ISSUES and RESEARCH NEEDS IDENTIFIED DURING DEVELOPMENT OF THE 2015 *NEHRP RECOMMENDED SEISMIC PROVISIONS FOR NEW BUILDINGS AND OTHER STRUCTURES*

Prepared for the Federal Emergency Management Agency by the Provisions Update Committee of the National Institute of Building Sciences Building Seismic Safety Council

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Building Seismic Safety Council

a council of the National Institute of Building Sciences

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INTRODUCTION

As part of its efforts to regularly update the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures*, the Building Seismic Safety Council (BSSC) is charged by the Federal Emergency Management Agency (FEMA) to identify and recommend issues to be addressed and research needed to advance the state of the art of earthquake-resistant design and to serve as the basis for future refinement of the *Provisions*. During the project to generate the 2015 edition of the *Provisions*, the various Issue Teams and Study Groups that assisted with the development of proposals for Provisions Update Committee and Member Organization ballots identified specific items that were beyond the scope of the 2015 *Provisions* update. These were assembled and edited by an Oversight Committee consisting of S. K. Ghosh, Ph.D., President, S. K. Ghosh Associates (Chair), John Gillengerten, S.E., Consulting Engineer, and Gyimah Kasali, G.E., Ph.D., Executive Principal, Rutherford + Chekene. The resulting list of recommendations is presented in two groups: "Future *Provisions* Issues" and "Research Needs" for the following categories.

- Design and Analysis
- Geotechnical/Ground Motion Considerations
- Concepts for Revision of Analysis Requirements in Chapter 12 of ASCE 7
- Concrete Structures
- Masonry Structures
- Steel Structures
- Wood Structures
- Nonbuilding Structures and Nonstructural Components

The Issue Team and Study Group authors that contributed recommendations for each category also are listed. No prioritization is intended by the order of either the recommendations or their categories.

Issues and Research Needs is a companion document to NIST GCR 13-917-23, *Development of NIST Measurement Science R&D Roadmap: Earthquake Risk Reduction in Buildings*, published by the National Institute of Standards and Technology (NIST) in January 2013. This BSSC document and the NIST GCR are complementary to each other and should be used as such.

The recommendations herein are intended for review by the broad seismic community. Please direct any feedback regarding these issues and research needs to: bssc@nibs.org.

DESIGN AND ANALYSIS

By John Hooper, Magnusson Klemencic; Kelly Cobeen, Wiss, Janney, Elstner Associates; and Curt Haselton, California State University of California, Chico

Several items related to the design and analysis requirements of the 2015 NEHRP *Provisions* require further fundamental research. The methodology developed in FEMA P-695 *Quantification of Building Seismic Performance Factors* is one such research approach. In addition, specific issues requiring attention, if defensible changes are to be made in current requirements, include system irregularities (both vertical and horizontal), dual frame systems, and importance factors for Risk Category III and IV structures.

Among the design and analysis issues identified for attention are the following:

Future Provisions Issues

1. Envisioned is a simplification of ASCE/SEI 7-10 Table 12.2-1that would be more generically based on an anticipated level of ductility (ordinary, intermediate, and special) for all material types. For example, special, intermediate, and ordinary systems would have the same seismic design coefficient factors regardless of material type. Likewise, the need for the system to be dependent on Seismic Design Category and the need for height limits would be reviewed and verified. Finally, the *R* factor basis would be verified, e.g., to determine whether seismic design objectives are best categorized as "life safety" or "collapse prevention". FEMA P-695 and NIST GCD 12-917-20 provide tools for use in verification. The performance goals of nonstructural systems also need to be considered, refined, and modified as necessary to produce the desired simplification. The determination of structural performance goals should be based, at least in part, on these research efforts.

2. The degree to which design requirements in the *Provisions* vary by Seismic Design Category (SDC) is not entirely consistent, e.g., the range associated with SDC D is larger than that associated with the lower SDCs. If SDCs are deemed necessary, a reassessment of the cut-offs between the various SDCs should be conducted, especially, in light of the new risk-targeted MCE (i.e., MCE_R) approach to ground motion mapping developed as part of the 2009 NEHRP *Provisions* update. Consolidation of SDCs should be considered a potential outcome of this effort.

3. System irregularity provisions, both horizontal and vertical, include both penalty factors (in the case of excessive torsional response) and prohibitions (in the case of weak-story mechanisms) that have not necessarily been supported on a technical basis. Research is currently underway to investigate system irregularity provisions. The research results should be reviewed and incorporated into the *Provisions* as appropriate.

4. Analytical studies suggest that the collapse probability of short-period buildings is significantly larger than that of buildings with longer periods and in some cases exceeds the ASCE 7 collapse safety objective of 10% given MCE_R ground motions. Observations of short-period building damage in recent earthquakes do not support this finding. Research is currently underway to investigate and resolve the short-period building "paradox" and to develop recommended improvements to short-period building design requirements, if justified. The research results should be reviewed and incorporated into the *Provisions* as appropriate. Clarification of what constitutes a short-period structure should be provided as part of this effort.

5. A lack of consistency currently exists between Chapter 16 acceptance criteria for force-controlled components, deformation-controlled components, and consideration of unacceptable response, and the corresponding criteria in ASCE 41-13. In another example, ASCE 41's force-controlled acceptance criteria checks are based on mean, unamplified demands from response history analysis, whereas Chapter 16 recommends amplification dependent on criticality. The acceptance criteria in both documents should be reviewed with the goal of creating consistency or, at the very least, providing commentary where consistency is lacking.

6. A future need is the further development of the alternate diaphragm design force methodology introduced in the 2015 NEHRP *Provisions* Volume I: Part 1 *Recommended Provisions* and Part 2 *Commentary*. To use this alternate method beyond the limited systems currently described, the development of a methodology for determination of diaphragm force reduction factors, R_S, is required. This needs to be a large-scale effort, not unlike the development of the FEMA P-695 and FEMA P-795 methodologies. The development of testing and analysis procedures across construction materials is required, in addition to material-specific implementation of testing and analysis methodologies.

7. Dynamic analysis procedures should be reconciled with diaphragm design requirements with the goal of using a single analysis to derive forces for the vertical frames and walls, and the diaphragm system.

8. During the development of the 2015 NEHRP *Provisions*, several issues related to Modal Response Spectrum Analysis (MRSA) were identified that require further evaluation. The list includes:

   - Possible adjustment of C_S in the short period range by the square root of $(2R/\Omega_o - 1)$;
   - Reduction by R only in the first mode (assuming higher modes are elastic);
   - Consideration (or reconsideration) of the appropriateness of current approaches for scaling to the results of an equivalent lateral force (ELF) analysis. This should include both the value matched (base shear, overturning moment, or other) and the

scaling factor to be used for matching. An MRSA process independent of ELF might be considered;

- Application of a multi-degree-of-freedom factor;
- De-coupling of the ELF procedure using an approach along the lines of the Diaphragm Issue Team (IT06);
- Changes to response spectra in the very short period range;
- Modification of MRSA to better target a probability of collapse of 1 % in 50 years;
- Integration with Chapter 19 on soil-structure interaction;
- Expanded commentary on the implementation of MRSA with Chapter 17 on seismically isolated structures, Chapter 18 on structures with damping systems, and Chapter 19 on soil-structure interaction;
- Application of accidental torsion to MRSA;
- Modeling issues – vertical mass, flexible diaphragms, etc.;
- Using modal analysis to establish equivalent static forces for design in the Commentary;
- Considering elimination of MRSA in view of the inclusion of Linear Response History Analysis (LRHA) in the 2015 NEHRP *Provisions*. MRSA may no longer be needed since MRSA is a simplification of LRHA; and
- Use of Conditional Mean Spectrum in elastic analysis.

9. During the development of the 2015 NEHRP *Provisions*, an alternative design method for rigid-wall flexible diaphragm buildings was initially developed. Discussion of this methodology is provided in *Resource Paper 5: One-Story, Flexible Diaphragm Buildings with Stiff Vertical Elements* in Volume II: Part 3 of the *Provisions.* A more complete presentation is provided in FEMA P-1026, *Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An Alternative Procedure*. The methodology described in these publications is currently only applicable to very simple building configurations. Effort is needed to better define and possibly expand the scope of buildings to which the *Provisions* apply, and to move the Part 3 methodology to Part 1 to make possible consideration for incorporation into ASCE 7. As part of a move to Part 1, any available input from users should be considered.

10. Risk Category IV structures are designated as essential facilities, such as hospitals or fire stations, that are intended to remain operational in the event of extreme loading. However, the current IBC Table 1613.3.5 treats all Risk Category structures the same for the lowest seismic hazard areas defined by Seismic Design Category A. Damaging earthquakes are a possibility even in these low seismic areas. A recent example of this is the Mineral Virginia earthquake, which while not centered in the lowest seismic hazard area, still resulted in damages to structures in other areas of lowest hazard. There are no seismic design requirements for Seismic Design Category A. Given the critical post-disaster needs of Risk Category IV structures, the minimal seismic design requirements contained in SDC B would at least provide some level of protection for these critical facilities. For this reason, the following changes should be considered for the next cycle of the *Provisions*:

- For short period (0.2 second) response acceleration, $S_{DS} < 0.167g$ and Risk Category IV, Seismic Design Category A should change to B.
- For one second period response acceleration, $S_{DS} < 0.067g$ and Risk Category IV, Seismic Design Category A should change to B.

Research Needs

1. Risk Category III and IV structures are assigned importance factors of 1.25 and 1.5, respectively. This design approach provides for a lower probability of collapse, given risk-targeted, maximum considered earthquake (MCE_R) ground shaking at the site, for critical and essential facilities relative to ordinary structures. Yet, it is not clear whether the factors are appropriate for the intended functionality of these structures. *Resource Paper 5: New Performance Basis for the Provisions* in Volume II: Part 3 of the 2015 NEHRP *Provisions* outlines a potential framework for defining and implementing functionality. This work needs to be taken to a conclusion. Consideration might also be given to the following: 1) Splitting Risk Category III and IV buildings to allow development of separate performance objectives and design requirements; 2) Determining whether nonstructural scoping and exceptions by SDC are appropriate for nonstructural components. (Currently, many nonstructural considerations are waived for RC II and for RC III with no difference between the two RC's.); 3) Determining whether drift limits are more appropriately specified by the RC or the SDC; and, 4) Working out the probability of collapse in a more useful way before asserting that RC III or IV reduces the 1% in 50 years probability to some other arbitrary level. Even if it is the best metric, the probability of collapse is not uniform at 1% in 50 years in places where it matters most, such as in Coastal California or anywhere in the deterministic parts of the map.

2. Even though vertical acceleration spectra were developed during the 2009 NEHRP *Provisions* update, an in-depth assessment of these spectra should be conducted. Results from this study could be used to determine both vertical acceleration requirements for the ASCE/SEI 7 load combinations (e.g., a critical review of the term $0.2S_{DS}$) and the vertical period appropriate for analysis and design.

3. Review and potential modifications of dual system requirements and associated design coefficients are needed. This is notably relevant to dual systems with both special and intermediate moment-frame back-up systems. It is not clear whether the design requirements currently prescribed will provide the desired low probability of collapse given MCE_R ground shaking at the site. The methodology outlined in FEMA P-695 could be used to assess these requirements. Similar consideration could be given to vertical combinations of systems in buildings, including, but not limited to, podium slab buildings.

4. FEMA P-695 studies are needed to address the current structural systems listed in ASCE/SEI 7-10 Table 12.2-1, especially those systems permitted for buildings assigned to Seismic Design Category C. These studies should cover the full range of permitted heights and possible configurations and permitted detailing, not just the worst cases. Of particular importance are ordinary systems, and those for which no seismic detailing is required (e.g., ordinary steel concentrically braced frames, ordinary steel moment frames, and steel systems not specifically detailed for seismic resistance), with the objective of verifying that performance objectives are being met for these systems as currently designed (not with the addition of detailing requirements). The studies should include appropriate component and system testing to support the analytical evaluations.

5. Research is needed to determine whether any changes to the *Provisions* drift analysis requirements are warranted given the adoption of the MCE_R ground motions associated with a 1 % probability of collapse in 50 years. This is especially important for drift-controlled systems such as steel and concrete moment frames. In addition, a review is needed to determine whether scaling to R/C_d is correct for drift determination. The C_d values need to be revisited.

6. The minimum base shear requirements control the design of many tall buildings and are based on historic precedents with limited verification. A future study would be useful to further investigate the minimum base shear requirements, and how they relate to the collapse safety goals of ASCE/SEI 7 for various structural systems. Such a study also could utilize recent earthquake data to revisit the near-source basis of ASCE/SEI 7-10 Equation 12.8-6.

7. While substantial progress has been made in Chapter 16 on response history procedures to link acceptance criteria more directly to the collapse safety goals of ASCE/SEI 7, further development and research could refine the calibration of the Chapter 16 implicit satisfaction of the collapse safety goals with more explicit methods. Additionally, drift acceptance criteria in ASCE/SEI 7 have remained unchanged for many years, and were simply adjusted for use in Chapter 16. Both topics could be advanced through future study.

8. The acceptance criteria of Chapter 16 are developed by individually calibrating each acceptance criterion to the collapse safety goals. Since collapse generally involves multiple components simultaneously, a future effort should look at how the collapse probability of a building is affected by the interaction between multiple individual element acceptance criteria. As part of this work, consideration should be given to grouping of similar elements (e.g. those due to symmetry), a requirement not explicitly included in Chapter 16.

9. When developing the acceptance criteria for force- and deformation-controlled actions, assumptions were made to address the probability of total or partial collapse conditioned on the exceedance of a single component (such as 100% for critical force-controlled actions, and 40% for critical deformation-controlled actions with an alternate load path). Future work should study in greater depth the consequences of failure and potentially refine the *Provisions*. The topic of this study would overlap with that presented under "structure of acceptance criteria" in recommendations 6, 7, and 8 above.

10. The acceptance criteria for force-controlled components in Chapter 16 are structured in such a way that the final criterion is independent of the strength reduction factor (ϕ). Further refinement might indicate a preference for making the acceptance criteria dependent on the value of ϕ.

11. The current orthogonal load requirements for elements of non-planar frames and walls do not adequately address potential bi-directional post-yield response. Research is needed to determine the appropriate level of post-yield response for the following cases:

    - Strong-column/weak-beam requirements for special moment frames;
    - Axial forces on braced-frame columns; and
    - Flange forces in walls.

CONCEPTS FOR REVISION OF ANALYSIS REQUIREMENTS IN ASCE7 CHAPTER 12

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Future Provisions Issues

1. Moving Towards the use of Full System Modeling

While the linear analysis procedures in ASCE 7-16 are effective, they are representative of computational technology that is at least 50 years old, and should be modified to reflect current capabilities and practices. Assuming that *R* will continue to be used as an ad-hoc methodology for incorporating inelastic effects, the remaining issues are related to the development of the mathematical model, and to the use of the computed results to assess conformance with deformation-based acceptance criteria. Additionally, it is important to provide greater consistency between the Equivalent Lateral Force (ELF), Modal Response Spectrum (MRS), and Linear Response History (LRH) analysis methods.

The current Chapter 12 analysis philosophy begins with the assumption that a 2-D statically loaded model is adequate, and then uses a rule-based procedure to determine whether the following analytical features should be included:

1. Three dimensional analysis (Section 12.7.3)
2. Semi-rigid diaphragms (Sections 12.3 and 12.7.3)
3. P-Delta effects (Sections 12.7.3 and 12.8.7)
4. Orthogonal load effects (Section 12.5)
5. Dynamic effects [using MRS or LRH in lieu of ELF] (Section 12.6)
6. Accidental torsion and torsional amplification (Sections 12.8.4.2 and 12.8.4.3)
7. Location for evaluating deformations (Section 12.8.6)
8. Inclusion of structural components that are not part of the main lateral load resisting system (this item is not explicitly included in chapter 12 of ASCE 7-16)

Fifty years ago each of the required increases in analytical resolution and related computational effort had significant practical consequences in terms of the time required to develop and analyze the model, and the time required to evaluate the results. Modern software enables very rapid development of the mathematical model in three dimensions, and analysis for the most complex systems (with tens of thousands of degrees of freedom) that execute in seconds, even for LRH analysis. Thus, it seems reasonable that the current "bottom up" approach should be replaced by a "top down" philosophy in which all of the features described in items 1-7 of the above list be initially required. In some cases exceptions could be provided to relax certain requirements (allowing, for example, the use of rigid diaphragms).

Another significant aspect of the analytical model not explicitly required in ASCE-7 is the inclusion of those portions of the structural system that are not detailed to resist lateral loads, specifically the gravity framing system. This is represented as item 8 in the list. The explicit modeling of the location and axial forces in the vertical elements of the gravity system is required to accurately capture 3-Dimensional P-Delta effects, but it is noted that the lateral stiffness and strength of the gravity system is not required.

The author has termed the revised analysis philosophy Full System Modeling (FSM), and is suggesting that Chapter 12 of ASCE 7 be revised to implement this concept. While some of the concepts related to FSM below may seem "too advanced" in relation to ASCE 7-16, it is important to note that the version of ASCE 7 that follows ASCE 7-16 will not likely be in use until 2023 or 2024. This provides sufficient time to move the practice forward, and to encourage software developers to implement the recommended features in future versions of their analysis and design programs.

In the next part of this discussion, the eight bullet items listed above are addressed individually. After this is done, recommendations are provided for three "Significant Research Efforts" that should be initiated simultaneously with the development of the proposed philosophy.

<u>1. Three Dimensional Analysis</u>

The next edition of ASCE 7 should require 3-D analysis for all building structures[1] except for those that can be shown to have flexible diaphragms at each level. This is a minor extension because a 3-D model is already required for all MRS or LRH analysis, and because 3-D analysis is required for ELF analysis if *any* horizontal irregularity exists (see next item below). Additionally, the presence of a torsional irregularity cannot be determined without a 3-D model.

<u>2. Semi-Rigid Diaphragms</u>

Section 12.3.1.2 requires the use of semi-rigid diaphragms when any horizontal irregularity exists. Two of the listed horizontal irregularities in Table 12.3-1 should be reviewed in association with this requirement.

Reentrant corner irregularity: It does not seem logical that a relatively small corner cutout would eliminate essentially rigid diaphragm behavior (Fig. 1a). Also, cutouts (notches- see Fig. 1b) not at the corners should be evaluated to assess the influence of diaphragm rigidity, and there is no clear requirement for this.

Diaphragm discontinuity irregularity: Diaphragm openings of less than 50% of the enclosed area can lead to significant in-plane deformations. In ASCE 7-16, the diaphragm shown in Fig. 1c would not be characterized as irregular, and this is clearly incorrect.

[1] This recommendation could be limited to buildings in Seismic Design Category C and higher.

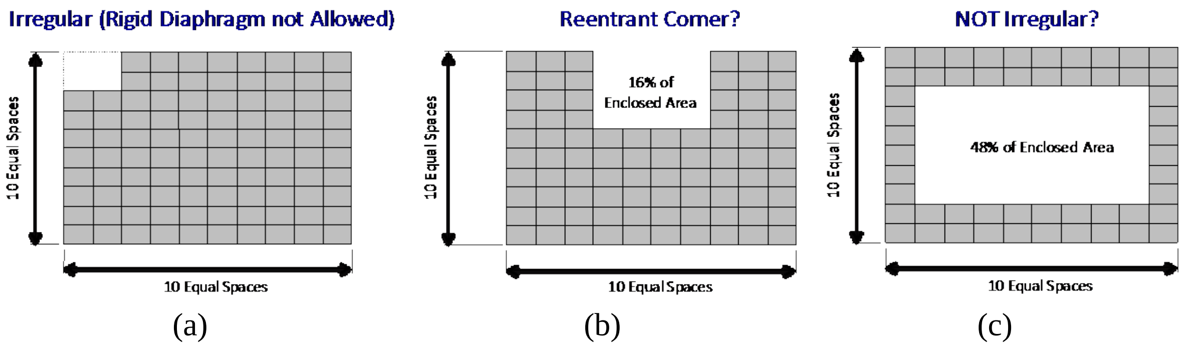

Figure 1. Diaphragm Irregularities

For linear analysis (including LRH) there is virtually no computational benefit to model diaphragms as rigid. Linear analysis of very complex 3-D systems with tens of thousands of degrees of freedom generally execute in a few seconds, and disc storage requirements are small relative to the overall computer system capacity.

Where the in-plane flexibility of diaphragms is included in the analytical model, membrane or shell elements should be used to represent the diaphragm. Where shell elements are used these elements must not resist out-of-plane bending, as this may reduce the moments in the seismic elements. While the diaphragm will certainly have some flexural resistance in the actual structure, it generally is detailed to provide adequate ductility (or in most cases will not have ductility that is equivalent to the main lateral load resisting system).

Out of plane moments will not occur if membrane elements are used to model the diaphragms. However, membrane nodes that are not attached to structural elements (for the main system and the gravity system) will need to be vertically and rotationally restrained.

Some guidance should be provided in the commentary as to the proper discretization of the diaphragm when semi-rigid diaphragms are required. A very coarse mesh can be used if it is only the flexibility of the diaphragm that is of issue. A much finer mesh is needed if stresses need to be recovered.

3. P-Delta Effects (See also Research Effort 1)

P-Delta effects should be included in all 3-D analysis, and the use of equation 12.8-16 should be discontinued. This equation is not applicable to 3-D systems because it does not include torsional P-delta effects (also known as P-Theta effects). In lieu of using equation 12.8-16, it is recommended that story stability limits be based on analysis performed for the system with and without P-delta effects. For example, the following equation could be used:

$$\theta = 1 - \frac{\Delta_0}{\Delta_f}$$

where Δ_0 and Δ_f are the story drift at the center of mass for the analysis with and without P-Delta effects, respectively. The same expression can be used to gage sensitivity to torsional P-delta effects by using the computed story torsional rotations instead of the story drift.

Additionally, equation 12.8-17 is logically flawed where $\beta = 1$ and $C_d > 5.0$. For example, if $\beta = 1$ and $C_d = 6$, $\theta_{max} = 0.0833$. Thus, $\theta > \theta_{max}$, but P-Delta effects need not be considered because $\theta < 0.1$. Additionally, if equation 12.17 is retained (modified to eliminate the logic error), the term β should be eliminated and replaced with a new term in the numerator, called (say) Ω_S, which represents the computed story overstrength. Consideration should also be given to placing an upper limit on Ω_S (perhaps as a fraction of Ω_0).

The inclusion of P-Delta effects in 3-D analysis requires an accurate representation of the spatial distribution of gravity loads (and the related geometric stiffness) in the mathematical model. As stated in the next section the preferred approach is to explicitly model all vertical elements, including gravity columns, and to assign the required geometric stiffness to these elements. An alternate approach is to use a few distributed “leaner columns”. However, the determination of the location and axial forces for each of the leaner columns is likely more complex than is modeling the actual columns directly.

4. Direction of Loading and Orthogonal Load Effects

Where the structural analysis is performed in 3 dimensions, it should be required that orthogonal load effects (subparagraph “a” of Section 12.5.3) are considered. This can be done automatically by software, and including the orthogonal load effect is, from a human perspective, less onerous than making the determination of whether or not it is needed (e.g. Section 12.5.4).

For ELF analysis the 100% / 30% procedure should be used. For MRS, it is recommended that the SRSS of the two horizontal results (each combined using CQC) be used. Currently, for LRH, it is required to apply the two horizontal components of spectrally matched ground motion simultaneously. In the future this requirement should be related to ground motion component correlation factors, as done in the analysis of nuclear structures (USNRC 2014).

5. Dynamic Effects

In the future, the use of ELF should be discouraged for use in the final design of all but the simplest structural systems. However, the method is necessary for preliminary design, and is needed for assessing the presence of torsional irregularities (unless alternate dynamic methods based on modal properties are developed). While the LRH method offers significant advantages over the MRS method, the MRS method should be the default method in the next version of ASCE 7. However, movement towards LRH, and perhaps even a simplified version on nonlinear response history analysis should be considered as the default in the not too distant future.

6. Accidental Torsion (See also Research Effort 2)

Where ELF analysis is used, accidental torsion must be modeled by application of static story torques, possibly amplified for dynamic effects. For MRS or LRH analysis, the static force approach may be used, or the center of mass may be adjusted (without amplification). In ASCE 7-16, there are no specific requirements for the approach taken for including accidental torsion in MRS analysis. However, for LRH analysis, it is stipulated that the center of mass adjustment method must be used. It is recommended that the center of mass adjustment approach be required where MRS is used. This provides consistency with LRH. It is recognized, however, that the approach is awkward if the software does not automatically handle the mass adjustment, and/or if the software does not allow for the combination of results using models with different modal properties.

A secondary consideration is the current requirement that accidental torsion need be amplified in static analysis, but not in dynamic analysis. The commentary of the 1990 SEAOC "Blue Book" (1990) states that the purpose of the torsional amplification factor A_x "is intended to represent the increased torsional eccentricity caused by yielding of perimeter elements". If this is true, it is not clear why in ASCE 7-16 the torsional amplification factor is not applicable in dynamic analysis. Also, it is noted that several code cycles ago the factor was applied to both inherent and accidental torsion, but more recently it is applied only to accidental torsion.

7. Determination of Deflections at the Edge of the Structure

Section 12.8.6 requires that drifts be checked at the edge of the building if a torsional irregularity exists. This requirement should be applied to all systems, regardless of the presence of an irregularity. Additionally, consideration should be given to the use of deformations that are more aligned with damage than is inter-story drift. Figure 2, from Aswegan et at. (2015) shows a building model in which shear strains in rectangular "Drift Damage Zones" are used as the damage measure. These are represented with "zero stiffness" elements and are easily implemented in 3D structural Analysis.

8. Modeling the Gravity System

It is recommended that all gravity columns be explicitly included in the analytical model. These columns should be placed in their actual locations, but should be modelled such that they cannot resist lateral shear forces. The gravity beams need not be modeled, but it may be convenient to do so to provide a realistic load path for transfer of loads to the columns. Gravity beams should not transfer moment into the gravity columns. As discussed in the previous section, modeling the gravity columns in this manner greatly facilitates the implementation of 3-dimensional P-Delta effects.

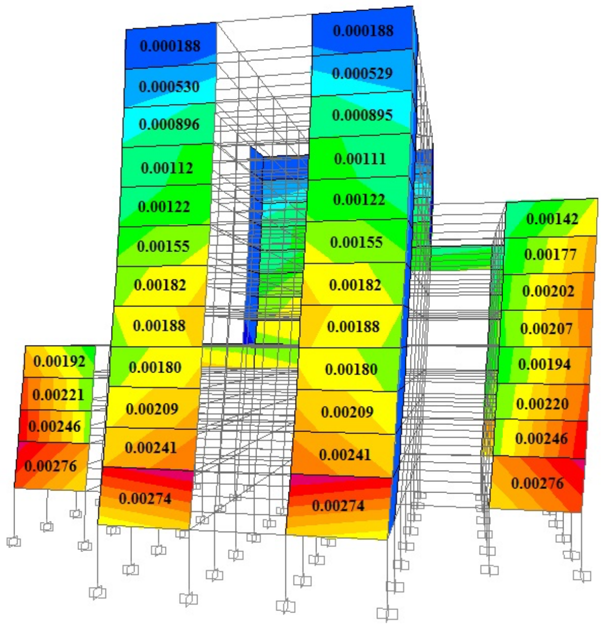

Figure 2. Using of Shear Strain as a Damage Measure

2. Other Issues that Need Addressing in Chapter 12 of ASCE 7-16

Minimum Base Shear for Computing Drift

Section 12.8.6.1 requires that lateral forces based on Eq. 12.8-6 be used in computing drift when the same equation controls base shear. This requirement should be revisited as the displacements increase quadratically for longer period systems.

Period for Computing Drift

Section 12.8.6.2 allows drifts to be computed on the basis of lateral forces developed using the computed period of vibration, without the upper limit C_uT_a. This seems to be in conflict with 12.8.6.1 when Eq. 12.8-6 controls the base shear. Additionally, and more importantly, there are concerns that computed periods are often much larger than C_uT_a. This is illustrated in Table 1, which presents a range of C_uT_a values for a variety of steel structures analyzed in the ATC 76 Project (NIST 2010). Also shown in the table is the ratio of flexibility $(T_{computed} / C_uT_a)^2$. As may be seen, the flexibility ratios are generally greater than 2, and in the case of the 16-story CBF designed using MRS, the ratio is 6.51. Such a ratio is not uncommon, and is very difficult to justify. Is the C_uT_a period too low, or is the computed period too high?

It is noted that for the 20-story SMF designed using ELF the flexibility ratio is 0.47, which seems to be an anomaly. It is noted, however, that all the ATC-76 systems were designed using C_d=R, and are thereby are more likely to be drift controlled than when traditional C_d values are used. This means than for many of the systems in Table 1 the flexibility ratio computed for systems designed under ASCE 7-16 would be even greater for some systems.

Table 1. Empirical and Computed Periods for A Selection of Frames Analyzed for ATC 76

System	Number of Stories	Design Basis	SDC	C_uT_a	$T_{computed}$	Flexibility Ratio
Steel CBF	2	ELF	Dmax	0.28	0.55	3.86
Steel CBF	6	ELF	Dmax	0.88	1.51	2.94
Steel CBF	12	ELF	Dmax	1.47	3.04	4.28
Steel CBF	16	MRS	Dmax	1.83	4.67	6.51
Steel BRB	2	ELF	Dmax	0.40	0.50	1.56
Steel BRB	6	ELF	Dmax	1.31	2.34	3.13
Steel BRB	12	MRS	Dmax	2.21	3.49	2.49
Steel BRB	16	MRS	Dmax	2.74	4.83	3.11
Steel SMF	2	ELF	Dmax	0.56	0.87	2.41
Steel SMF	4	ELF	Dmax	0.95	1.30	1.87
Steel SMF	20	ELF	Dmax	3.37	2.48	0.54
Steel SMF	2	MRS	Dmax	0.56	0.91	2.64
Steel SMF	4	MRS	Dmax	0.95	1.62	2.90
Steel SMF	12	MRS	Dmax	2.25	3.12	1.92
Steel SMF	20	MRS	Dmax	3.37	4.47	1.76

Additional research is needed to determine the reasons for the wide variations in empirical and computed period (and flexibility ratio). Until this can be done, it is recommended that an upper limit be placed on the period that is used for computing drift. A limit of $1.4C_uT_a$ is suggested. If this is done, drifts computed using MRS or LRH analysis would need to be scaled to drifts that are computed using ELF analysis with forces based on T=$1.4C_uT_a$.

Use of Importance Factor in Chapter 12

The manner in which the importance factor, *Ie*, is used in ASCE 7-16 is inconsistent and illogical. In equations 12,8-2, 12.8-3, 12.8-4, and 12.8-6 I_e appears in the denominator of the denominator (R/I_e), which makes it appear that it is a modifier on R, which it not because R is a system detailing parameter that is independent of I_e. If the term is moved to the numerator in these equations, I_e appears as a multiplier on ground motions (e.g. $S_{DS}I_e$ in Eq. 12.8-2), which it is not because the seismic hazard is independent of I_e. It is noted however, that I_e could be interpreted as a multiplier on seismic hazard in Equation 12.9-5 (I_e had to be placed here because there is no R term in the equation).

The inclusion of I_e in equations 12.8-2 to 12.8-6 requires that it be placed in the denominator of equations 12.8-15 (and 12.12-1) because allowable drifts in Table 12.12-1 are a function of risk category (or of I_e since I_e is directly related to risk category). I_e appears in the numerator of equation 12.8-16 to cancel out the fact that it is included in V_x.

The real purpose of I_e is to provide an intended overstrength in the structure, which in association with the reduced drift limits in Table 12.12-1 (for some systems) has the effect of reducing the ductility demand, and thereby limiting damage due to inelastic deformations. Recognizing this, it is recommended that I_e be removed from equations 12.8-2, 12.8-3, 12.8-4, 12.8-5, 12.8-6, 12.8-12, 12.8-16, and 12.12-1, and instead be placed as a multiplier on the term Q_E wherever Q_E occurs in a load combination. This significantly simplifies the code, thereby reducing the potential for error.

Research Needs

1. P-Delta Effects

There is some inconsistency in the manner in which P-Delta effects are handled in the linear analysis procedures of Chapter 12 of ASCE 7-16. In Section 12.7.3 it is stated that "the mathematical model shall be constructed for the purpose of determining member forces and structural displacements resulting from applied loads and any imposed displacements or P-Delta effects". This implies that P-delta effects should be included in the mathematical model, but the procedures in 12.8-7 are based on analysis without P-Delta. (It is noted that Section 12.8.7 provides an adjustment to equation 12.8-17 when P-delta effects are included). Additionally, equation 12.7-16 of ASCE 7-16, as written, is applicable only to 2-D systems and does not capture the potentially important "P-Theta effect" (Flores, et al. 2015).

In the Full System Modeling concept described earlier in this summary, it is recommended that P-Delta effects be included directly in the analysis. In ASCE 7-16, this is already a requirement for LRH analysis, but not for ELF and MRS analysis. While direct inclusion of P-Delta effects seems to be rational requirement, there is little knowledge of what the consequences are of this approach compared to the post-facto approach used in Section 12-8.7. Specific issues of concern are:

- For ELF analysis, are the incremental differences member forces and displacements obtained from models that directly incorporate P-Delta effects similar to those obtained using the post-facto approach?
- In MRS analysis and in LRH analysis performed using modal superposition, are the incremental differences in member forces and displacements obtained from models that directly incorporate P-Delta effects similar to those obtained using the post-facto approach? It is noted that in such analysis the effects of P-Delta are represented only in the modal properties, which in turn are based on a constant initial geometric stiffness.
- Where LRH is run using direct integration, are the incremental differences in member forces and displacements obtained from models that directly incorporate P-Delta effects similar to those obtained using the modal LRH approach?

A much broader issue is whether or not including P-Delta effects in elastic analysis is even reasonable. There is significant evidence that it is not reasonable because residual displacements and possible dynamic instability cannot be represented in a linear model (Gupta and Krawinkler, 2000). This would be especially true for systems that had a very large stability index, which is possible in ASCE 7-16. Consider for example a special moment frame with C_d=5, and an expected story overstrength of 2.0, producing $\beta = 0.5$ in Equation 12.8-17. Thus, using Equation 12.7-17, $\theta_{\max} = 0.5/(\beta C_d) = 0.5/(0.5x5.0) = 0.2$. If the system being analyzed had such a large stability index, the computed displacements would, due to P-Delta effects, almost certainly increase by a factor greater than 1/(1-0.2)=1.25 relative to the analysis without P-delta effects.

In the 2009 NEHRP Provisions (FEMA 2010), it was recommended to eliminate Equation 12.8-17, and instead set $\theta_{\max}$ to 0.1, unless it could be shown by nonlinear static analysis, or nonlinear dynamic analysis, that a larger value could be tolerated. This approach was not adopted in ASCE 7-16 because it is most likely that θ values > 0.1 would occur primarily in the lower SDCs, and practitioners working in lower seismic hazard areas would not be equipped to perform peer reviewed nonlinear analysis that would be required to enable values of θ > 0.1. Also, it is not clear how P-Delta effects would be incorporated into linear analysis if θ > 0.1. It would seem that if θ > 0.1, the solution would be to prohibit the use of linear analysis, and require nonlinear analysis in accordance with Chapter 16. Again, this is not a practical solution, particularly if the analysis is performed using Chapter 16 of ASCE 7-16 which is, frankly, practical only for those with very significant expertise in nonlinear analysis.

In lieu of requiring the analyst to perform a pushover analysis or dynamic analysis in association with the 2009 NEHRP recommendation, it might be more practical to establish system-dependent limits on θ, where these limits are based on evaluation of a large number of analyses, such as performed using the FEMA P-695 (FEMA, 2009) procedure.

Clearly, the most rational method to include P-Delta effects in seismic analysis is to perform nonlinear response history analysis. A reasonable approach, which may be developed prior to the next edition of ASCE 7, is to develop a "simplified" version of nonlinear analysis that utilizes less complex nonlinear force-deformation models and fewer ground motions, and employs more conservative acceptance criteria to compensate for the reduced quality in the mathematical model.

2. Accidental Torsion

During the development of the new Chapter 16 in ASCE 7-16 it was decided to include accidental torsion in the analysis where the structure has a Type 1a or 1b torsional irregularity. The accidental torsion is to be included by displacing the center of mass 5% of the building dimension perpendicular to the direction of load. This leads to four accidental torsion cases for each ground motion analyzed, which may be an onerous requirement for some systems. The requirement to include accidental torsion was based in part on research performed by Flores et al. (2015).

The main sources of accidental torsion are (1) uncertainty in locating center of rigidity and center of mass, (2) effect of migrating center of rigidity during inelastic response, and (3) torsional components of ground motion.

In nonlinear analysis, only items (1) and (3) are present because the migration of the center of rigidity migration will occur automatically as the system components yield. This would seem to indicate that the 5% eccentricity used in linear analysis is perhaps excessive for nonlinear analysis. Additionally, it would seem that on the average, the influence of the uncertainty on center of mass location would be reduced for taller buildings because it is unlikely that the "error" in locating the center of mass would be aligned in the same direction along the height.

Given the above, an approach that might reduce the number of occasions for which accidental torsion must be considered in nonlinear analysis would be to reduce the accidental mass eccentricity to some value less than 5% of the building dimension. Additionally, it seems that the magnitude of the accidental eccentricity should reduce with building height. For example, the eccentricity could be 4% for buildings 5 stories in height or less, and reduce in some fashion to 2% at 30 stories, and stay at 2% above 30 stories.

Research would need to be performed to develop the actual numbers used. This could be based on uncertainty analysis of actual building properties. Once the values are developed they could be provided in a prescriptive manner, or procedures to develop the values in a building-specific manner could be developed (but not necessarily codified).

Alternate methods to reduce the number of accidental torsion cases, where required, could be based on pushover analysis. Such analysis can predict the sensitivity of the structure to torsional loading (Flores et al. 2015). Additionally, approaches based on applying torsional ground motions might be used in lieu of shifting the location of the center of mass (Basu, et al. 2014). It is noted that this approach is based not on the determination of the true torsional ground motion hazard, but rather on emulating the effect of sources of accidental torsion listed above.

3. Simplified Nonlinear Analysis

The concept of the R and C_d factors, which are based in principle on the "equal displacement concept", allows for the effect of inelastic behavior to be included in an ad-hoc manner. Fifty years ago this assumption was essential because it was virtually impossible to perform nonlinear dynamic analysis, even for the simplest structures. Today, nonlinear analysis is available on many commercial analysis platforms. These programs provide features that simplify the cumbersome task of developing a force-deformation relationship for those portions of the structure that are expected to respond in the inelastic range during the maximum considered earthquake. Given these current features, and projecting the capabilities of software that might be available in the next decade, consideration should be given to the development of "simplified" analysis and design procedures that explicitly account for inelastic effects, rather than relying on linear analysis in concert with R and C_d. One of the key advantages of this approach is that it provides a means to rationally include P-Delta effects, which is impossible when linear analysis is used. Additionally, the procedure would be more rational in the design of very short period systems for which the equal displacement concept is not applicable.

It could be argued that nonlinear procedures are already incorporated into Chapter 16, and that new procedures are not needed. On the other hand, it must be recognized that the procedures in Chapter 16 are quite advanced, and very significant expertise on the part of the analyst is needed in addition to mandatory peer reviews. In the simplified procedure, Chapter 16 would serve as the starting point, and the following modifications would be considered:

- The requirement to use spectrally matched ground motions, with these motions provided by a web-based utility that requires only latitude and longitude and site class to develop an appropriate suite of motions. All analysts using the simplified procedure would use the same set of ground motions, matched to the ASCE 7 spectrum. The number of ground motions to use should be debated, and the necessity to explicitly account for near-field effects should be considered;
- Simplified force-deformation relationships would be used for displacement-controlled elements, with degrading strength and stiffness not required if it is shown that the deformations do not exceed given values that are related to the structural system being analyzed. For example, non-degrading relationships could be used for special steel moment frames if MCE_R drift ratios are less than 0.03;
- The method would only be applicable to systems for which accidental torsion need not be included in the analysis. Based on the results from research effort 2 (accidental torsion), there might be a requirement that the system be exempted from the in Chapter 16 requirements to include accidental torsion;
- Other restrictions might be enforced, such as allowing only rigid diaphragms, restricted ratios of dead load to live load, restricted building height, other restricted irregularities; and
- Simplified but possibly more restrictive acceptance criteria, which ensures that the system has sufficient reserve ductility.

The simplified method would be included as a separate methodology in the next edition of ASCE 7 (Section 16.1 is the current nonlinear analysis; section 16.2 is simplified nonlinear analysis).

The methodology would be validated (and possibility calibrated) by extensive analysis of a design space of realistic systems. A precedent for this type of effort is the development of the Direct Design method in the design of steel structures, wherein simplified ad-hoc methods (k-factors) were eventually eliminated by use of modern analysis procedures.

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GEOTECHNICAL/GROUND MOTION CONSIDERATIONS

By C.B. Crouse, URS Corporation; Robert Bachman, RE Bachman Consulting Engineers; Nicolas Luco, U.S. Geological Survey; Gyimah Kasali, Rutherford + Chekene; and Robert Pekelnicky, Degenkolb Engineers

Future Provisions Issues

1. Multi-period MCE_R response spectra should be developed for each grid point in the USGS national seismic hazard model. In these spectra, the effects of the following should be included: (1) Various local soil geologies from rock (Site Classes A & B) to soft soil (Site Class E); and, (2) Deep basins in urban areas, such as Los Angeles, the Bay Area, Seattle, and Salt Lake City, where sufficient subsurface seismic velocity data are available to model the effect. In the calculation of multi-period MCE_R response spectra, ground-motion prediction equations should be used that cover a broad natural period range, as do the NGA West2 equations, which cover the period band from 0 to 10 sec. The NGA West2 equations, applicable to shallow crustal earthquakes in the Western US, were updated in 2013. Equations applicable to the Eastern US and the subduction zones in the Pacific Northwest and Alaska are being updated to cover a broad period range. To include the effects of various local soil geologies and deep basins, significant research needs remain. The multi-period response spectra will require significant changes to other chapters of ASCE 7 where the terms S_{DS} and S_{D1} are used.

2. The current approach used to derive ground motions for design from scientific estimates of seismic hazard should be reviewed in light of constantly evolving seismic hazard models and their inherent uncertainties. The importance of stability in the design ground motions should be considered, in addition to the appropriateness of the degree of precision (i.e., number of decimal places) required by current design provisions, such as those related to the definition of seismic design categories.

3. The current definition of the deterministic ground motions that cap MCE_R values near very active faults should be reviewed, given the recent move away from the concept of "characteristic" earthquakes in the Uniform California Earthquake Rupture Forecast (UCERF). For the 2015 NEHRP *Provisions*, the same characteristic earthquakes considered for the 2009 *Provisions* were used to calculate deterministic ground motions, but it was recognized that there are numerous other UCERF3 earthquakes that could be considered in future updates.

4. Resource Paper 12: *Evaluation of Geologic Hazards and Determination of Seismic Lateral Earth Pressures* in Part 3 the 2009 NEHRP *Provisions* should be updated to capture new developments in the area of geologic hazard evaluation and determination of seismic lateral pressures.

5. Work is needed to extend the inertial interaction provisions to deep foundations.

Research Needs

1. An NGA project for the subduction-zone earthquakes that govern the hazard in the Pacific Northwest and southern Alaska is needed. The need for this project was identified during the ATC-35 Ground Motion Mapping Workshop in December 2006. The development efforts need to be coordinated and managed so that the resulting equations can be used for updating the ground-motion hazard in these two regions, where magnitude M8-9+ earthquakes have occurred roughly every several hundred years. With the NGA updates for the eastern United States and the subduction zones, it will be possible to add a new set of maps for very long period motions so that the T_L maps currently in ASCE/SEI 7 can be removed.

2. Outside of the portions of the Western U.S., where shallow crustal earthquakes govern the seismic hazard, research is needed to include the effects of various local soil geologies and deep basins in MCE_R response spectra. The NGA West2 ground-motion prediction equations applicable to shallow crustal earthquakes already capture these effects. The current equations for subduction-zone and Eastern US earthquakes require further research in this respect.

3. It is widely acknowledged that the uniform-hazard shape of design and maximum considered earthquake spectra is conceptually not the most appropriate shape for the target spectrum used to select and modify acceleration histories. This issue might have been exacerbated by the introduction of risk-targeted ground motions and maximum-direction spectral response acceleration. Research is needed to define a more appropriate target spectrum, such as a conditional mean spectrum. To a lesser extent, research on more appropriate selection/modification criteria and a better justified number of acceleration histories also might be warranted. Research on this issue was started in the 2015 cycle, and the use of conditional mean spectra as target spectra is allowed by the revised Chapter 16.

4. Throughout the process of updating the soil structure interaction (SSI) provisions in Chapter 19, a number of issues were identified that require additional research. The most significant issue is the need for greater understanding of how SSI interfaces with structural yielding. Both inertial and kinematic SSI have been shown through theory, research studies, and post-earthquake assessments to reduce the inertial forces imparted to a structure that responds elastically. The previous edition of the *Provisions* had inertial SSI as an additive reduction to the reduction in forces due to structural

yielding. It was postulated that this is not the case and the reduction was reduced if significant yielding was anticipated in the structure. Additionally, kinematic SSI was not allowed to be used in linear procedures due to its unknown interplay with structural yielding. Further research into the amount of reduction due to SSI that can be added to reductions in design forces due to structure yielding would help to reduce or validate the conservative restrictions placed on these procedures.

CONCRETE STRUCTURES

By Andrew Taylor, KPFF Consulting Engineers; Neil Hawkins, University of Illinois; and S. K. Ghosh, S. K. Ghosh Associates

Future Provisions Issues

1. There is a major need for an effort to improve the seismic design of reinforced concrete shear walls. The Canadian Standards Association Standard A23.3, *Design of Concrete Structures*, the counterpart of ACI 318 and the National Building Code of Canada, contain much more comprehensive provisions for such design than the U.S. codes and standards. CSA A23.3 distinguishes between ductile and less ductile walls, between squat and slender walls, and, very importantly, recognizes coupled shear walls.

2. Seismic force-resisting systems incorporating coupled shear walls are highly suitable for multistory reinforced concrete buildings. Yet, they are not recognized as distinct entities in Table12.2-1 of ASCE 7-10. Therefore, such systems currently are designed using *R*-values that essentially ignore the considerable benefits of having coupling beams, which can dissipate much of the energy generated by earthquake excitation. A convincing case can most likely be put together, based on available information, for an increase in *R*-value for a seismic force-resisting system featuring coupled shear walls, over that for an identical system without the coupling beams. However, these days, FEMA P-695 studies (FEMA P-695 Quantification of Building Seismic Performance Factors) are often deemed necessary in order to justify adding a "new" system and associated seismic design parameters to Table 12.2-1. In the case of coupled shear walls, discussion needs to focus on whether it is a new system, and whether seismic design parameters can be justified in absence of FEMA P-695 studies. One strategy might be to define coupled shear wall characteristics (R, Ω, C_d, etc.) based on a systematic, quantitative, study of the observed performance of coupled shear walls in past earthquakes.

3. Consensus is needed on what portion of the gravity load reinforcing steel in a slab can be used as diaphragm reinforcement. ASCE 7 is explicit about which combinations of gravity loads are required to be considered in design in combination with seismic loading. This can apply to the design of the floor slab/diaphragm shear reinforcement, and for the chord reinforcement that might be part of a perimeter beam.

4. While some engineers take advantage of gravity steel to limit the amount of additional diaphragm shear or chord reinforcement, other engineers do not. For the moment, this decision should probably continue to be left to the engineer's discretion. However, it must be recognized that any added chord reinforcement in a perimeter beam will increase the shear demand in that beam, and will lower the column-to-beam flexural strength ratio. Mandatory requirements for consideration of these aspects need to be developed.

5. The Precast Concrete Diaphragm Connector and Joint Reinforcement Qualification Procedure, and the precast concrete diaphragm design methodology itself, will need to be refined in view of experience gained from their usage.

6. With more and more recycling, steel strengths are increasing, and CRSI and other groups are encouraging the use of Grade 75 and higher reinforcement. But ductility is a concern, and strain compatibility needs to be considered.

Research Needs

1. In the process of selecting an appropriate R_s value for cast-in-place reinforced concrete diaphragms, no test results were found to be available. Instead, extrapolations were made from limited shear wall tests that appeared to have some relevance. As a result, an R_s value was assigned to cast-in-place concrete diaphragms, which was acknowledged to be quite conservative. It needs to be determined whether there currently exists information that can serve as the basis for further examination of R_s values. If not, the testing information required should be identified.

2. In addition to testing cast-in-place diaphragms, analytical studies are needed to derive or verify R_s factors based on global ductility and displacement capacity. This research should consider buildings with well-spaced structural walls, as well as buildings with a central core, or similar buildings in which inertia forces are collected by only few elements and connected to vertical elements of the seismic force resisting system.

3. Further research is needed on the constraints imposed on shear connection designs by the precast diaphragm seismic design methodology, which are intended to remain elastic in the MCE through application of the shear overstrength factor, and on ductile mechanical parts in a connection that transfer shear, and on the shear overstrength factor Ωv. Connectors that could provide improved diaphragm connection performance under strong seismic excitation need to be developed and qualified in accordance with the qualification procedure being developed by ACI Committee 550 – Precast Concrete Structures.

4. Design requirements for tilt-up wall systems are based primarily on data for roof diaphragm systems constructed using wood structural panel sheathing in combination with wood framing. Many modern tilt-up systems use other roof diaphragm systems, and in particular, untopped cold-formed steel deck in combination with a variety of framing member and fastener types. Seismic design requirements for the walls of such structures, and the anchorage of its walls to the diaphragms, need correlation with the structural performance measured in recent earthquakes.

5. Seismic design requirements for concrete piles are well defined. However, few test data are available and detailed design requirements are needed for the anchorage of piles to pile caps, mats, and other foundation systems.

6. Studies are desirable on the seismic performance of lightweight concrete structures with specified concrete strengths greater than the 5 ksi limit currently imposed by ACI 318.

MASONRY STRUCTURES

By Richard Bennett, University of Tennessee; Jason Thompson, National Concrete Masonry Association; and Phillip Samblanet, The Masonry Society

Future Provisions Issues

1. Seismic design procedures are needed for masonry shear walls with irregular opening configurations.

Research Needs

1. Research is needed to provide experimental and analytical verification of the hysteretic behavior of:

    - Masonry shear walls with different aspect ratios, axial loads, and configurations of prescriptive reinforcement;
    - Partially grouted shear walls;
    - Dry-stack masonry shear walls;
    - Pre-stressed masonry shear walls; and
    - Masonry shear walls with confined boundary elements.

2. Research is needed on masonry structures constructed with lightweight grout, particularly, with respect to development/splice lengths and shear strength.

STEEL STRUCTURES

Structural Steel Recommendations by Rafael Sabelli, Walter P. Moore and Associates, and James Malley, Degenkolb Engineers
Cold-formed Steel Recommendations by Bonnie Manley, AISI, and Kelly Cobeen, Wiss Janney Elstner Associates

Future Provisions Issues

1. A comparison of various approaches is needed for establishing building period for design, including parameters such as limit-state stiffness, as used in the direct analysis method, and second-order effects. This comparison should assess building performance and drift prediction. The study, if successful, could be extended beyond structural steel to other materials.

2. The $R = 3$ approach is only allowed for steel systems. There are some who suggest this also should apply to composite systems. This should be looked into with pros and cons thoroughly examined.

3. Realistic bounds for prequalification of structural steel moment frame connections need to be established, and attention to top-of-column conditions, sloped and skewed conditions needs to be mandated. There is no guidance in AISC 358 or AISC 341 on how engineers should deal with sloped (usually in roof conditions) or skewed (non-orthogonal) connections.

4. Work related to FEMA P-695 studies of Seismic Design Parameters (SDPs) for cold-formed steel light frame seismic force-resisting systems (SFRS) is needed to provide guidelines to FEMA P-695 users on the judgments made when attempting to apply the methodology to cold-formed steel light-frame SFRS. These guidelines should include consideration of attributes of the archetypical designs, as well as the number of them, and how to characterize model, data, and design method quality. Guidance also is needed for those reporting results of a FEMA P-695 study, so that readers understand the important judgments made on all of the above, and on more detailed aspects of the design basis.

5. Use of mid-rise cold-formed steel light-frame construction is increasing rapidly in the United States and Canada. For this construction type, the adequacy of formulas for fundamental period should be evaluated and corrected, if necessary, and accurate procedures for calculating deflections due to seismic loads should be developed.

Research Needs

1. The use of high-strength structural steel in seismic applications should be studied. The seismic applications and seismic requirements that might not be valid with high-strength

structural steel (e.g., slenderness limits, local buckling parameters, flange bracing, and yield to ultimate stress ratio) should be identified.

2. Research is needed on the seismic capacity of structural steel ordinary concentrically braced frames, and structural steel ordinary moment frames for a variety of configurations commonly used in buildings and nonbuilding structures designed with the *Provisions* in conjunction with AISC 341 for detailing. Relaxation of the height and other limitations for lower-ductility systems, such as ordinary concentrically braced frames, should be considered. Opportunities for such relaxations on limits for nonbuilding structures similar to buildings should be studied, perhaps in the context of FEMA P-695 analyses.

3. Data are needed on the behavior of long encased composite structural steel and concrete columns under cyclic loads, particularly, when high-strength steel or concrete is used. Additionally, data on the importance of the detailing of the transverse reinforcement on the performance of these columns are needed.

4. For concrete-filled structural steel tube beam-columns, more accurate axial, flexural, and interaction formulas are needed, particularly, with respect to the use of high-strength concrete and high-performance steel materials.

5. Experimental research is needed on structural steel moment frame systems that use member types other than typical H shapes. Hollow structural sections (HSS) used for both beams and columns in buildings and other structures, such as walkways and canopies, and different connection configurations should be considered. Results from Japanese research and applications will need to be considered in these studies.

6. Research is needed to establish a method for determining appropriate design forces for columns in multistory braced frames, and for structural steel plate shear walls (SPSW) based on linear analysis.

7. Research is needed to develop and validate a design method for structural steel special concentrically braced frame (SCBF) columns without lateral bracing at beam levels, such as a three-level frame with out-of-plane bracing only at top and bottom.

8. An investigation is needed to study the effect, if any, of attachments to protected zones such as flanges of shear-governed EBF (eccentrically brace frame) links, SCBF braces, and SPSW web plates.

9. The design of systems in which energy dissipation is focused in optimized replaceable energy-dissipating fuses should be examined. Ideally, such research will include self-centering capabilities to maximize the value of fuse replacement.

10. System design and detailing procedures are needed for structural steel and concrete composite structures in the low and moderate seismic design categories.

11. Structural steel construction details that might provide higher viscous damping or supplementary lateral strength and stiffness need to be developed.

12. Current approaches to the design of cold-formed steel deck diaphragms in multistory buildings, as well as in rigid frame, flexible diaphragm buildings, need further research. Applicability of the alternative diaphragm design approach in Part 1 of the 2015 NEHRP *Provisions* was not extended to cold-formed steel deck diaphragms, because available research was not sufficient to lead to an appropriate diaphragm design force reduction factor.

13. Cost-effective solutions for midrise buildings dictate the need for higher capacity cold-formed steel shear wall seismic force-resisting systems. This need could be met with hybrid systems utilizing both structural and cold-formed steel elements. It could also be met with the integration of very high strength steels (automotive steels) in seismic force-resisting systems (SFRS). However, research on seismic-force resisting systems that utilize different types of steel as well as an expanded understanding of the design space are needed.

14. Performance-based seismic design procedures are needed for cold-formed steel light-frame buildings that take into account the effect of nonstructural interior and exterior wall finishes. The CFS-NEES (Cold-Formed Steel – Network for Earthquake Engineering Simulation) project demonstrated that finish materials significantly influence the seismic performance of cold-formed steel light-frame buildings. However, meaningful guidance on how to consider these effects in building design needs to be developed.

15. Research is needed to provide guidance to designers on distribution of forces in the design of cold-formed steel light-frame buildings. Questions exist on whether seismic forces in cold-formed steel light-frame buildings should be distributed using flexible or rigid diaphragm assumptions, or whether some other solution is needed. Prior to the 2010 edition, ASCE 7 seismic provisions required semi-rigid analysis, which was both impractical and impossible for light-frame buildings. In ASCE 7-10 this has been changed to allow most light-frame construction to be designed using a flexible diaphragm idealization. This might be overly simplistic and might not result in good seismic performance. Guidance needs to be based on building performance resulting from practical analysis techniques. Research is needed to quantify performance.

16. Research is needed to assess the performance of and develop design guidance for cold-formed steel light-frame hillside construction. Concern has been voiced about both the torsional response of hillside dwellings due to significant differences in stiffness of uphill and downhill walls, and the performance of stepped or sloped cripple walls.

Research is needed to quantify at what slope or under what circumstances hillside dwellings become vulnerable, and to identify design approaches for reducing that vulnerability.

17. Research is needed to assess the performance of and develop design guidance for open-front cold-formed steel light-frame construction. While current seismic requirements still permit construction of this building configuration when certain conditions are met, research is needed to quantify at what point this configuration becomes vulnerable, and to identify design approaches for reducing that vulnerability.

18. Work related to FEMA P-695 studies of cold-formed steel shear wall systems is needed to:

    - Verify existing seismic design parameters and associated detailing requirements for cold-formed steel light-frame shear wall structures;
    - Review issues related to meeting margin of collapse criteria for short period buildings; and
    - Identify key variables to address in a "sensitivity" study methods defined in FEMA P-695 as they pertain to light-frame shear wall systems. This study should document expected results due to changes in system ductility, drift capacity, and overstrength. The results would have many uses including identifying critical aspects of system behavior that contribute significantly to reducing the collapse margin ratio, as well as providing an authoritative source of information for eventual users and product approval bodies.

19. Using FEMA P-695, the effects of soft/ weak stories on the performance of cold-formed steel light-frame construction should be evaluated, and design guidance to ensure performance of buildings prone to soft/ weak stories should be developed.

20. Testing and analysis should be provided to further develop capacity-based design procedures for cold-formed steel light-frame structures.

21. Seismic force and ductility demands on diaphragms in cold-formed steel light-frame buildings, and the adequacy of current design methods need to be better understood. Additional cyclic testing of full-scale diaphragms could verify hysteretic behavior. This information would allow more rigorous analytical verification of the alternative diaphragm design methodology included in Part 1 of the 2015 NEHRP *Provisions* and establish the diaphragm design force reduction factor for the types of diaphragm systems typically used in cold-formed steel light-frame construction.

WOOD STRUCTURES

By Kelly Cobeen, Wiss, Janney, Elstner Associates; and Philip Line, American Wood Council

Future Provisions Issues

1. Quantification of seismic performance and design coefficients is needed for heavy timber systems, such as timber braced frames.

2. Work related to FEMA P-695 studies of *R* factors for shear wall systems is needed to provide guidelines to FEMA P-695 users on the judgments made when attempting to apply the methodology to wood light-frame shear wall systems. These guidelines should include consideration of attributes of the archetypical designs, as well as the number of them, and how to characterize model, data, and design method quality. Guidance also is needed for those reporting results of a FEMA P-695 study, so that readers understand the important judgments made on all of the above, and on more detailed aspects of the design basis.

3. Use of mid-rise wood light-frame construction is increasing rapidly in the United States and Canada. For this construction type, the adequacy of formulas for fundamental period should be evaluated and corrected, if necessary, and accurate procedures for calculating deflections due to seismic loads should be developed. The formulas developed for low-rise construction are not considered representative of mid-rise construction.

4. The performance of wood light-frame shear walls as a function of the uplift deflection permitted at tie-down devices should be evaluated. Criteria should be developed for uplift limitations, as required, to ensure shear wall performance.

Research Needs

1. Research is needed to determine detailing requirements to achieve the intended seismic performance for light-frame shear walls. Resource Paper 11, *Shear Wall Load-Deflection Parameters and Performance Expectations*, in Part 3 of the 2009 NEHRP *Provisions* defined the load-deflection parameters and performance expectations for wood structural panel sheathed shear walls with wood framing to guide development of detailing recommendations. A conflict currently exists between the philosophies that detailing for overstrength should be provided, and the practical observation that much of the testing conducted to date has shown detailing without overstrength provisions to be adequate. Issue focused research is needed to determine whether current detailing practice can consistently provide adequate performance. The research should consider the range of wall configurations and sheathing materials permitted under current design

standards, and implications for both single-story and multistory walls. Detailing considerations should include both force and deformation.

2. Performance-based seismic design procedures are needed for light-frame buildings that take into account the effect of nonstructural interior and exterior wall finishes. The CUREE (Consortium of Universities for Research in Earthquake Engineering) and NEESWood (Network for Earthquake Engineering Simulation) projects and FEMA P-695 indicate that finish materials significantly influence the seismic performance of wood light-frame buildings. However, meaningful guidance on how to consider these effects in building design is lacking. (See Resource Paper 13, *Light-Frame Wall Systems with Wood Structural Panel Sheathing* in Part 3 of the 2009 NEHRP *Provisions*, FEMA P-750.)

3. Research is needed to provide definitive guidance to designers on distribution of forces in the design of light-frame buildings. Significant controversy exists between whether seismic forces in wood light-frame buildings should be distributed using flexible or rigid diaphragm assumptions, or whether some other solution is needed. Prior to the 2010 edition, ASCE 7 seismic provisions required semi-rigid analysis, which was both impractical and impossible for light-frame buildings. In ASCE 7-10 this has been changed to allow most light-frame construction to be designed using a flexible diaphragm idealization. This might be overly simplistic and might not result in good seismic performance. Guidance needs to be based on building performance resulting from practical analysis techniques. Research is needed to quantify performance.

4. Research is needed to assess the performance of and develop design guidance for wood light-frame hillside construction. The 1994 Northridge earthquake demonstrated the vulnerability of hillside dwellings with several collapses, and a number of damaged buildings. Concern has been voiced about both the torsional response of hillside dwellings due to significant differences in stiffness of uphill and downhill walls, and the performance of stepped or sloped cripple walls. Besides the hillside provisions in the City of Los Angeles Building Code, no design guidance has been provided to structural engineers, and vulnerable dwellings continue to be constructed. Research is needed to quantify at what slope or under what circumstances hillside dwellings become vulnerable, and to identify design approaches for reducing that vulnerability.

5. Research is needed to assess the performance of and develop design guidance for open-front light-frame construction. Significant performance issues were seen with open-front light-frame construction in both the Loma Prieta and Northridge earthquakes. While current seismic requirements still permit construction of this building configuration when certain conditions are met, research is needed to quantify at what point this configuration becomes vulnerable, and to identify design approaches for reducing that vulnerability.

6. Work related to FEMA P-695 studies of *R* factors for shear wall systems is needed to:

   - Evaluate FEMA P-695 methodologies and results as they relate to seismic coefficients for shear wall structures;
   - Review issues related to meeting margin of collapse criteria for short period buildings; and
   - Identify key variables to address in a "sensitivity" study methods defined in FEMA P-695 as they pertain to light-frame shear wall systems. This study should document expected results due to changes in system ductility, drift capacity, and overstrength. The results would have many uses including identifying critical aspects of system behavior that contribute significantly to reducing collapse margin ratio, as well as providing an authoritative source of information for eventual users and product approval bodies.

7. Using FEMA P-695, the effects of soft/ weak stories on the performance of light-frame construction should be evaluated, and design guidance to ensure performance of buildings prone to soft/ weak stories should be developed, which will require research.

8. Cost-effective methods of seismic retrofit for existing buildings with soft and weak first stories should be evaluated.

9. The seismic performance of retrofits for cripple wall and hillside buildings should be evaluated with consideration for both the adequacy of currently used retrofit methods, and potential new systems for building configurations not addressed by current methods (tall cripple walls, higher load walls, hillside conditions).

10. Testing and analysis to further develop capacity-based design procedures for wood and light-frame structures should be provided.

11. Seismic force and ductility demands on wood-frame diaphragms and adequacy of design methods need to be better understood. Issue Team No. 6 (IT06), that addressed diaphragm issues, investigated anticipated seismic demands for diaphragms ranging from near elastic to inelastic behavior. As a part of this effort, a limited analytical study was conducted, which showed that the significant displacement capacity of wood diaphragms, along with associated overstrength, tended to greatly reduce forces from those anticipated with near elastic diaphragm response. However, more rigorous studies are needed. To support such studies, additional cyclic testing of full-scale diaphragms would be of benefit to verify hysteretic behavior, and validate analysis models, since most test data currently available are monotonic, and do not necessarily capture peak strength and deformation capacity. This further information would allow more rigorous analytical verification of the alternative diaphragm design methodology and diaphragm design force reduction factor included in Part 1 of the 2015 NEHRP *Provisions*.

NONBUILDING STRUCTURES AND NONSTRUCTURAL COMPONENTS

By John Silva, Hilti; Robert Bachman, RE, Bachman Consulting Engineers; Greg Soules; Consulting Structural Engineer; and John Gillengerten, Consulting Engineer

During the development of the 2015 NEHRP *Provisions*, there was extensive discussion on establishing performance objectives for nonstructural components. Fundamental research is needed to relate nonstructural component design requirements to expected system performance. In addition, specific issues have been identified that significantly influence the performance of nonstructural components, but are not adequately covered in the current *Provisions*.

Future Provisions Issues

1. The mapped values of T_L require updating. Site-specific studies indicate that these values probably range from 4 to 6 seconds instead of 4 to 16 seconds. The value of T_L has a significant impact on the seismic freeboard, and the convective forces in tanks. This might be addressed by the Multi-Point Spectrum and Project 17.

2. Research on the behavior of large anchors under tension and shear loading should be reviewed and collated. Proposals need to be drafted for the improvement of current ACI provisions for defined cases, such as anchors in pedestals and shear lugs.

3. The performance of "pedestal" type systems typically used for coker structures in refineries should be studied, and a specific entry for this system in ASCE 7-10 Table 15.4-2 should be developed. A typical coker structure will have 2, 4, 6, or more coke drums supported on a very thick slab supported on legs. Currently, this system is treated as an ordinary moment frame, but generally behaves significantly better than an ordinary concrete moment frame in seismic events.

4. The component response modification factor R_p is principally based on the concept of "ruggedness". A rationale and definition should be developed to better describe the concept and provide a vehicle for development of rationally based R_p values.

Research Needs

1. Provisions for preservation of means of egress, including requirements for doors and ceilings, should be established, which might involve studying the response of exit components, such as doors to transient and residual story drift, and development of egress system components that will remain operable following a strong earthquake, while meeting fire protection and security requirements.

2. Displacement demands for nonstructural components and systems should be reviewed. In the current *Provisions*, drift-controlled components are required to "accommodate"

story drift, but there is little guidance on the meaning of this term. For example, how much inelastic behavior (damage) to the component is acceptable? Performance requirements for essential (Risk Category IV) versus other types of structure also should be studied.

3. Design requirements for piping systems should be reviewed, including studying nozzle loads, and reconciling mechanical and structural design requirements. R_p values for piping systems (e.g., ASME B31 and ASCE/SEI 7-10) should also be reviewed.

4. Provisions should be further developed to address 1) Potential adverse interactions between nonstructural components and other portions of the structure; 2) Determination of generic relative displacement between points of attachment for distributed systems such as piping, and 3) Enhancement, as necessary, of requirements to preclude inadvertent sprinkler activation and/or wet system pipe rupture during earthquakes.

5. A rationale and recommendations should be developed for revising the current 25% rule for establishing the boundary between nonstructural components and nonbuilding structures.

6. Experimental investigation of content falling hazard posed by palletized rack systems, such as those found in big-box stores, should be conducted.

7. Research is needed on the performance of floating roofs over tanks during seismic events. Floating roofs are used to cover volatile petroleum products to reduce the possibility of fire. No specific design procedures exist for floating roofs under seismic loads. Some floating roofs perform well, while others sink usually followed by fire.

8. Research is needed on the performance during seismic events of large-bore (diameter) piping systems used in industrial facilities to connect various structures. Large-bore piping systems common in refineries and chemical plants are currently treated as nonstructural components. But, such systems have significant stiffness and behave as structural systems spanning between adjacent structures. A study should include the interaction between these piping systems and the structures to which they are connected.

The following research issues might be addressed by ATC-120:

9. Generic design earthquake in-structure floor spectra for buildings and nonbuilding structures should be developed to create rational seismic qualification procedures and consistent simplified nonstructural equations.

10. Overstrength and displacement demand requirements for anchorage should be reviewed, including anchorage response/degradation that might be much more influential than the component response itself. This is necessary to create rational

seismic cyclic testing requirements and consistent simplified nonstructural equations for relative displacement demands.

11. Available records for shake table testing of nonstructural components and systems should be examined and recommendations developed to improve design based on the tests.

12. Recent instrumented building records from strong earthquakes should be examined for a comparison of recorded nonstructural component response to response predicted by the *Provisions*. If instrumented nonbuilding structures are found, it will be worthwhile comparing their recorded response with response predicted by the *Provisions* as well.

13. Performance expectations should be developed for nonstructural components at several levels of earthquake motion. Performance levels provided by the current *Provisions* at different shaking intensities should be assessed to determine whether changes are needed to meet performance objectives.