure interaction (SSI) and, thus, should not be treated as a nonyielding wall. Soil-structure interaction includes both a kinematic component—the interaction of a massless rigid wall with the adjacent soil as modeled by Wood (1973) but including the wave propagation in the soil—and an inertial component—the interaction of the wall, connected to a responding superstructure, with the adjacent soil. Detailed SSI analyses incorporating kinematic and inertial interaction may be considered for the estimation of seismic earth pressures on critical walls.

~~**Observations and Discussion for Partially Embedded Structures**~~. ~~Chang et al. (1990) have found that dynamic earth pressures recorded on the wall of a model nuclear reactor containment building were consistent with dynamic pressures predicted by the M-O solution. Analysis by Chang et al. indicated that the dynamic wall pressures were strongly correlated with the rocking response of the structure.~~ Whitman (1991) has suggested that SSI effects on basement walls of buildings reduce dynamic earth pressures and that M-O pressures may be used in design except where structures are founded on rock or hard soil (that is, where no significant rocking occurs). In the latter case, the pressures given by the Ostadan (2005) method with the Wood (1973) formulation as the upper bound would appear to be more applicable. The effect of rocking in reducing the dynamic earth pressures on basement walls also has been suggested by Ostadan and White (1998). This condition may be explained if it is demonstrated that the dynamic displacements induced by kinematic and inertial components are out of phase. Chang et al. (1990) have found that dynamic earth pressures recorded on the wall of a large-scale model nuclear reactor containment building (e.g., 1/4 the size of a full-size power block) were consistent with dynamic pressures predicted by the M-O solution. Analysis by Chang et al. indicate that the dynamic wall pressures were strongly correlated with the rocking response of the structure.

Effect of Saturated Backfill on Wall Pressures.

The previous discussions on yielding and nonyielding walls are limited to backfills that are not water-saturated. In current (2008~~6~~) practice, drains typically are incorporated in the design to prevent groundwater from building up within the backfill. This is not practical or feasible, however, for waterfront structures (such as quay walls) where most of the earthquake-induced failures have been reported (Seed and Whitman, 1970; Ebeling and Morrison, 1992; ASCE-TCLEE, 1998).

During ground shaking, the presence of water in the pores of a backfill can influence the seismic loads that act on the wall in three ways (Ebeling and Morrison, 1992; Kramer, 1996): (1) ~~-~~by altering the inertial forces within the backfill, (2) by developing hydrodynamic pressures within the backfill and (3) by generating excess porewater pressure due to cyclic straining. Effects of the presence of water in cohesionless soil backfill on seismic wall pressures can be estimated using formulations presented by Ebeling and Morrison (1992) and Kramer (1996). The effects of soil liquefaction associated with excess porewater pressure generation on wall pressures is discussed in the subsection entitled *Increased lateral earth pressure* within the **Liquefaction Hazard** section.

~~A soil liquefaction condition behind a wall may under the design earthquake have a pronounced effect on the wall pressures during and for some time after the earthquake. At present there is no general consensus established for estimating lateral earth pressures for liquefied backfill conditions. One simplified and probably somewhat conservative approach is to treat the liquefied backfill as a heavy viscous fluid exerting a hydrostatic pressure on the wall. In this case, the viscous fluid has the total unit weight of the liquefied soil. If unsaturated soil is present above the liquefied soil, it is treated as a surcharge that increases the fluid pressure within the underlying liquid soil by an amount equal to the thickness times the total unit weight of the surcharge soil. In addition to these "static" fluid pressures exerted by a liquefied backfill, hydrodynamic pressures can be exerted by the backfill. The magnitude of any such hydrodynamic pressures would depend on the level of shaking following liquefaction. Hydrodynamic effects may be estimated using formulations presented by Ebeling and Morrison (1992).~~

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Bachman (YR): The vote of TS-3 on this proposal should be provided to the PUC along with any comments if not resolved. I would change my vote to Y if the vote of TS-3 on this proposal were provided along with any unresolved comments and the TS-3 Response.

Proposal 7-10R1 (Y=14, YR=2, N=0, NV=6 --100%)

At the December 9, 2008 PUC meeting, TS 7 provided a revised proposal for the PUC to review. It contained the persuasive comments from the earlier PUC review and additional comments from TS 7 members when it was balloted among them in November, 2008. The persuasive comments have been incorporated in the revised proposal. And negative votes were found nonpersuasive. Furthermore, the TS No votes are somewhat satisfied by the fact that this proposal is being submitted now as a Part 3 proposal. The PUC agreed with the TS 7 revised proposal by a vote of (18,0,0).

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Bachman (YR): The vote of TS-7 on this proposal should be provided to the PUC along with any comments if not resolved. I would change my vote to Y if the vote of TS-7 on this proposal were provided along with any unresolved comments and the TS-7 Response.

Aschheim (YR): Some minor editorial suggestions:
Page 2 Line 27: “The effect of…”
Page 3 Line 9: “…the effect of…”
Page 5 Line 47: …can affect wood…”
Page 7 Line11: suggest “…was identified experimentally…”