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Evaluation of ASCE 4-16 and ASCE 43-18 (Draft) for Use in the Risk-Informed, Performance-Based Seismic Design of Nuclear Power Plant Structures, Systems, and Components

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Disclaimer

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ABSTRACT

This report describes an assessment of American Society of Civil Engineers (ASCE) Standards 4-16, “Seismic Analysis of Safety-Related Nuclear Structures,” and 43-18 (Draft), “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities,” for use in the risk-informed, performance-based (RIPB) seismic design of structures, systems, and components (SSCs) at nuclear power plants. This work was performed for the U.S. Nuclear Regulatory Commission’s (NRC’s) Office of Nuclear Regulatory Research, to support potential endorsement of these industry standards for the design of nuclear power plants based on the RIPB approach. Currently, the NRC endorses a deterministic approach for demonstrating the design adequacy of SSCs, based on NUREG-0800, “Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition,” and NRC regulatory guides. In the RIPB approach, the design criteria are developed to achieve a target performance goal, which is defined by the annual frequency of occurrence of the design-basis earthquake (i.e., seismic design category) and the acceptable level of structural performance (i.e., limit state) for the SSCs.

ASCE 4-16 provides methods for performing seismic analysis of structures to obtain their seismic responses (e.g., building displacements, accelerations, in-structure response spectra), which are used in the design of the SSCs. ASCE 4-16 also provides methods for performing seismic analysis of SSCs to determine the seismic demands (e.g., member forces and displacements) needed to design individual SSCs. ASCE 43-18 (Draft) provides the criteria for the seismic design of SSCs using the seismic demands developed in ASCE 4-16. In turn, ASCE 43-18 (Draft) relies on other consensus codes and standards, such as the American Concrete Institute’s ACI 349-13, “Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary,” for reinforced concrete; the American Institute of Steel Construction’s AISC N690, “Specification for Safety-Related Steel Structures for Nuclear Facilities,” for steel structures; the American Society of Mechanical Engineers Boiler and Pressure Vessel Code, Section III, “Rules for Construction of Nuclear Facility Components,” for pressure-retaining mechanical components and containments; and the Institute of Electrical and Electronics Engineers standard IEEE 344, “IEEE Standard for Seismic Qualification of Equipment for Nuclear Power Generating Stations,” for Class 1E equipment.

The goal of this technical review is to assess whether the provisions in these standards are adequate for use by the NRC in developing regulatory guidance for the seismic design of SSCs in nuclear power plants, based on the RIPB approach. The research reported herein describes the basis for acceptance of the new standards and identifies areas where additional staff guidance is needed.

This technical review has determined that ASCE 4-16 and ASCE 43-18 (Draft) provide an appropriate framework for the seismic design of SSCs at nuclear power plants using an RIPB approach. However, some of the criteria in these standards warrant exceptions, qualifications, or clarifications.

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1 EXECUTIVE SUMMARY/INTRODUCTION

In August 2018, the U.S. Nuclear Regulatory Commission's (NRC's) Office of Nuclear Regulatory Research requested technical assistance from Brookhaven National Laboratory (BNL) to assess the suitability of two standards by the American Society of Civil Engineers (ASCE) and Structural Engineering Institute (SEI) for implementation as part of a new risk-informed, performance-based (RIPB) seismic design methodology for commercial nuclear power plants:

- ASCE/SEI 4-16, "Seismic Analysis of Safety-Related Nuclear Structures" (hereafter referred to as ASCE 4-16)
- ASCE/SEI 43-18 (Draft), "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities" (hereafter referred to as ASCE 43-18)

1.1 Objective

This review aims to verify the adequacy of the criteria in both ASCE standards, as well as to identify areas where additional guidance from the NRC staff is needed and to provide recommendations to address such areas. These recommendations will enable the NRC to develop regulatory guidance for appropriate implementation of these ASCE standards in the RIPB seismic design process. The staff anticipates issuing a regulatory guide (RG) defining the complete RIPB seismic design process as an alternative to the current guidance in NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition" (SRP).

1.2 Approach

Both ASCE 4-16 and ASCE 43-18 apply to the seismic analysis and design of SSCs at nuclear facilities, which include facilities that process, store, or handle radioactive materials as well as commercial nuclear power plants. The scope of the current review of these two ASCE standards is limited to their applicability to commercial nuclear power plants.

ASCE 4-16, published in 2016, is an update of the prior standard ASCE 4-98. The version of ASCE 43-18 reviewed in this report is a draft revision of ASCE 43-05, identified as Revision M, dated October 23, 2018.

The NRC's regulatory guidance has not generically endorsed prior versions of ASCE 4. However, it has identified selected criteria in ASCE 4-98 as acceptable methods or as additional acceptable guidance, specifically in SRP Section 3.7.3, "Seismic Subsystem Analysis," for the seismic analysis of aboveground tanks and buried pipes or conduits; SRP Section 3.8.1, "Concrete Containment," for modeling stiffness of concrete; and SRP Section 3.8.4, "Other Seismic Category I Structures," for lateral earth pressure and earth-retaining walls.

In the case of ASCE 43, BNL reviewed the 2005 version in a prior research project for the NRC. NUREG/CR-6926, "Evaluation of the Seismic Design Criteria in ASCE/SEI Standard 43-05 for Application to Nuclear Power Plants," issued February 2007, documents the results of that review. However, the review was limited to determining whether ASCE 43-05 would be acceptable for the design of commercial nuclear power plant SSCs similar to those covered by NRC regulatory guidance (i.e., the SRP and RGs) existing at the time of the review. Therefore, only the acceptance criteria applicable to Seismic Design Category (SDC)-5 and Limit State D were reviewed. In SDC-5, the target performance goal is set to $1x10^{-5}$/year, and Limit State D corresponds to essentially elastic behavior. The acceptance criteria in ASCE 43-05 dealing with inelastic energy absorption factors applicable to Limit States C, B, and A were not reviewed.

Implementation of the RIPB approach to the seismic design of SSCs for commercial nuclear power plants will rely heavily on insights gained from generic and facility-specific seismic probabilistic risk assessments (SPRAs). Assigning a single SDC to a facility would ensure a common facility wide seismic design basis and minimize the possibility of losing design control. The facility SDC would be specified by experts in the assessment of public risk resulting from potential radiation releases from the facility. Existing commercial nuclear power plants represent the highest risk and are consequently assigned to SDC-5. SPRAs can provide critical information for selecting an acceptable limit state for safety-significant SSCs. By developing fragilities corresponding to multiple limit states, the SPRA analyst can test the sensitivity of the results to shifts in an SSC fragility curve, then assign to the SSC the lowest limit state that produces acceptable results. While this process is straightforward for a single SSC, it becomes complicated for multiple SSCs that must function simultaneously. Consequently, parametric studies will likely focus on SSCs that offer the potential for significant life-cycle cost savings.

This project's evaluation of ASCE 4-16 and ASCE 43-18 encompassed review of four areas:

(1) the seismic design criteria contained in the standards
(2) information presented in the commentary sections
(3) sources referenced by the standards
(4) additional pertinent technical material known to the reviewers

The project did not cover seismic isolation. The NRC staff has a separate ongoing project investigating the applicability of seismic isolation to commercial nuclear power plants.

The implementation of ASCE 4-16 and ASCE 43-18 will achieve two target performance goals:

(1) less than an approximately 1-percent probability of unacceptable performance, conditioned on the design-basis earthquake (DBE) ground motion

(2) less than an approximately 10-percent probability of unacceptable performance, conditioned on 150 percent of the design-basis earthquake ground motion

ASCE 4-16 and ASCE 43-18 state that these performance goals are achieved when the seismic demand is determined at an approximately 80-percent no exceedance probability for the specified input response spectrum, and the seismic capacity is based on a 98-percent exceedance level. This report reviews this statement for acceptability.

During the review of each section of ASCE 4-16 and ASCE 43-18, the reviewers identified and documented any concerns or issues, then developed a recommendation for resolution for staff consideration. The resolution could be taking an exception, supplementing the criteria, or providing clarifying information. The report provides the technical basis for any exceptions taken.

It is noted that peer review is required by both ASCE 4-16 and ASCE 43-18. Modern structural engineering practice considers peer review an essential element in a performance-based design approach. An independent peer review provides confidence in the technical adequacy of the procedures and results of analysis, testing, and calculation used to demonstrate compliance with standards. Accordingly, this report identifies specific provisions in both ASCE 4-16 and ASCE 43-18 where peer review is especially important.

The technical positions documented in this report represent the consensus of the report authors. Each author brought his own special expertise to this project and was assigned review responsibility for his area of expertise. All authors reviewed all of the proposed technical positions. Each author had the opportunity to review and comment on positions and conclusions advanced by the other authors, and to discuss his views in an open forum.

1.3 General Conclusions

The performance-based standards ASCE 4-16 and ASCE 43-18 provide a valuable framework for implementing the RIPB approach for seismic analysis and design of SSCs in commercial nuclear power plants. The standards are judged to be an acceptable technical basis for the NRC staff's planned RIPB seismic design alternative, provided the concerns and issues identified herein are appropriately addressed.

Apart from the specific issues identified, this report evaluates the possibility of achieving the overall ASCE 4-16 target for demand, namely that "the seismic demand is determined at about 80% probability of not being exceeded for the specified input response spectrum," in light of the relatively large number of modeling and analysis approximations that Chapters 3 and 4 of ASCE 4-16 permit. There is concern that the cumulative effect of several cascading underpredictions may compromise the achievement of the 80-percent nonexceedance probability. In several cases, ASCE 4-16 specifies current state-of-the-art guidance that improves modeling and analysis accuracy, alongside more traditional approximations with 10-percent acceptance bands. To ensure the achievement of the 80-percent nonexceedance probability on demand, this report recommends the use of the latest modeling and analysis guidance provided in the current SRP Section 3.7.2, "Seismic System Analysis," some of which has already been incorporated into ASCE 4-16.

ASCE 4-16 cites a limited number of anecdotal studies in support of the conclusion that it achieves the target nonexceedance probability (see Commentary Section C1.1). The commentary also indicates that additional studies will be performed to further support this conclusion. This report recommends that probabilistic seismic analysis studies be performed that focus specifically on the effects of the 10-percent acceptance bands on achieving the target nonexceedance probability. In the interim, BNL recommends performing an evaluation, including peer review, during the design phase to verify this assumption.

Another key issue identified is the standards' assumption that current material codes and specifications deliver capacities at the 98th-percentile exceedance probability (referred to as the 2-percent exclusion limit in many material specifications). This assumption has its roots in the seismic margins studies of the 1990s. ASCE 4-16, Section 1A.2.2.1, "Median Strength Conservatism Ratio," states the following:

> According to a review of median capacities from past seismic probabilistic risk assessment studies versus U.S. code-specified ultimate strengths for several failure modes, the determination is that for ductile failure modes when the

> conservatism of material strengths, code strength equations, and seismic strain-rate effects are considered, at least a 98% probability exists that the actual strength will exceed the code strength. For low ductility failure modes, an additional factor of conservatism of about 1.33 is typically introduced.

The assumption that material standards deliver capacities at the 98th-percentile exceedance probability is nearly three decades old. Since its introduction, both design and construction practices and the governing codes have changed. In addition, the assumption is not supported by any peer- reviewed references.

The appendix to this report evaluates some common structural components against the assumption of 98th-percentile exceedance probability. Based on this evaluation and the above discussion, the assumption appears to be reasonable, for the most part; therefore, ASCE 43-18, Section 1.1, “Seismic Design Criteria,” is judged to be acceptable. However, it is strongly recommended that a study be performed to provide additional data on the assumption of 98th-percentile exceedance probability. It is also recommended that a peer review be performed during the design phase to confirm satisfaction of this assumption.

In addition to the high-level conclusions discussed above, Chapter 3 of this report summarizes the reviewers’ technical conclusions and recommendations for the acceptable implementation of ASCE 4-16 and ASCE 43-18. Chapter 2 and the appendix document the detailed assessments leading to these conclusions and recommendations.

2 ASSESSMENT OF ASCE 4-16 AND ASCE 43-18

The updated standards ASCE 4-16 and ASCE 43-18 show significant improvements from the prior versions of these two standards. This section summarizes the results of the review; the appendix contains further details and clarifications on some of the items below.

2.1 Assessment of ASCE 4-16

2.1.1 Three-Dimensional vs. Planar Models

ASCE 4-16, Section 3.1.1, "Methods for Horizontal and Vertical Motions"

ASCE 4-16, Section 3.1.1, "Methods for Horizontal and Vertical Motions," indicates that separate analytical planar models for excitations in each direction may be used as long as significant coupling does not exist between structural responses (i.e., seismic input in one direction does not cause significant response in orthogonal directions). Otherwise, the seismic response analysis should use a single combined three-dimensional analytical model.

Planar models for individual-direction excitations should not be used unless there is quantitative evidence that any error introduced by their use is conservative, relative to results from a three-dimensional analytical model.

2.1.2 Mesh Refinement

ASCE 4-16, Section 3.1.2, "Multistep and One-Step Methods of Seismic Response"

ASCE 4-16, Section 3.1.2.1, "Models for Multistep Analysis," states, "The first-step model shall be sufficiently detailed so that the responses calculated for input to subsequent steps or for evaluation of the first-step model would not change by more than 10% upon further refinement."

ASCE 4-16, Section 3.1.3, "Discretization Considerations"

ASCE 4-16, Section 3.1.3.2, "Selection of Mesh Size," states, "Conformance to Section 3.1.3.2 may be demonstrated by performing convergence studies with a small dynamically similar structure and developing specific modeling guidelines for a specific element and computer code. The criteria in Section 3.1.4 shall be used to judge convergence."

ASCE 4-16, Section 3.1.4, "Alternate Methods"

ASCE 4-16, Section 3.1.4, "Alternate Methods," states, "Alternate methods may be used to satisfy the requirements of Chapter 3 provided that it can be demonstrated that the response parameter(s) of interest are not underestimated by more than 10%."

The guidance provided above is not considered appropriate for developing an adequate mesh. The mesh should be sufficiently refined to accurately calculate all responses needed for input to subsequent steps or for evaluation of the first-step model. Mesh convergence can be demonstrated by plotting key response quantities against mesh size, which should asymptotically approach the converged response with decreasing mesh size. If convergence is from above (i.e., conservative), the 10-percent guideline is acceptable. If convergence is from below (i.e., unconservative), a more stringent convergence criterion (e.g., 5 percent) is recommended.

The mesh size for the second-step model should satisfy the same criterion for the key response quantities needed for structural design.

If a single model is used for both the overall dynamic analysis and the calculation of the key response quantities needed for structural design (one-step analysis), then the mesh convergence study should include the key response quantities needed for structural design as well as the key dynamic analysis response quantities.

The detailed guidance on mesh refinement in SRP Section 3.7.2 may be used to supplement the guidance in ASCE 4-16 and meet the acceptance criteria recommended above.

2.1.3 Damping

Various sections of ASCE 4-16 and ASCE 43-18 specify structural and equipment damping values. These sections were reviewed and compared to current SRP guidance (RG 1.61, Revision 1, “Damping Values for Seismic Design of Nuclear Power Plants,” issued March 2007). The specified Response Level 1 and Response Level 2 damping values are consistent with the operating-basis earthquake (OBE) and safe-shutdown earthquake (SSE) damping values in RG 1.61, Revision 1, with one notable exception.

For piping damping, ASCE has adopted the recommendation of the American Society of Mechanical Engineers (ASME), namely 5-percent damping for both Response Level 1 and Response Level 2. The current staff position in RG 1.61, Revision 1, for piping damping is 3 percent for OBE and 4 percent for SSE. The latter values have been confirmed in a study performed by BNL for the staff, following release of RG 1.61, Revision 1. (See PVP2010-25465, “Assessing Equivalent Viscous Damping Using Piping System Test Results,” issued 2010.) Consequently, BNL recommends that an exception be taken to the Response Level 1 and 2 piping damping values listed in ASCE 4-16 and ASCE 43-18, replacing them, respectively, with the OBE and SSE piping damping values in RG 1.61, Revision 1.

ASCE 4-16 and ASCE 43-18 indicate the use of Response Level 1 (comparable to OBE) damping values when explicit nonlinear inelastic analysis is performed. This is consistent with the current staff position in RG 1.61, Revision 1.

ASCE 4-16 and ASCE 43-18 indicate the use of Response Level 1 damping in building analyses intended to generate in-structure response spectra (ISRS) for subsequent analysis of attached SSCs. This is also consistent with the current staff position in RG 1.61, Revision 1.

The review also considered the correct use of Response Level 3 damping in analyses for Limit States C, B, and A (limited permanent distortion to very large permanent distortion). BNL finds the Response Level 3 damping values specified in ASCE 4-16 and ASCE 43-18 to be acceptable for use with linear elastic analysis.

However, the significant technical issue is whether Response Level 3 damping values are appropriate when inelastic energy absorption factors are also applied to linear elastic results. This represents a double reduction of demand (i.e., energy dissipation by use of the inelastic energy absorption factor and energy dissipation by use of the Response Level 3 damping).

BNL’s review of prior research in this area (e.g., NUREG/CR-3805, “Engineering Characterization of Ground Motion—Task 1: Effects of Characteristics of Free-Field Motion on Structural Response,” issued May 1984) does not appear to support the guidance in the

Commentary to ASCE 43-18, namely that Response Level 3 damping can be used in conjunction with inelastic energy absorption factors. The technical basis cited in the Commentary is extremely limited in scope. BNL considers that a more comprehensive technical basis is needed to support the use of Response Level 3 damping in conjunction with inelastic energy absorption factors.

Until such a technical basis is developed, BNL recommends using Response Level 2 damping values in conjunction with inelastic energy absorption factors.

ASCE 4-16, Section 3.5 and Commentary Section C3.5, "Modeling of Damping," contain several detailed numerical procedures for modeling damping in finite element analysis. These procedures are generally acceptable based on the cited references. Implementations of these methods should be peer-reviewed for technical adequacy.

Appendix A1, "Damping," gives further details on the evaluation of damping.

2.1.4 Modeling of Stiffness

<u>ASCE 4-16, Section 3.3.1, "General Requirements"</u>

ASCE 4-16, Section 3.3.1, "General Requirements," indicates the following three points:

(1) The mathematical model shall represent, at a minimum, the structural elements that form the primary load-resisting system of the structure.

(2) The stiffness of secondary structural elements, which are not part of the primary load path, may be omitted from the mathematical model, provided that secondary structural elements do not unconservative affect the response parameters of interest by more than 10 percent.

(3) Structural elements that are rigid compared with the stiffness of other structural elements in the load path may be considered as rigid bodies in the analysis, provided that the inclusion of rigid elements does not affect the response parameters of interest by more than 10 percent compared with a realistic stiffness model.

BNL considers it unreasonable and inappropriate to try to separate the effect of omitting some structural elements from the effect of viewing other structural elements as rigid. Once all model simplifications have been implemented, key responses of the simplified model can be compared to the corresponding responses of a "realistic stiffness model." Typically, this will involve comparison of frequencies and mode shapes to establish dynamic equivalency for the frequency range of interest. If the simplified model has missing modes, then it will need refinement until all key responses at all key locations can be quantitatively shown to be within 10 percent on the conservative side or about 5 percent on the unconservative side. For this study, the total mass and the mass distribution must be essentially the same in both the simplified model and the realistic stiffness model.

2.1.5 Modeling of Mass

<u>ASCE 4-16, Section 3.4, "Modeling of Mass"</u>

ASCE 4-16, Section 3.4.1, "Discretization of Mass," paragraph (b), states, "Mass for some degrees of freedom, such as rotational degrees of freedom, may be neglected, provided that their exclusion does not unconservatively affect the response parameters of interest by more than 10% and the torsional response is not affected." BNL considers this reasonable, because the current norm is to develop three-dimensional models with structure mass distributed throughout the model. In such models, mass is typically not assigned to rotational degrees of freedom, and this has minimal impact on the solution. For stick models, however, rotational degrees of freedom may be significant and should not be neglected.

ASCE 4-16, Section 3.4.1, "Discretization of Mass," paragraph (b)(2), states the following:

> The number of dynamic degrees of freedom, and hence the number of lumped masses, shall be selected so that all significant vibration modes (at least 90% effective mass participation) of the structure can be evaluated. For a structure with distributed mass, the number of degrees of freedom in a given direction shall be equal to at least twice the number of significant modes in that direction.

BNL notes that the current SRP Section 3.7.2 provides sensible guidance that allows the analyst to assess when a sufficiently accurate model of mass has been developed for dynamic analysis. BNL recommends this guidance over the traditional 90-percent effective mass participation rule of thumb. Appendix A.1, under the heading "Modeling of Mass," presents further details of the evaluation.

ASCE 4-16, Section 3.4.2, "Determination of Mass," describes the mass to be included in the seismic analysis model. Paragraph (a) indicates that the mass should include all mass expected to be present at the time of the earthquake and should not include added conservatism. This mass will include, for example, the effects of dead load, stationary equipment, piping, and the appropriate part of the live load and snow load. Paragraph (b) indicates that live load not less than 25 percent of the specified design live load for design loads 200 pounds per square foot (psf) and smaller, and not less than 50 psf for design live loads greater than 200 psf, should be included. Paragraph (c) states, "Design flat roof snow loads of 30 psf or less need not be included. Where flat roof snow loads exceed 30 psf, 25% of the uniform design snow load shall be included."

The above guidance differs in several respects from the current staff guidance given in SRP Section 3.7.2, which states the following:

> In addition to the structural mass, mass equivalent to a floor load of 50 pounds per square foot should be included, to represent miscellaneous dead weights such as minor equipment, piping, and raceways. Also, mass equivalent to 25 percent of the floor design live load and 75 percent of the roof design snow load, as applicable, should be included. The mass of major equipment should be distributed over a representative floor area or included as concentrated lumped masses at the equipment locations.

SRP Section 3.7.2 specifies 50 psf for miscellaneous dead weight, whereas ASCE 4-16 specifies actual miscellaneous dead weight. ASCE 4-16 is considered acceptable for an RIPB design methodology.

SRP Section 3.7.2 specifies 25 percent of design live load, whereas ASCE 4-16 specifies 25 percent of design live load, with a maximum of 50 psf for live loads greater than 200 psf. ASCE 4-16 is considered acceptable for an RIPB design methodology.

SRP Section 3.7.2 specifies 75 percent of design snow load, whereas ASCE 4-16 specifies that design flat roof snow loads of 30 psf or less need not be included, and that, where flat roof snow loads exceed 30 psf, 25 percent of the uniform design snow load must be included. NUREG/CR-6926 (which reviewed the adequacy of ASCE 43-05) accepted the use of 25 percent of the design snow load, based on review of other standards (e.g., ASCE 7-05, "Minimum Design Loads for Buildings and Other Structures," or the International Building Code (IBC) 2003); therefore, it is acceptable.

2.1.6 Hydrodynamic Mass Effects

ASCE 4-16, Section 3.6.3, "Hydrodynamic Mass Effects on Building Model"

ASCE 4-16, Section 3.6.3, "Hydrodynamic Mass Effects on Building Model," paragraph (c), states the following:

> When the basin walls do not respond as a rigid body, or when local stresses are of interest, the masses and associated sloshing mode horizontal springs shall be distributed over part of the basin wall height as shown in Fig. 3-1. The impulsive mass may be uniformly distributed over a height equal to twice the distance from the bottom of the basin to the center of mass (as determined for the case of a single impulsive mass). Similarly, the horizontal springs for the sloshing effect shall be distributed over a height from the top of the water surface to the center of mass (as determined for the case of a single sloshing mass). The sloshing mass shall be attached, through a rigid link, to the distributed springs.

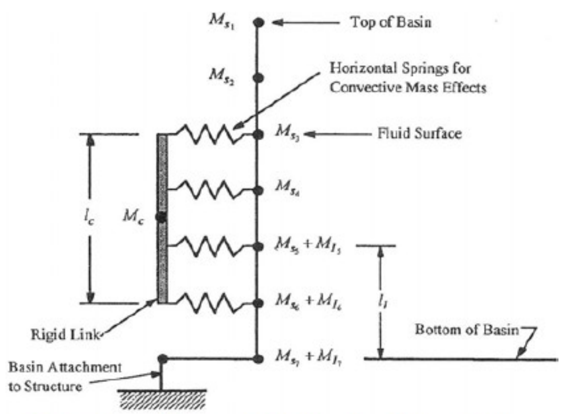

FIGURE 3-1. Distribution of Fluid Mass for Horizontal Seismic Response Analysis of Basins with Flexible Walls and/or Local Stress Problems

ASCE 4-16 does not provide the technical basis for Figure 3-1. Commentary Section C3.6.3 provides no additional information or references for Figure 3-1; it simply states, "If the walls are not relatively rigid and/or the local stress effects are important, then the impulsive mass and convective spring constants should be distributed as specified in this standard." Implementations of Figure 3-1 for analysis of hydrodynamic mass effects should be peer-reviewed for technical adequacy.

2.1.7 Dynamic Coupling Criteria

ASCE 4-16, Section 3.7, “Dynamic Coupling Criteria”

ASCE 4-16, Section 3.7.1, “General Requirements,” provides the general criteria for decoupling when a primary structure and secondary system exist. Implementing Section 3.7.1 may involve a substantial calculational effort to satisfy decoupling criteria for multi-degree-of-freedom (MDOF) models typical of those developed for seismic/structural analysis of nuclear power plant SSCs.

In many cases, supported components may be amenable to modeling with a single degree of freedom (SDOF) but are attached to supporting MDOF structure or system models. Therefore, the decoupling criteria in ASCE 4-16, Section 3.7.2(a), Figure 3-2, applicable to an SDOF supporting element and SDOF supported element, must be implemented with caution. Commentary Section C3.7.2 states, “Therefore, the application of the criteria in Fig. 3-2 to practical situations such as multi-degrees-of-freedom secondary systems multiply connected to several multi-degrees-of-freedom primary systems requires judgment, caution, and additional considerations.” Consequently, implementations of the criteria in Figure 3-2 for MDOF supporting elements and either MDOF or SDOF supported elements should be peer-reviewed for technical adequacy.

Decoupling is intended to simplify the analysis process without compromising the validity of the results; it can be implemented if it is demonstrated to produce results close to, or conservative relative to, the coupled response. BNL considers a difference of up to 10 percent between coupled response and uncoupled response to be acceptable if it is on the conservative side. Excessive conservatism may indicate improper modeling. BNL further recommends the use of a coupled model if the uncoupled response is unconservative by more than 5 percent relative to the coupled response. This ensures that decoupling will not contribute significantly to underprediction of the seismic response.

It is noted that the straightforward decoupling criteria in SRP Section 3.7.2 have been in use for many years and are relatively easy to implement. They should be considered as an alternative to those in ASCE 4-16, Section 3.7.

Appendix A.1, under the heading “Dynamic Coupling Criteria,” presents further details on the evaluation of dynamic coupling.

2.1.8 Additional Requirements for Modeling Specific Structures

ASCE 4-16, Section 3.8.1.3, “Requirements for Lumped-Mass Stick Models”

ASCE 4-16, Section 3.8.1.3, “Requirements for Lumped-Mass Stick Models,” paragraph (a)(2), states the following:

> The vertical response analysis determines seismic motions at different elevations of the structure and not at various points on a vertically nonrigid floor. However, if the vertical flexibility of the floors is included in the model, then response values may be determined at various points on the floors.

ASCE 4-16 does not discuss the horizontal response of nonrigid walls. The Economic Simplified Boiling Water Reactor (ESBWR) is the most recent design to use a lumped-mass stick model for seismic analysis of the nuclear island. The ESBWR Design Control Document (2014) states the following:

> The vertical floor frequencies are obtained at major floor locations by independent modal analysis of the respective floor finite element model. These frequencies are included in the stick model by a series of vertical single degree-of-freedom oscillators at the corresponding floor elevations.
>
> The out-of-plane vibration frequencies of walls are evaluated by using finite element models in the same manner as the slab frequencies. These frequencies are included in the stick model by a series of horizontal single degree-of-freedom oscillators at the corresponding wall elevations to obtain design loads of these walls and design FRS for the components attached to these walls.

SDOF oscillators tuned to the fundamental out-of-plane frequencies of the floors and walls can be incorporated directly into a stick model. BNL recommends that this be the modern standard for lumped-mass stick models, to ensure that their analysis can identify potential out-of-plane amplification of both wall and floor responses. Depending on the frequency range of interest in the analysis, it may be necessary to add more SDOF oscillators representing higher modes of the walls and floors.

2.1.9 Requirements for Adjacent Structures

<u>ASCE 4-16, Section 3.8.5, "Requirements for Adjacent Structures"</u>

ASCE 4-16, Section 3.8.5, "Requirements for Adjacent Structures," states the following:

> The relative deformations between structures shall be considered in the analysis of elements connected to or supported by multiple structures and in specifying clearance between structures. Adjacent structure displacements may be combined by the square-root-of-the-sum-of-squares (SRSS) method to obtain relative deformations.

For elements connected to or supported by multiple structures, the commentary does not offer any information or identify references providing the technical basis for using the SRSS method. By comparison, the SRP guidance indicates that the absolute sum (ABSUM) in the worst possible configuration should be considered in evaluation of seismic anchor movements. This applies to "elements connected to or supported by multiple structures." Therefore, to prevent underestimation of the relative deformations between structures, BNL recommends that an exception be taken and ABSUM be used.

For clearance between structures, SRP Section 3.7.2.8, "Interaction of Non-Seismic Category I Structures with Seismic Category I SSCs," under Criterion C, states the following:

> The maximum permissible displacement of the non-seismic Category I structure in any direction is determined by subtracting the maximum calculated displacement of each adjacent seismic Category I SSC in the direction of the non-seismic Category I structure from the minimum as-designed gap, considering construction tolerances.

Based on SRP Section 3.7.2.8, the required clearance for design is the ABSUM of the adjacent structures' independent displacements, plus a construction tolerance. While SRP Section 3.7.2.8 specifically addresses seismic Category II/I interactions, the guidance for determining the required clearance applies equally to adjacent Category I structures. For seismic design, to ensure adequate seismic gaps between adjacent structures, BNL recommends that the ASBUM be used.

SRP Section 3.7.2.8 allows nonseismic Category I (Category II) structures to undergo inelastic nonlinear response, similar to Limit State C and possibly Limit State B from ASCE 43-18, provided that (1) there is sufficient clearance to accommodate the increased displacement of the Category II structure, and (2) structural stability of the Category II structure is demonstrated at the limit state. This provision requires that a nonlinear analysis be conducted to predict the increased displacement.

ASCE 43-18, Section 7.3, "Seismic Separation," give the criteria for acceptable clearances between adjacent SSCs to preclude impact. The acceptance criteria are independent of ASCE 4-16, Section 3.8.5. Section 2.2.8 of this report reviews these criteria separately.

2.1.10 Analysis of Structures

ASCE 4-16, Section 4.2, "Linear Response-History Analysis"

For mode-superposition time history analysis, ASCE 4-16, Section 4.2.1(d)(3), states, "Including all the modes in the analysis with frequencies less than the zero-period acceleration (ZPA) frequency shall be sufficient, provided that the residual rigid response due to the missing mass is calculated and combined algebraically with the modal responses."

However, ASCE 4-16, Section 4.2.1(d)(4), states, "As an alternative to the previous item 3, the number of modes included in the analysis shall be sufficient to ensure that inclusion of the remaining modes does not result in more than a 10% increase in any response measure of interest."

The alternative offered in paragraph 4.2.1(d)(4) was eliminated from SRP Section 3.7.2 in 2007 and replaced with calculation of the missing mass contribution to total response, by reference to RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis." Paragraph 4.2.1(d)(3) above is consistent with current SRP guidance and should be followed. NUREG/CR-6645, "Reevaluation of Regulatory Guidance on Modal Response Combination Methods for Seismic Response Spectrum Analysis," issued December 1999, presents the technical basis for the elimination of the 10-percent criterion.

ASCE 4-16, Section 4.3, "Linear Response-Spectrum Analysis"

For response spectrum analysis, ASCE 4-16, Section 4.3.1(b), states, "A sufficient number of modes shall be included in the analysis to ensure that the inclusion of the remaining modes does not result in more than a 10% increase in the responses of interest."

However, ASCE 4-16, Section 4.3.1(c), states the following:

> Alternatively, for modal combination purposes, high frequency modes may be combined into a single residual mode. The residual rigid response shall be considered as an additional mode with a frequency equal to that corresponding to the ZPA or the highest target frequency.

Current practice for seismic response spectrum analysis for nuclear power plants, in accordance with SRP Section 3.7.2, is to follow the approach in Section 4.3.1(c). The approach in Section 4.3.1(b) is no longer considered acceptable. The assessment above for Section 4.2 includes more information.

2.1.11 Combination of Multiple Response Parameters

ASCE 4-16, Section 4.3.4, "Combination of Multiple Response Parameters"

ASCE 4-16, Section 4.3.4, "Combination of Multiple Response Parameters," paragraph (a), states the following:

> When more than one response parameter exists, such as column axial force and moment, in the design calculation, the combined value of each response shall be calculated by SRSS or 100-40-40, including the effects of rigid body response. In the subsequent design calculations, all possible combinations of these values shall be considered. For *M* response parameters of interest, 2^M sets of response combinations exist to be considered.

ASCE 4-16, Commentary Section C4.3.3, "Combination of Spatial Components," provides the following example illustrating the 100-40-40 method for combination of multiple response parameters:

> Consider a shear wall oriented in the north–south direction with seismic responses for four design parameters: P = axial load, V = shear, M_{ip} = in-plane moment, and M_{op} = out-of-plane moment. Assume that responses for these parameters have been obtained from dynamic analysis as shown in Table C4-1. (Note: for simplicity the signs are ignored in this example.)
>
> The first three rows in the table show the calculated responses due to seismic input in each direction. The fourth row is the SRSS combination. The next three rows give the design values using the 100-40-40 method. All three rows obtained from the 100-40-40 method would be used in design as individual seismic load combinations.
>
> As can be seen from the table, the most severe design condition is produced by the SRSS method. This is the case in which each response parameter is dominated by a particular earthquake direction. However, the design resulting from each of the three 100-40-40 method combinations will be less demanding, and the final design will be more realistic as each of these spatial response combinations are likely to occur, but at different points in time.
>
> Using the maximums of each parameter from the three factored combinations yields results similar to SRSS but negates any benefit of 100-40-40 and is inconsistent with the goal of determining seismic design parameters that are most likely to occur simultaneously.

Table C4-1. Application of the SRSS and 100-40-40 Methods

Seismic Load	P, kip	V, kip	M_{ip}, kip-ft	M_{op}, kip-ft
N-S earthquake	0	500	10,000	100
E-W earthquake	0	30	500	500
Vert. earthquake	400	0	0	0
SRSS	400	501	10,012	510
Factored 1: 100 + 40 + 40	160	**512**	**10,200**	300
Factored 2: 40 + 100 + 40	160	230	4,500	**540**
Factored 3: 40 + 40 + 100	**400**	212	4,200	240

In the past, the guidance in ASCE 4-16, Section 4.3.4(a), has been subject to differing interpretations by BNL and by industry. To clarify this issue, BNL has performed a simple quantitative study that validates industry's implementation of 100-40-40 combination for multiple response parameters, as presented in Commentary Section C4.3.3.

Appendix A.1, under the heading "Combination of Multiple Response Parameters," presents details of BNL's quantitative study. Based on that evaluation, BNL recommends the use of the 100-40-40 combination method for multiple response parameters. SRSS remains acceptable because it is conservative compared to the 100-40-40 method.

2.1.12 Combination of Seismic Inertial Response with Seismic Anchor Movements

ASCE 4-16, Section 4.3.5, "Systems with Multiple Supports"

ASCE 4-16, Section 4.3.5, "Systems with Multiple Supports," paragraph (e), states, "Responses using either the envelope-spectrum method or the multiple-spectrum method described previously shall be combined with the responses due to the relative support displacements using the SRSS rule."

ASCE 4-16, Commentary Section C4.3.5, under the heading "Combination of Inertial and Seismic Anchor Displacement Effects," states the following:

> Displacement-induced (secondary) responses and inertial induced (primary) responses are not phase uncorrelated. In fact, they often have a negative phase correlation. Therefore, the SRSS combination of primary and secondary responses cannot be justified on theoretical grounds. However, peak primary and peak secondary responses would be highly unlikely to occur concurrently. Therefore, an absolute sum (ABS) combination would generally be excessively conservative. An SRSS combination is preferable even though unjustified on theoretical grounds. Ibrahim (1979) demonstrates that SRSS-combined primary and secondary responses have a 96.4% nonexceedance probability. A BNL study also recommends an SRSS combination (Subudhi et al. 1984).

For the uniform support motion (USM) method, referred to as the envelope-spectrum method in ASCE 4-16, the guidance in SRP Section 3.7.3 is that the responses due to the inertia effect and relative displacements should be combined by the ABSUM method.

For the independent support motion (ISM) method, referred to as the multiple-spectrum method in ASCE 4-16, SRP Section 3.7.3 references Section 2 of NUREG-1061, Volume 4, "Report of the U.S. Nuclear Regulatory Commission Piping Review Committee," issued December 1984, which states that, for the total response, dynamic and pseudostatic responses should be combined by the SRSS rule.

Current SRP guidance is consistent with ASCE 4-16 for ISM, but not for USM. However, in light of the work conducted at BNL in the early 1980s, which recommended SRSS combination, and the demonstration by Ibrahim (1979) that SRSS-combined primary and secondary responses have a 96-percent nonexceedance probability, use of SRSS combination for USM is judged to be consistent with RIPB principles and is acceptable.

2.1.13 Nonlinear Static Analysis

ASCE 4-16, Section 4.9, "Nonlinear Static Analysis"

Review of ASCE 4-16, Section 4.9, "Nonlinear Static Analysis," and Commentary Section C4.9, determined that the methodology presented relies on the approach in ASCE 41-13, "Seismic Evaluation and Retrofit of Existing Buildings," which is used specifically for existing commercial and residential buildings. Consequently, BNL concludes that ASCE 4-16, Section 4.9, is not applicable to design of commercial nuclear power plant structures.

2.1.14 In-Structure Response Spectra

ASCE 4-16, Section 6.2.3, "Treatment of Uncertainties in Generating In-Structure Response Spectra"

Response spectrum peak clipping is discussed in ASCE 4-16, Section 6.2.3, "Treatment of Uncertainties in Generating In-Structure Response Spectra," paragraph (b), which states the following:

> In conjunction with response-spectrum peak broadening, a 15% reduction in the narrow frequency peak amplitude is permissible if the subsystem damping is less than 10%. This 15% reduction is only to be applied to narrow frequency peaks of the unbroadened response spectrum with a bandwidth-to-central-frequency ratio, B, less than 0.30:
>
> $$B = \Delta f_{0:8} / f_c < 0.30 \qquad (6\text{-}1)$$
>
> where $\Delta f_{0:8}$ = total frequency range over spectral amplitudes that exceed 80% of the peak spectral amplitude; and f_c = central frequency for the frequencies that exceed 80% of the peak amplitude. Further reductions are permissible if the probability of nonexceedance for the resulting spectrum can be shown to be at least 80%.

Section 6.2.3, paragraph (c), which discusses peak shifting as an alternate to peak broadening, indicates that reduction in peak spectral amplitude is not permissible in conjunction with this peak shifting method.

Section 6.3.2, “Equivalent Broadening and Lowering of In-Structure Time Series,” indicates that lowering of time series motions to be used in the seismic analysis of subsystems consistent with Section 6.2.3(b) may be implemented.

The ASCE 4-16 Commentary Section cites ASCE Manuals and Reports on Engineering Practice, No. 58, “Structural Analysis and Design of Nuclear Plant Facilities,” issued 1980, as the technical basis for peak reduction (the Commentary does not use the word “clipping”). The Commentary notes that ASCE (1980) compares equal-probability-of-exceedance ISRS with deterministic ISRS. The equal-probability-of-exceedance spectra have much broader spectrum peaks and much lower peak amplitudes than do the deterministic spectra. Deterministic spectrum peaks, broadened to account for the effects of uncertainties, introduce considerable conservatism within this broadened peak region unless a corresponding reduction in peak amplitudes is allowed. Reduction (clipping) of spectrum peak amplitudes such that the probability of nonexceedance for the resulting spectra is not less than 80 percent is proposed as a rational seismic design basis for subsystem design. In lieu of a probabilistic evaluation, a 15-percent reduction in peak amplitude of deterministic spectra is proposed as reasonable and conservative. ASCE (1980), Section 5.8.3, “Studies on Uncertainties in Floor Spectra,” provides the technical basis for reducing spectral peaks in deterministically derived ISRS.

Based on the limited data presented, a 15-percent reduction in peak amplitude for narrow, sharp peaks in the ISRS appears to be reasonable and by itself would not jeopardize achievement of the target of 80-percent nonexceedance probability on demand.

However, when each of several other approximations currently permitted (although not necessarily promoted) in the ASCE 4-16 analysis process is allowed to underpredict the response by up to 10 percent, the deterministically derived ISRS with sharp, narrow peaks may have already compromised the achievement of 80-percent nonexceedance probability. Therefore, the acceptance of peak clipping as defined in ASCE 4-16, Sections 6.2.3 and 6.3.2, is conditional, pending detailed demonstration that the entire analysis process, considering all sources of potential unconservatism, meets the 80-percent nonexceedance probability target.

2.1.15 Dynamic Soil Pressures on Walls

ASCE 4-16, Section 8.2, “Embedded Building Walls”

This section describes acceptable procedures for determining the seismic loads due to soil pressures on embedded walls. It describes the use of the finite element analysis method (ASCE 4-16, Section 8.2.1), a simplified method based on the solution of Wood (1973) (ASCE 4-16, Section 8.2.2), and an alternate method (ASCE 4-16, Section 8.2.3). As described in Commentary Section C8.2.2, in some cases, Wood’s method may not represent an upper bound and then the dynamic finite element or alternate method should be used. Thus, depending on the foundation configuration and other factors, sometimes Wood’s method governs and sometimes the finite element method governs. Also, Wood’s method may govern over certain regions while the finite element method governs over the other regions. The report “Technical Rationale for Proposed Enhancements to Seismic and Structural Review Guidance,” Revision 1, dated February 19, 2013 (Xu et al., 2013), describes several uncertainties and variabilities that may arise in each method. Finally, for structural stability evaluations, the design of the embedded walls needs to account for the potential development of large lateral soil pressures (approaching the passive pressure case). The magnitude of the lateral soil pressure developed depends on the maximum relative displacement occurring between the structure and surrounding soil during a seismic event.

Based on the above discussion, BNL recommends the use of one of the following bounding seismic soil pressures on embedded walls, unless otherwise justified:

(1) pressure calculated in accordance with ASCE 4-16, Section 8.2.1, “Dynamic Finite Element Analysis”

(2) pressure calculated in accordance with ASCE 4-16, Section 8.2.2, “Simplified Method”

(3) pressure equal to the fraction of the passive earth pressure that is used for the structure stability evaluation

2.1.16 Distribution Systems

<u>ASCE 4-16, Chapter 10, “Distribution Systems”</u>

This chapter provides the seismic analysis criteria for mechanical and electrical distribution systems consisting of piping, tubing, ductwork, and raceways and their supports. The criteria in ASCE 4-16, Chapter 10, “Distribution Systems,” have extremely limited utility:

(1) Analytical methods and design criteria for piping (Section 10.2) and piping supports (Section 10.3) are well documented elsewhere; they primarily follow the rules of ASME. In several cases the criteria repeat those that are already in ASCE 43-18 (e.g., inelastic energy absorption factors (F_{μ}), which appear in Section 8.2.2.2 and Table 8-1 of ASCE 43). It is not recommended that the same information appear in two different standards, as this may lead to inconsistencies.

    In addition, some of the information is too prescriptive (e.g., specifying piping deadweight support spacings), which contravenes the RIPB philosophy of allowing designers flexibility in achieving performance goals. Also, some of the information relates to design (e.g., allowable stress in the piping); this is not appropriate for ASCE 4, which should focus on the derivation of the seismic demand.

    The information provided in ASCE 4-16 is therefore judged not to be useful. This also applies to Section 10A, “Attachment: Simplified Design of Cold Piping by the Load Coefficient Method and Design by Rule (Nonmandatory).”

(2) The information on tubing (Section 10.4) and ductwork (Section 10.5) is either generic or references recognized standards. ASCE 4-16 provides no useful additional information.

(3) The detailed guidance for raceways (Section 10.6) is based on raceway design in older nuclear power plants, which made extensive use of trapeze rod hangers for deadweight support of conduits and cable trays. These supports were the subject of a major seismic qualification program under the Systematic Evaluation Program, which included extensive dynamic testing of prototypical configurations. While the guidance for analysis of trapeze rod hanger systems is reasonable, modern support design for safety-significant raceways (conduits and cable trays) in commercial nuclear power

plants is very unlikely to use trapeze rod hanger supports.

(4) The discussion of damping (Section 10.7) adds no new information but does contain an erroneous statement: “The values used for ductwork, conduit, and tubing are the same as for piping.” This is inconsistent with ASCE 4-16, Table 6-2, for conduit. In addition, Table 6-2 does not address ductwork.

BNL recommends that an exception be taken to ASCE 4-16, Chapter 10, on the basis that it provides no useful additional information for deriving the seismic demand on distribution systems in new commercial nuclear power plants. Instead, the codes and standards applicable to these types of distribution systems should be used.

2.1.17 Dynamic Sliding and Uplift Analysis

ASCE 4-16, Section 11.1, “General”

This section indicates that anchoring of components to prevent sliding and uplift is “preferable” and that structures are usually built without any anchorage. For nuclear power plants, BNL recommends that the NRC require components (excluding structures on grade) to be anchored to prevent sliding and uplift (rather than identifying anchoring as a preference), unless the components are expected to be in place for a short period (e.g., in the case of concrete block walls for radiation shielding).

ASCE 4-16, Section 11.2, “Analysis Methods,” and Section 11.3, “Acceptable Approximate Methods for Analysis of Sliding and Overturning of an Unanchored Rigid Body”

ASCE 4-16, Section 11.2, “Analysis Methods,” describes simplified analysis and nonlinear response-history analysis methods to evaluate unanchored components and structures. ASCE 4-16, Section 11.3, “Acceptable Approximate Methods for Analysis of Sliding and Overturning of an Unanchored Rigid Body,” provides detailed formulations. The derivation of these formulations does not require review since unanchored components should not be used except possibly in isolated cases. Furthermore, these methods apply to rigid bodies, which the relevant components often are not. In addition, the use of unanchored components makes it necessary to evaluate additional design/configuration conditions involving potential interactions with adjacent structures and components. It is also noted that these analysis methods address the sliding and uplift stability, not the design, of the structures and components.

Sliding of unanchored components can be highly nonlinear and problem dependent, depending in particular on the motions selected for the seismic analysis. Based on this and the above discussion, BNL recommends that the use of unanchored structures and components (excluding buildings) be avoided. If this is not possible, then the nonlinear analysis method used should be justified and should address the items described above.

ASCE 4-16, Section 11A, “Attachment: Comments on Analysis and Design of Anchorage for Structures and Components (Nonmandatory)”

This section provides a simplified approach to evaluating the potential for overturning of structures and components, based on a formulation developed for a rigid body having a large footprint and low aspect ratio. If the potential for overturning is high, then anchorage is needed. Guidance is also provided on the analysis of structures and components. For the same reasons discussed above for Section 11.2 of ASCE 4-16, and because this attachment is nonmandatory, Section 11A is not evaluated.

2.1.18 Seismically Isolated Structures

<u>ASCE 4-16, Chapter 12, “Seismically Isolated Structures”</u>

This section was considered out of scope, because the NRC has a separate ongoing project investigating the applicability of seismic isolation to commercial nuclear power plants.

2.1.19 Appendix A, “Procedures to Identify Plant-Level Seismic Vulnerabilities and Risk (Nonmandatory)”

Appendix A, “Procedures to Identify Plant-Level Seismic Vulnerabilities and Risk (Nonmandatory),” gives an overview of and background on methodologies for seismic margin assessments and SPRAs, which are used to identify potential seismic vulnerabilities in nuclear facilities. Since ASCE 4-16 does not apply this information directly to the seismic design of SSCs, and because Appendix A is nonmandatory, it was considered out of scope.

2.1.20 Appendix B, “Nonlinear Time-Domain Soil-Structure Interaction (Nonmandatory)”

Nonlinear time-domain soil-structure interaction analysis is an evolving technology, for which limited experience exists in the design of nuclear power plants; furthermore, Appendix B is nonmandatory. Therefore, it was considered out of scope.

2.2 Assessment of ASCE 43-18

2.2.1 Seismic Design Criteria

<u>ASCE 43-18, Section 1.1, “Seismic Design Criteria”</u>

The seismic design criteria in ASCE 43-18 are graded to acceptable risk, with more stringent criteria prescribed for SSCs whose failure has more serious consequences. The seismic design basis is a combination of limit states and SDCs, which are defined in the American Nuclear Society standard ANS 2.26-2004, “Categorization of Nuclear Facility Structures, Systems, and Components for Seismic Design.”

The procedures in this standard are based on two key assumptions:

(1) ASCE 4-16 predicts seismic demands for DBE at the 80th-percentile nonexceedance probability.

(2) Material standards such as American Concrete Institute (ACI) 349-13, “Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary,” and American Institute of Steel Construction (AISC) N690, “Specification for Safety-Related Steel Structures for Nuclear Facilities,” deliver capacities at the 98th-percentile exceedance probability (it is customary to refer to this value as the 2-percent exclusion limit, a term that will be used subsequently in this review).

Used together, these assumptions are intended to ensure (1) a 1-percent (or smaller) annual frequency of unacceptable performance, conditioned on DBE shaking, and (2) a 10-percent annual frequency of unacceptable performance, conditioned on 150 percent of DBE shaking. The use of inelastic energy absorption factors in demonstrating compliance is permitted. The objectives (1) and (2) are comparable if the overall uncertainty, β, defined as the SRSS of the log-standard deviations in demand, β_R, and capacity, β_C, is approximately 0.39. For larger β, (1) controls, while for smaller β, (2) controls.

Section 2.1 of this report discusses the adequacy of the first assumption above in its review of ASCE 4-16. The 80th percentile is approximately one standard deviation above the median level of the ground motion.

The adequacy of the second assumption is more difficult to assess. As noted in Section 1.3 of this report, the assumption has its roots in the seismic margins studies of the 1990s, where it appears in a number of conference papers and reports. ASCE 4-16, Section 1A.2.2.1, "Median Strength Conservatism Ratio," asserts the following:

> According to a review of median capacities from past seismic probabilistic risk assessment studies versus U.S. code-specified ultimate strengths for several failure modes, the determination is that for ductile failure modes when the conservatism of material strengths, code strength equations, and seismic strain-rate effects are considered, at least a 98% probability exists that the actual strength will exceed the code strength. For low ductility failure modes, an additional factor of conservatism of about 1.33 is typically introduced.

The construction provided by these standards "should ensure seismic ruggedness with a high degree of reliability" (Commentary Section C1.1). Although the second assumption appears to be reasonable in most cases, it is nearly three decades old; design and construction practices have changed since its introduction. Furthermore, no peer-reviewed references support it.

The section on ASCE 43-18 in the appendix to this report evaluates some common structural components and limit states, together with the associated probabilities that are used to judge whether the 2-percent (or less) exclusion limit of the second assumption is adequate. The appendix shows that for many reinforced concrete and steel members, the limit state probabilities exceed the 2-percent exclusion limit value. However, these values would be lower for nuclear power plants for three reasons:

(1) Quality assurance in construction of nuclear plant structures is typically higher than in ordinary building construction.

(2) The probabilities do not reflect the effect of ductility, which may significantly increase the capacity for deformation-controlled structural actions such as those due to earthquakes.

(3) Strengths are reported for static rates of load, which are lower than strengths for dynamic loads.

Based on the above and the more detailed analysis in the appendix, the assumption that current material codes and specifications deliver capacities at the 2-percent (or less) exclusion limit appears to be reasonable, for the most part; Section 1.1 of ASCE 43-18 is therefore judged to be acceptable. However, it is strongly recommended that a study be performed to provide further data to support this judgment. It is also recommended that, during the design phase, a peer review be performed for the 2-percent exclusion limit on design strength for the SSCs and the corresponding codes and standards being used. Section 10.1 of ASCE 43-18 includes guidance for performing an independent peer review.

2.2.2 Integration of Other Codes and Standards

ASCE 43-18, Section 1.2, “Integration of Other Codes and Standards with ASCE 43”

ASCE 43-18, Section 1.2, “Integration of Other Codes and Standards with ASCE 43,” and other sections in the standard cite codes and standards without dates or do not give dates consistently. Some chapters (e.g., Chapter 1) give dates in the reference section which follows the Commentary. Others (e.g., Chapter 2) give references only for the Commentary, not for the provision section. Some chapters give no references for codes or standards and fail to identify their titles and dates (e.g., Chapter 4 for ACI 530/530.1-13, “Building Code Requirements and Specification for Masonry Structures and Companion Commentaries”). Most voluntary consensus standards are updated periodically to maintain ANSI compliance, which might affect the validity of provisions in ASCE 43-18. Therefore, BNL recommends that the versions of all codes and standards cited in ASCE 43-18 be taken as those in effect on October 23, 2018, which is the date of the draft of ASCE 43-18 reviewed in this report. If a later date is chosen, then any changes in the later version should be evaluated.

2.2.3 Component Capacities

ASCE 43-18, Section 4.2.2, “Reinforced Concrete,” and Section 4.2.3, “Structural Steel”

Like other sections in ASCE 43-18, Section 4.2.2, “Reinforced Concrete,” indicates that the design and detailing of reinforced concrete should meet the requirements of ACI 349-13. This is acceptable; however, the NRC recently completed its review of ACI 349-13 and published regulatory guidance on it, which should be used as well. Therefore, BNL recommends that the design and detailing of reinforced concrete also conform to RG 1.142, Revision 3, “Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments),” issued May 2020, and RG 1.199, Revision 1, “Anchoring Components and Structural Supports in Concrete,” issued April 2020.

ASCE 43-18, Section 4.2.3, “Structural Steel,” states the following:

> Strength design shall be used for force-capacity of structural steel components wherever possible. If force capacity is established using a standard based on allowable strength design, it is not permitted to be greater than 1.5 times that value calculated using allowable stress design.

This section also indicates that the capacity of a steel component should be determined by the following standards:

- carbon steel components: AISC N690
- stainless steel components: ASCE 8, “Specification for the Design of Cold-Formed Stainless Steel Structural Members”
- cold-formed carbon steel components: American Iron and Steel Institute (AISI) S100, “North American Specification for the Design of Cold-Formed Steel Structural Members”

AISC N690-18 contains two approaches for design of steel structures: Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD). LRFD is a probability-based methodology and is well suited to performance-based design. Therefore, for RIPB applications, BNL recommends the LRFD option.

Section 4.2.2 does not address component capacity for reinforced concrete containments and prestressed concrete containments, and Section 4.2.3 does not address component capacity for steel containments.

For reinforced and prestressed concrete containments, BNL recommends the use of ASME Boiler and Pressure Vessel Code (BPVC), Section III, “Rules for Construction of Nuclear Facility Components,” Division 2, “Code for Concrete Containments,” and the associated RG 1.136, Revision 3, “Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments,” issued March 2007, to determine component capacity.

For steel containments, BNL recommends the use of ASME BPVC Section III, Division 1, Subsection NE, “Class MC Components,” and the associated RG 1.57, Revision 2, “Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components,” issued May 2013, to determine component capacity.

<u>ASCE 43-18, Section 4.2.4, “Steel Composite (SC)”</u>

Like other sections of ASCE 43-18, Section 4.2.4, “Steel Composite (SC),” indicates that the design of SC walls constructed from steel faceplates with concrete infill should be based on AISC N690-12, Supplement 1; however, this standard has been revised and updated. BNL recommends the use of the updated standard AISC N690-18 and the associated NRC draft regulatory guide DG-1304, “Safety-Related Steel and Steel-Plate Composite Structures Other Than Reactor Vessels and Containments,” issued February 2021, for design of SC walls.

<u>ASCE 43-18, Section 4.2.5, “Reinforced Masonry”</u>

ASCE 43-18, Section 4.2.5, “Reinforced Masonry,” permits reinforced masonry walls if they are designed to SDC-5 and in accordance with ACI 530 and ACI 530.1. However, ACI 530 and ACI 530.1 have been superseded by The Masonry Society (TMS) standards TMS 402-16, “Building Code Requirements for Masonry Structures,” and TMS 602-16, “Specification for Masonry Structures,” respectively. The NRC has not previously reviewed any of these standards. Therefore, while the use of reinforced masonry walls is acceptable, their design, whether following ACI 530 and ACI 530.1 or the updated TMS 402-16 and TMS 602-16, should be peer-reviewed.

2.2.4 Inelastic Energy Absorption Factor F_μ and Deformation Acceptance Criteria

<u>ASCE 43-18, Section 5.1.3, “Inelastic Energy Absorption Factor, F_μ”</u>

ASCE 43-18, Table 5-1, summarizes the inelastic energy absorption factors. The Commentary to ASCE 43-18 indicates that the criteria for the inelastic energy absorption factors and deformation limits in ASCE 43-18 were based on ASCE 41-13; FEMA 273, “NEHRP Guidelines for the Seismic Rehabilitation of Buildings,” issued 1997; and WSRC-TR-2001-00037, “Force Reduction Factors for the Structural Design and Evaluation of Facilities Containing Nuclear and Hazardous Materials.” The approach in WSRC-TR-2001-00037 used the data in ASCE 41-13 and relied on certain assumptions and judgments about the design and the range of the fundamental frequency of structures used in nuclear facilities. For example, in the case of reinforced concrete shear walls dominated by flexure, the approach assumed less than 0.10 f'_c axial stress, symmetric reinforcing, and confined boundary elements. Also, it was judged that the fundamental horizontal frequency of these structures would be less than 3 herz (Hz).

While these assumptions and judgments are often reasonable for nuclear power plants, they may not always be valid. Furthermore, they may not be appropriate for future designs of nuclear plants, for which technology-neutral criteria are desirable. Therefore, when using the inelastic energy absorption factors in ASCE 43-18, Section 5.1.3, it should be shown that the relevant assumptions and judgments in WSRC-TR-2001-00037 apply to the particular structure being designed.

ASCE 43-18, Section 5.2.3, "Deformation Acceptance Criteria"

Tables 5-2 and 5-3 of ASCE 43-18 summarize the deformation limits, with Table 5-2 giving the allowable drift ratio limits and Table 5-3 the allowable rotational limits for nonlinear analysis. As with the inelastic energy absorption factors, the criteria for the allowable deformation limits in ASCE 43-18 rely on ASCE 41-13, FEMA 273, and WSRC-TR-2001-00037. Again, these criteria involve some assumptions and judgments. For example, in the case of a special moment-resisting frame, the following are assumed:

- Symmetric reinforcement is used
- Transverse reinforcing with a spacing less than $d/3$ in plastic hinge regions is used
- V_s > 75 percent of V_u for components with a ductility greater than 3
- Beams with a span-to-depth ratio greater than 15 have a shear stress less than $3\sqrt{f_c'}$, while beams with a span-to-depth ratio less than 10 may have a shear stress greater than $6\sqrt{f_c'}$,.
- Columns have an axial load greater than $0.4ff'_{cc}$ but less than 70 percent of the concentric axial load capacity

While these assumptions and judgments are often reasonable for nuclear power plants, they may not always be valid. Furthermore, they may not be appropriate for future designs of nuclear plants, for which technology-neutral criteria are desirable. Therefore, when using the allowable deformation limits in ASCE 43-18, Section 5.2.3, it should be shown that the relevant assumptions and judgments in WSRC-TR-2001-00037 apply to the particular structure being designed.

2.2.5 Anchorage

ASCE 43-18, Section 6.3, "Anchorage"

ASCE 43-18, Section 6.3, "Anchorage," permits the use of adhesive anchors, including in environments with elevated temperature or radiation, provided the anchors are qualified in these environments. It also indicates that design and installation of adhesive anchors in concrete should conform to ACI 318-14, "Building Code Requirements for Structural Concrete and Commentary," and that adhesive anchors need to meet the qualification requirements in ACI 355.4-11, "Qualification of Post-Installed Adhesive Anchors in Concrete." The current RG 1.199, Revision 1, states that adhesive anchors are not within the scope of the RG.

Consequently, design and installation procedures for adhesive anchors should be peer-reviewed to ensure that they meet the requirements in ACI 318-14 and ACI 355.4-11 for the specific application and environment.

2.2.6 Rocking and Sliding of Unanchored Bodies

ASCE 43-18, Section 7.1, "Rocking and Sliding of Unanchored Bodies"

This section applies to unanchored bodies; the review of Section 7.2 below addresses buildings. The allowable capacity for sliding and rocking is taken as one-half of the displacement (linear or angular) available before unacceptable contact or other unacceptable performance (e.g., overturning, falling, or reaching design limits of umbilicals or attachments). As noted in the Commentary for this section, the basis for this limit is the committee's judgment that the probability of exceeding these design values for sliding and rocking is less than about 1 percent for the design input motion and less than about 10 percent for 150 percent of the design input motion.

Section 2.1.17 of this report, which contains the review of ASCE 4-16, Section 11, "Dynamic Sliding and Uplift, recommends requiring that components (excluding structures on grade) shall be anchored (not simply identified as a preference) to prevent sliding and uplift, unless they are expected to be moved occasionally (e.g., concrete block walls for radiation shielding).

Based on the evaluation of ASCE 4-16, Section 11, and the highly nonlinear nature of rocking and sliding for unanchored bodies, and because the allowable capacity is based on the committee's judgment, the value selected for the allowable capacity should be justified for the case being evaluated, with the justification addressing the items in Section 2.1.17 of this report.

2.2.7 Building Sliding and Overturning

ASCE 43-18, Section 7.2.1, "Building Sliding"

Static Coefficient of Friction

This section indicates that the static resistance should be greater than 1.1 x $V_{\mathrm{BaseShear}}$. This is consistent with the criterion in SRP Section 3.8.5, "Foundations," and is thus acceptable. Section 7.2.1 also provides Equation 7-1(b) for calculating the static resistance, based on the resistance from the effective cohesion force at the soil/concrete interface, frictional resistance due to the building normal force times the coefficient of friction (static), and passive resistance.

BNL recommends that the use of the static coefficient of friction along with the passive resistance be justified in the specific case being evaluated, since developing full or substantial passive resistance requires sufficient displacement to mobilize the passive resistance. Thus, the value selected for the coefficient of friction should be consistent with the extent of passive resistance relied on to resist sliding. If larger displacements corresponding to more substantial passive resistance are relied on, the dynamic rather than the static coefficient of friction may be required.

Governing Coefficient of Friction

Questions raised in past licensing reviews for nuclear power plants revealed that in some cases, the lowest coefficient of friction was not used for sliding evaluations. Therefore, to ensure that the evaluation uses the governing coefficient of friction, BNL recommends selecting the lowest coefficient of friction, considering the various sliding interfaces that exist (e.g., soil shear failure, concrete to soil, waterproofing material to soil/concrete, and concrete basemat to concrete mudmat (depending on the surface finish of the basemat)).

ASCE 43-18 and ASCE 4-16 do not describe ways to consider two or three dimensions in analyzing sliding and overturning stability, or ways to check stability when using nonlinear time history analysis. Therefore, implementations of these approaches in the stability analysis should be peer-reviewed for technical adequacy.

Alternative Approach When Exceedance of 1.1 x $V_{BaseShear}$ Occurs

For the case where Equation 7-1(a) of ASCE 4-16 is not satisfied, Section 7.2.1 states the following:

> ...it is permissible to demonstrate an acceptable condition by estimating the larger foundation movement resulting when the dynamic base shear momentarily exceeds the soil resistance and demonstrating the building remains functional for the resulting displacements. A refined estimate of the foundation movement shall recognize the time-dependent reversing nature of earthquake motion and the nonlinear soil resistance. If the functionality of the building is demonstrated for the larger displacements, an acceptable condition exists.

In view of the nonlinear nature of this type of analysis, the number of earthquake time histories needed for the evaluation, the approach used to implement the factor of 1.1 in the analysis, and the means of demonstrating that the building and other systems and components remain functional, the implementation of these items in the analysis should be peer-reviewed for technical adequacy.

Use of Nonlinear Analysis Approach in ASCE 4-16

Section 7.2.1 also describes a nonlinear analysis approach for estimating the foundation sliding movement. This approach uses the method in ASCE 4-16, which considers the sliding frequency of the structure and an effective sliding frequency defined in ASCE 4-16, Section 11.3.1, “Approximate Method for Analysis of Sliding of an Unanchored Rigid Body.” BNL recommends justifying any use of this simplified approach, addressing the items described in Section 2.1.17 of this report.

2.2.8 Seismic Separation

ASCE 43-18, Section 7.3, “Seismic Separation”

ASCE 4-16, Section 7.3, “Seismic Separation,” states that a minimum separation should be maintained between adjacent SSCs to accommodate the relative displacements using 2.0 times the SRSS of the two adjacent SSC displacements, as given by Equation 7-5 of Section 7.3. The section also makes an exception to this criterion, allowing the factor of 2.0 to be reduced, but to no less than 1.0, if the two displacements are based on a nonlinear response history analysis of the soil-structure system, provided that this achieves the performance goals of (1) less than a 1-percent probability of impact for the DBE shaking and (2) less than a 10-percent probability of impact for 150 percent of the DBE shaking.

In the evaluation of ASCE 4-16, Section 3.8.5, BNL recommends in Section 2.1.9 of this report the use of the ABSUM approach to calculate the relative deformation between structures when evaluating elements connected to or supported by multiple structures and when specifying the minimum clearance between structures. This recommendation of the ABSUM approach over the SRSS approach also applies to Equation 7-5 of ASCE 43-18, Section 7.3. The resulting relative deformation value should then be multiplied by the factor of 2.0 given in Equation 7-5, which apparently is needed to meet the two performance goals stated above. In addition, for design purposes, the acceptance criteria for checking clearances should account for tolerances in the construction of the SSCs. The effects of the construction tolerances should be added to 2.0 times the ABSUM of the calculated relative deformations.

As noted above, an exception in ASCE 43-18, Section 7.3, permits a reduction in the factor of 2.0 in some cases. Since the basis for such reductions is not evident and the amount of the reduction is problem dependent, any reduction in the factor of 2.0 should be justified.

Section 7.3 also states that the two displacements may be based on either linear elastic or nonlinear analysis, and that they should account for foundation/soil deformations if the two structures do not share a common foundation. Furthermore, if the two displacements from the adjacent structures are calculated using a linear elastic analysis of an inelastically designed (Limit State A, B, or C) structure with a fundamental frequency greater than 1 Hz, the calculated displacements should be increased to account for the higher ratio of inelastic to elastic displacement associated with such structures. Although the criteria in Section 7.3 are not clear, BNL recommends that this additional increase factor be applied simultaneously with the factor of 2.0 by which the ABSUM is multiplied; the tolerances of the adjacent structures should then be added. The value of the factor increasing the ratio of inelastic to elastic displacement should be justified.

2.2.9 Unreinforced Masonry Used as Movable Partitions, Barriers, and Radiation Shielding

<u>ASCE 43-18, Section 7.5, "Unreinforced Masonry Used as Movable Partitions, Barriers, and Radiation Shielding"</u>

This section states that unreinforced masonry is not an acceptable structural system for supporting nuclear-safety-related loads. It indicates that when used as a barrier, shielding, or partition, height-to-length ratios should be limited to prevent uplift. Also, this section allows the application of a reduction factor of 1/1.16 to the allowable stresses in TMS 402/602-16.

BNL recommends against the use of unreinforced masonry as a structural system for supporting nuclear-safety-related loads. Also, unreinforced masonry should not be placed near any safety-related SSCs, so that failure of masonry (including structural collapse, tipping, or sliding) does not affect the SSCs.

If unreinforced masonry used as movable partitions, barriers, or radiation shielding cannot be placed sufficiently far from safety-related SSCs, then BNL recommends supporting the unreinforced masonry for all applicable loadings, including seismic loadings, in order to prevent its failure and protect the integrity of any adjacent SSCs. The strength of an unreinforced masonry wall should not be relied upon to resist the applicable loadings, since cracks could develop over time and reduce its strength to an unknown value.

2.2.10 Seismic Qualification by Analysis

ASCE 43-18, Section 8.2, "Seismic Qualification by Analysis"

Chapter 8 of ASCE 43-18 addresses seismic qualification of equipment and distribution systems. ASCE 43-18, Section 8.2, "Seismic Qualification by Analysis," describes the seismic qualification by analysis using either an equivalent static analysis or a dynamic analysis method.

ASCE 43-18, Section 8.2.1.1, "Equivalent Static Methods"

For equipment and distribution systems, ASCE 43-18 cites the analytical procedures of ASCE 4-16, Section 4.5, "Equivalent Static Analysis." For the equivalent static analysis of piping systems, ASCE 43-18, Section 8.2.1.1, "Equivalent Static Methods," indicates that the method described in paragraph N-1225, "Simplified Dynamic Analysis," of ASME BPVC Section III, Division 1, Appendix N, is acceptable. For the dynamic analysis method, Section 8.2.1.2 also indicates the approach in Appendix N of ASME BPVC Section III.

In some cases, the approach in Appendix N of ASME BPVC Section III, which the NRC has not previously reviewed and endorsed, presents very specific and prescriptive methods for safety-significant nuclear power plant piping systems. The goal of the RIPB approach is to permit flexibility in the analysis while meeting performance objectives. Therefore, BNL recommends peer review of the methods used in a particular application, rather than endorsement by the NRC of specific methods such as those in Appendix N.

ASCE 43-18, Section 8.2.2.4, "Total Demand for Qualification by Analysis"

This section describes the calculation of total demand on equipment and distribution systems qualified by analysis for the combination of nonseismic and seismic loads. The seismic demand on a component may have an inertia component and a relative displacement component (often referred to as seismic anchor motion). For the relative displacement component, BNL recommends deriving the elastically computed relative displacements in accordance with Section 2.1.9 of this report. When inertia effects are combined with relative displacement effects, the method of combination should be consistent with Section 2.1.12 of this report.

2.2.11 Qualification by Testing and Experience Data

ASCE 43-18, Section 8.3, "Qualification by Testing and Experience Data"

It is generally acceptable to use Institute of Electrical and Electronics Engineers (IEEE) 344, "IEEE Standard for Seismic Qualification of Equipment for Nuclear Power Generating Stations," and ASME QME-1, "Qualification of Active Mechanical Equipment Used in Nuclear Facilities," for seismic qualification of electrical and mechanical components. However, some aspects of the use of testing and experience data in these standards require justification to demonstrate their seismic adequacy. These include the restriction of the testing frequency range to under 33 Hz, the use of earthquake experience data and test experience data, and consideration of the effects of high-frequency ground motions, if applicable (see RG 1.100, Revision 4, "Seismic Qualification of Electric and Mechanical Equipment for Nuclear Power Plants," issued May 2020, for further details on these areas). Therefore, while the use of IEEE 344 and ASME QME-1 is acceptable, the qualification and justification should be consistent with RG 1.100.

ASCE 43-18, Section 8.3.2.1, "Demand for Qualification Testing"

This section states that the demand for qualification by testing is given by the following equation

(Equation 8-3 in ASCE 43-18) with loads as defined below:

$$D = D_{NS} + \gamma_{test} D_S, \qquad \text{(Eq. 8-3)}$$

where—

- D denotes the total demand on the component.
- D_{NS} denotes additional loads, such as those arising from pressure or "state" of the equipment (e.g., energized electrical panels or devices), that are expected to be present during the earthquake shaking. Also, D_{NS} could include any pre-aging required before the seismic test. Often, $D_{NS} = 0$, other than the dead weight of components.
- D_S denotes seismic demand (response) of the component due to the seismic loads from an equivalent static analysis or from a dynamic analysis, generated, in either case, on input from a DBE corresponding to the SDC of the component.
- The factor γ_{test} is the ratio of the test response spectra to the required response spectra for qualification by testing.

If the qualification testing is performed in accordance with IEEE 344, setting the factor γ_{test} to 1.33 will meet the performance goals of this standard. If testing follows other procedures and criteria, the value of γ_{test} should be determined so as to meet the performance goals of ASCE 43-18.

Commentary Section C8.3.2.1 describes the basis for the value of 1.33 for γ_{test} using Reference C8-12 (published in 1994). This reference identifies the variability in seismic demand as 0.35 and the variability in seismic test capacity as 0.20, which lies at the 5-percent exclusion limit. Similar values are found in other reports referenced in this commentary section.

These references are more than two decades old; they were prepared at the time that the seismic margins studies were conducted. Of the eight references cited in Commentary Section C8, only Reference C8-13 has a date later than 2000. The approach described in this section does not reflect additional data on equipment performance collected in the past two decades (e.g., Refs. C8-7, C8-9, C8-11, C8-13). While the procedure and resulting load factors appear reasonable based on the cited references, BNL recommends the evaluation of more current data to support them.

<u>ASCE 43-18, Section 8.3.2.2, "Demand for Qualification by Test Experience Data," and Section 8.3.2.3, "Demand for Qualification by Earthquake Experience Data"</u>

Qualification by test experience is again given by Equation 8-3, except with γ_{test} replaced by γ_{TES}, which is the factor that must be applied to the test experience spectra to meet the performance goals of ASCE 43-18. In this case, qualification by test experience is given by

$$D = \gamma_{TES} D_S.$$

Similarly, qualification by earthquake experience data is given by

$$D = \gamma_{EED} D_S,$$

where γ_{EED} is the factor that must be applied to the earthquake experience data to meet the performance goals of ASCE 43-18.

In both cases, the load factors are determined so as to achieve the target performance goal using the method summarized in the discussion of Commentary Section C8.3.2.1 above. This procedure is acceptable, with the same caveat as identified above for Section 8.3.2.1.

2.2.12 Seismically Isolated Structures

ASCE 43-18, Chapter 9, "Seismically Isolated Structures"

This project did not cover seismic isolation. The NRC staff has a separate ongoing project investigating the applicability of seismic isolation to commercial nuclear power plants.

3 CONCLUSIONS AND RECOMMENDATIONS

This chapter discusses the parts of ASCE 4-16 and ASCE 43-18 that, in the authors' best technical judgment, warrant an exception, additional information, or a clarification. The details supporting the authors' conclusions and recommendations appear in Chapter 2 and in the appendix.

The review of ASCE 43-18 also identified several editorial and typographical errors to be addressed. These are listed at the end of Appendix A.2.

3.1 ASCE 4-16 Conclusions and Recommendations

3.1.1 Three-Dimensional vs. Planar Models

Separate analytical planar models for individual-direction excitations should not be used unless there is quantitative evidence that any error introduced by their use is conservative, relative to results from a three-dimensional analytical model.

3.1.2 Mesh Refinement

If the mesh convergence study indicates that convergence is from above (i.e., conservative), the 10-percent guideline is acceptable. If convergence is from below (i.e., unconservative), a more stringent convergence criterion (e.g., 5 percent) is recommended. The detailed guidance on mesh refinement in SRP Section 3.7.2 may be used to supplement the guidance in ASCE 4-16.

3.1.3 Damping

A more comprehensive technical basis is needed to support the use of Response Level 3 damping in conjunction with inelastic energy absorption factors. Until such a technical basis is developed, BNL recommends using Response Level 2 damping values in conjunction with inelastic energy absorption factors.

3.1.4 Modeling of Stiffness

If the simplified stiffness model has missing modes, then it will need refinement until all key responses at all key locations can be quantitatively shown to be within 10 percent on the conservative side or about 5 percent on the unconservative side. For this study, the total mass and the mass distribution must be essentially the same in both the simplified stiffness model and the realistic stiffness model.

3.1.5 Modeling of Mass

The current SRP Section 3.7.2 provides sensible guidance that allows the analyst to assess when a sufficiently accurate model of mass has been developed for dynamic analysis. BNL recommends this guidance over the traditional 90-percent effective mass participation rule of thumb.

SRP guidance and ASCE 4-16, Section 3.4.2, differ in three ways for specifying mass to be included in the seismic analysis model:

(1) SRP Section 3.7.2 specifies 50 psf for miscellaneous dead weight, whereas ASCE 4-16 specifies actual miscellaneous dead weight. ASCE 4-16 is considered acceptable for an RIPB design methodology.

(2) SRP Section 3.7.2 specifies 25 percent of design live load, whereas ASCE 4-16 specifies 25 percent of design live load, with a maximum of 50 psf for live loads greater than 200 psf. ASCE 4-16 is considered acceptable for an RIPB design methodology.

(3) SRP Section 3.7.2 specifies 75 percent of design snow load, whereas ASCE 4-16 specifies that design flat roof snow loads of 30 psf or less need not be included, and that, where flat roof snow loads exceed 30 psf, 25 percent of the uniform design snow load must be included. NUREG/CR-6926 (which reviewed the adequacy of ASCE 43-05) accepted the use of 25 percent of the design snow load, based on review of other standards (e.g., ASCE 7-05, IBC 2003); therefore, it is acceptable.

3.1.6 Hydrodynamic Mass Effects

ASCE 4-16, Section 3.6.3(c), describes the treatment of impulsive mass and sloshing mass for situations where the basin wall is not rigid or when local stresses are of interest, as depicted in Figure 3-1; however, it provides no references for the technical basis. The Commentary provides no additional information or references for Figure 3-1; it simply states, “If the walls are not relatively rigid and/or the local stress effects are important, then the impulsive mass and convective spring constants should be distributed as specified in this standard.” Implementations of Figure 3-1 should be peer-reviewed for technical adequacy.

3.1.7 Dynamic Coupling Criteria

BNL recommends the use of a coupled model if the uncoupled response is unconservative by more than 5 percent relative to coupled response, to ensure that decoupling will not contribute significantly to underprediction of the seismic response.

3.1.8 Additional Requirements for Modeling Specific Structures

SDOF oscillators tuned to the fundamental out-of-plane frequencies of the floors and walls can be incorporated directly into a stick model. BNL recommends that this be the modern standard for lumped-mass stick models, to ensure that their analysis can identify potential out-of-plane amplification of both wall and floor responses. Depending on the frequency range of interest in the analysis, it may be necessary to add more SDOF oscillators representing higher modes of the walls and floors.

3.1.9 Requirements for Adjacent Structures

BNL recommends ABSUM over SRSS for checking building clearances and analyzing seismic anchor movements.

3.1.10 Analysis of Structures

The missing mass contribution must be incorporated in both mode-superposition time history analysis and response spectrum analysis to ensure accurate results. The NRC eliminated the earlier 10-percent approximation from SRP Section 3.7.2 in 2007 and replaced it with calculation of the missing mass contribution to total response.

3.1.11 Combination of Multiple Response Parameters

To clarify the proper implementation of 100-40-40 spatial combination for multiple response parameters, BNL has performed a simple quantitative study that validated industry's implementation, as presented in Commentary Section C4.3.3. BNL recommends 100-40-40 spatial combination over SRSS spatial combination.

3.1.12 Combination of Seismic Inertial Response with Seismic Anchor Movements

SRSS combination is acceptable for both USM and ISM analysis, consistent with the guidance in ASCE 4-16.

3.1.13 Nonlinear Static Analysis

This section is not applicable to design of commercial nuclear power plant structures.

3.1.14 In-Structure Response Spectra

A 15-percent reduction in peak amplitude for narrow, sharp peaks in the ISRS appears to be reasonable and by itself would not jeopardize achievement of the target of 80-percent nonexceedance probability. However, when each of several other approximations currently permitted (although not necessarily promoted) in the ASCE 4-16 analysis process is allowed to underpredict the response by up to 10 percent, the deterministically derived ISRS with sharp, narrow peaks may have already compromised the achievement of 80-percent nonexceedance probability.

Therefore, the acceptance of peak clipping as defined in ASCE 4-16, Sections 6.2.3 and 6.3.2, is conditional, pending detailed demonstration that the entire analysis process, considering all sources of potential unconservatism, meets the 80-percent nonexceedance probability target.

3.1.15 Dynamic Soil Pressures on Walls

BNL recommends the use of one of three bounding seismic soil pressures on embedded walls, unless otherwise justified:

(1) pressure calculated in accordance with ASCE 4-16, Section 8.2.1, "Dynamic Finite Element Analysis"

(2) pressure calculated in accordance with ASCE 4-16, Section 8.2.2, "Simplified Method"

(3) pressure equal to the fraction of the passive earth pressure that is used for the structure stability evaluation

3.1.16 Distribution Systems

An exception should be taken to ASCE 4-16, Chapter 10, on the basis that it provides no useful additional information for deriving the seismic demand on distribution systems in new commercial nuclear power plants. Instead, the codes and standards applicable to these types of distribution systems should be used.

3.1.17 Dynamic Sliding and Uplift Analysis

BNL recommends avoiding the use of unanchored structures and components (excluding buildings). If this is not possible, BNL recommends that the implementation of the nonlinear response analysis be peer-reviewed for technical adequacy.

3.2 ASCE 43-18 Conclusions and Recommendations

3.2.1 Seismic Design Criteria

The assumption that current material codes and specifications deliver capacities at the 2-percent (or less) exclusion limit appears to be reasonable, for the most part; Section 1.1 of ASCE 43-18 is therefore judged to be acceptable.

BNL strongly recommends performing a study to provide further data to support this assumption. It also recommends performing a peer review during the design phase for the 2-percent exclusion limit on design strength for the SSCs and the corresponding codes and standards being used.

3.2.2 Integration of Other Codes and Standards

Some sections of ASCE 43-18 cite codes and standards without dates or do not give dates consistently.

BNL recommends that the versions of all codes and standards cited in ASCE 43-18 be taken as those in effect on October 23, 2018, which is the date of the draft of ASCE 43-18 reviewed in this report. If a later date is chosen, then any changes in the later version should be evaluated.

3.2.3 Component Capacities

BNL recommends that the design and detailing of reinforced concrete conform to ACI 349-13 and RG 1.142, Revision 3, for concrete structures, and RG 1.199, Revision 1, for anchoring components and structural supports in concrete.

BNL recommends the use of the updated AISC N690-18 and the associated NRC draft regulatory guide DG-1304 for the design of steel structures and SC walls.

AISC N690-18 contains two approaches for design of steel structures: LRFD and ASD. LRFD is a probability-based methodology and is well suited to performance-based design. Therefore, for RIPB applications, BNL recommends the LRFD option in AISC N690-18.

For reinforced and prestressed concrete containments, BNL recommends the use of ASME BPVC Section III, Division 2, and the associated RG 1.136, Revision 3, to determine component capacity.

For steel containments, BNL recommends the use of ASME BPVC Section III, Division 1, Subsection NE, and the associated RG 1.57, Revision 2, to determine component capacity.

The use of reinforced masonry walls is acceptable; however, their design, whether following ACI 530 and ACI 530.1 or the updated TMS 402-16 and TMS 602-16, should be peer-reviewed for technical adequacy, because the NRC has not previously reviewed any of these standards.

3.2.4 Inelastic Energy Absorption Factor, F_μ, and Deformation Acceptance Criteria

When using the inelastic energy absorption factors in Section 5.1.3 or the allowable deformation limits in Section 5.2.3 of ASCE 43-18, it should be shown that the relevant assumptions and judgments in WSRC-TR-2001-00037 apply to the particular structure being designed.

3.2.5 Anchorage

Adhesive anchors are unique; they are subjected to dynamic loads, there is limited experience of their use in nuclear power plant applications, and they are outside the scope of RG 1.199, Revision 1. Furthermore, if qualified, they can be used in environments with elevated temperatures or radiation. Therefore, design and installation procedures for adhesive anchors should be peer-reviewed to ensure that they meet the requirements in ACI 318-14 and ACI 355.4-11 for the specific application and environment.

3.2.6 Rocking and Sliding of Unanchored Bodies

Based on the evaluation of ASCE 4-16, Section 11, and the highly nonlinear nature of rocking and sliding of unanchored bodies, and because the allowable capacity is based on the committee's judgment, the value selected for the allowable capacity should be justified for the case being evaluated, with the justification addressing the items in Section 2.1.17 of this report.

3.2.7 Building Sliding and Overturning

Static Coefficient of Friction

The value selected for the coefficient of friction should be consistent with the extent of passive resistance relied on to resist sliding. If larger displacements corresponding to more substantial passive resistance are relied on, the dynamic rather than the static coefficient of friction may be required.

Governing Coefficient of Friction

To ensure that the evaluation uses the governing coefficient of friction, BNL recommends selecting the lowest coefficient of friction, considering the various sliding interfaces that exist (e.g., soil shear failure, concrete to soil, waterproofing material to soil/concrete, and concrete basemat to concrete mudmat (depending on the surface finish of the basemat)).

ASCE 43-18 and ASCE 4-16 do not describe ways to consider two or three dimensions in analyzing sliding and overturning stability, or ways to check stability when using nonlinear time history analysis. Therefore, implementations of these approaches in the stability analysis should be peer-reviewed for technical adequacy.

Alternative Approach When Exceedance of 1.1 x $V_{BaseShear}$ Occurs

In view of the nonlinear nature of the analysis described in this section, the number of earthquake time histories needed for the evaluation, the approach used to implement the factor of 1.1 in the analysis, and the means of demonstrating that the building and other systems and components remain functional, the implementation of these items in the analysis should be peer-reviewed for technical adequacy.

Use of Nonlinear Analysis Approach in ASCE 4-16

BNL recommends justifying any use of the simplified approach described in ASCE 4-16, Section 7.2.1, addressing the items described in the review of ASCE 4-16, Section 11.3, in Section 2.1.17 of this report.

3.2.8 Seismic Separation

BNL recommends the use of the ABSUM of two adjacent SSCs, rather than the SRSS method, to determine the minimum separation required. The resulting relative deformation value should then be multiplied by the factor of 2.0 given in Equation 7-5 of ASCE 43-18, Section 7.3. In addition, for design purposes, the acceptance criteria for checking clearances should account for tolerances in the construction of the SSCs.

An exception in ASCE 43-18, Section 7.3, permits a reduction in the factor of 2.0 discussed above, but to no less than 1.0, if the two displacements are based on a nonlinear response history analysis of the soil-structure system and the performance goals described in ASCE 4-16, Section 7.3, are achieved. Since the basis for such reductions is not evident and the amount of the reduction is problem dependent, any reduction in the factor of 2.0 should be justified.

Section 7.3 indicates that if the two displacements from the adjacent structures are calculated using a linear elastic analysis of an inelastically designed (Limit State A, B, or C) structure with a fundamental frequency greater than 1 Hz, the calculated displacements should be increased to account for the higher ratio of inelastic to elastic displacement associated with such structures. BNL recommends that this additional increase factor be applied simultaneously with the factor of 2.0 by which the ABSUM is multiplied; the tolerances of the adjacent structures should then be added. The value of the factor increasing the ratio of inelastic to elastic displacement should be peer-reviewed for technical adequacy.

3.2.9 Unreinforced Masonry Used as Movable Partitions, Barriers, and Radiation Shielding

BNL recommends against the use of unreinforced masonry as a structural system for supporting nuclear-safety-related loads. Also, unreinforced masonry should not be placed near any safety-related SSCs, so that failure of masonry (including structural collapse, tipping, or sliding) does not affect the SSCs.

If unreinforced masonry used as movable partitions, barriers, or radiation shielding cannot be placed sufficiently far from safety-related SSCs, then BNL recommends that the unreinforced masonry be supported for all applicable loadings, including seismic loadings, in order to prevent its failure and protect the integrity of any adjacent SSCs.

3.2.10 Seismic Qualification by Analysis

For the equivalent static analysis and dynamic analysis of piping systems, BNL recommends that if the approach of ASME BPVC Section III, Division 1, Appendix N, is used, it should be peer-reviewed for the specific application.

The seismic demand on a component may have an inertia component and a relative displacement component. For the relative displacement component, BNL recommends that the elastically computed relative displacements be derived in accordance with Section 2.1.9 of this report. When the inertia effects are combined with the relative displacement effects, the method of combination should be consistent with Section 2.1.12 of this report.

3.2.11 Qualification by Testing and Experience Data

It is generally acceptable to use IEEE 344 and ASME QME-1 for seismic qualification of electrical and mechanical components. However, some aspects of the use of testing and experience data in these standards require justification to demonstrate their seismic adequacy. These include the restriction of the testing frequency range to under 33 Hz, the use of earthquake experience data and test experience data, and consideration of the effects of high-frequency ground motions, if applicable (see RG 1.100 for further details on these areas). Therefore, while the use of IEEE 344 and ASME QME-1 is acceptable, the qualification and justification should be consistent with RG 1.100.

<u>ASCE 43-18, Section 8.3.2.1, "Demand for Qualification Testing"</u>

This section states that the demand for qualification by testing is given by Equation 8-3 of ASCE 43-18:

$$D = D_{NS} + \gamma_{test} D_S.$$

The factor γ_{test} is the ratio of the test response spectra to the required response spectra for qualification by testing; its value is taken as 1.33 in ASCE 43-18. The references cited in support of this value are more than two decades old. While the procedure and resulting load factors appear reasonable based on these references, BNL recommends the evaluation of more current data to support them.

<u>ASCE 43-18, Section 8.3.2.2, "Demand for Qualification by Test Experience Data," and Section 8.3.2.3, "Demand for Qualification by Earthquake Experience Data"</u>

Qualification by test experience is again given by Equation 8-3, except with γ_{test} replaced by γ_{TES}, which is the factor that must be applied to the test experience spectra to meet the performance goals of ASCE 43-18. In this case, qualification by test experience is given by

$$D = \gamma_{TES} D_S.$$

Similarly, qualification by earthquake experience data is given by

$$D = \gamma_{EED} D_S,$$

where γ_{EED} is the factor that must be applied to the earthquake experience data to meet the performance goals of ASCE 43-18.

In both cases, the load factors are determined so as to achieve the target performance goal using the method summarized in the discussion of Commentary Section C8.3.2.1 above. This procedure is acceptable, with the same caveat as identified above for Section 8.3.2.1.

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APPENDIX: FURTHER DETAILS ON ASSESSMENT OF ASCE 4-16 AND ASCE 43-18 (DRAFT)

Section 2 of this report gave the details of Brookhaven National Laboratory's (BNL's) assessment of the analysis and design provisions in American Society of Civil Engineers (ASCE) Standards 4-16, "Seismic Analysis of Safety-Related Nuclear Structures," and 43-18 (Draft), "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities." This appendix provides further details and clarifies certain items. It also lists some editorial and typographical corrections to be made in ASCE 43-18.

A.1 ASCE 4-16

A.1.1 Damping

ASCE 4-16, Section 3.2.2, "Damping," states the following:

(a) Values of damping to be used in linear dynamic analyses are given in Table 3-1 as a function of response level. The response level is determined on a component basis and is given as

1. At response level 1, the average stresses in members of steel frames should be low. For steel beams, columns, and braces, demands are less than 50% of the capacity remote from member connections. At response level 1, concrete walls, beams, and columns have not cracked significantly. Demands in concrete columns and beams are less than 50% of the nominal strength. Average shear stress demands in shear critical (low aspect ratio) walls are less than 3 $(f'_c)^{1/2}$, f'_c = specified 28-day compressive strength of concrete (psi);

2. At response level 2, demands on steel beams, columns, and braces at locations remote from connections are less than the nominal capacities determined using national consensus standards. Member demands are generally between about 50% and 100% of the nominal strength, and/or stresses are generally between about 50% and 100% of the yield capacity of major resisting structural elements. At response level 2, concrete walls, beams, and columns have cracked significantly. Demands in concrete columns and beams are greater than 50% the nominal strength. Demands in shear critical walls are greater than 3 $(f'_c)^{1/2}$.

3. At response level 3, structures have responses ranging from "limited permanent distortion" to "large permanent distortion," which corresponds to ASCE 43-05 limit states C, B, or A. The structural element forces calculated with the inelastic energy absorption factor, F_μ = 1, for the loading combination, must exceed the nominal code capacity, or stresses must exceed the yield strength, in major load-resisting structural elements.

The modified structural element design forces calculated with the appropriate F_μ in ASCE 43-05 for all loading combinations shall be less than the appropriate nominal capacity.

These values are applicable to all modes of a structure constructed of the same material. Damping values for systems that include two or more substructures, such as a combined concrete and steel structure, or soil-structure systems, shall be obtained as described in Section 3.5.

(b) For generating input motions to subsystems (i.e., equipment or piping attached to a building) or for evaluating structural displacements, the level of damping will depend upon the response levels and the extent of concrete cracking. In lieu of iterative analyses to determine the actual response level and associated damping value, response level 1 damping values may be used for generation of in-structure spectra and displacements.

Table 3-1. Viscous Damping Expressed as a Fraction of Critical Damping

Structure Type	Response Level 1	Response Level 2	Response Level 3
Welded aluminum structures	0.02	0.04	0.04
Welded and friction-bolted steel structures	0.02	0.04	0.07
Bearing-bolted steel structures	0.04	0.07	0.10
Prestressed concrete structures (without complete loss of prestress)	0.02	0.05	0.07
Reinforced concrete structures	0.04	0.07	0.10
Reinforced masonry shear walls	0.04	0.07	0.10

(c) Response level 1 damping values shall be used for uncracked concrete, and response level 2 damping values may be used for significantly cracked concrete.

(d) For design or analysis of safety-related structures, response level 2 damping values may be used independent of the state of stress in the structures for limit states A, B, and C. For limit state D, either response level 1 or response level 2 damping may be appropriate and shall be justified on a case-by-case basis with consideration of the safety function of the structure.

(e) Damping values higher than response level 2 values are generally appropriate for structures responding well into their nonlinear range. The analysis may account for such damping, if properly justified, either through the use of a response level 3 viscous damping value or a combined response level 1 viscous damping value and hysteretic energy dissipation mechanism.

(f) The use of damping values higher than those in Table 3-1 is permitted if justified.

ASCE 4-16, Commentary Section C3.2.2, “Damping,” states the following:

C3.2.2 Damping

(a) The damping values indicated in this standard are taken from ASCE 43-05. Damping values for structures constructed with high-strength friction-bolted connections, which behave similarly to welded structures, are set lower than damping values for structures utilizing bearing bolted connections because the latter are much more likely to experience higher damping in an earthquake through slippage and working of joints and connections.

(b) For structural design or analysis, the use of the higher damping values (response level 2) is justified because before the structural elements reach a code allowable stress limit, they would experience response level 2. The exception is made for elastic buckling, where the limiting value may be reached before response level 2 is reached.

(c) The in-plane concrete shear wall should be considered significantly cracked when either the average shear stress state exceeds $3\sqrt{f'_c}$ or when the peak flexural stress calculated by elastic analysis exceeds the modulus of rupture of concrete, $7.5\sqrt{f'_c}$ (ACI 349). Rocks et al. (2011) provides experimental data for shear wall testing that provides the basis for $3\sqrt{f'_c}$, defining significant cracking as $\frac{k_s}{k_o} \cong 0.5$, ratio of secant stiffness to the initial stiffness.

ASCE 4-16, Section 3.5 and Commentary Section C3.5, “Modeling of Damping,” contain several detailed numerical procedures for modeling damping in finite element analysis. These procedures are generally acceptable based on the cited references. However, their implementation should be peer-reviewed for technical adequacy.

ASCE 4-16, Section 6.5, “Subsystem Damping Values,” states the following:

The requirements for in-structure response spectra for subsystem seismic analyses depend on subsystem damping. Subsystem damping depends on response levels expected during the excitation. Generally, response levels depend on the demand-to-capacity ratio (D/C) of seismic load-resisting elements in the subsystem to be analyzed. Table 6-1 approximately relates demand-to-capacity ratios to response levels, where D/C is determined as an average over the seismic load-resisting elements.

For representative subsystems, Table 6-2 provides recommended damping values for various categories of subsystems. These damping values provide guidance as to the damping values for which in-structure response spectra should be provided.

Table 6-1. Estimating Subsystem Response Levels

Response Level	*D/C*
1	≤ 0.5
2	≈ 0.5 to 1.0
3	≥ 1.0

Table 6-2. Damping Values for Subsystems

	Damping (% of critical)		
Type of Subsystem	Response Level 1	Response Level 2	Response Level 3
Piping	5	5	5
Distribution systems			
Cable trays 50% or more full and ZPA of support locations of 0.25*g* or greater	5	10	15
For other cable trays, those with rigid fireproofing and conduits	5	7	7
Massive, low-stressed mechanical components (pumps, compressors, fans, motors, etc.)	2	3	[a]
Light-welded instrument racks	2	3	[a]
Electrical cabinets and other equipment	3	4	5[b]
Liquid-containing metal tanks—impulsive mode	2	3	4
Liquid-containing reinforced concrete tanks—impulsive mode[c]	3	5	7
Sloshing mode (metal and concrete tanks)	0.5	0.5	0.5

[a]Should not be stressed to Response Level 3.
[b]5% damping may be used for anchorage and structural failure modes that are accompanied by at least some inelastic response. Response Level 1 damping values shall be used for functional failure modes such as relay chatter or relative displacement issues that may occur at a low cabinet stress level.
[c]If an unlined tank is intended to function as a liquid retention barrier, then the tank should not be stressed beyond Response Level 1.

ASCE 4-16, Commentary Section C6.5, “Subsystem Damping Values,” states the following:

> Selection of the damping values to be considered in the development of in-structure response spectra for subsystem seismic analyses should be guided by ASCE/SEI 43-05 (ASCE 2005). Damping values for structural subsystems shall be the same as those given for similar structural systems in Section 3 of this standard.

ASCE 4-16, Section 10.7, “Damping,” states the following:

> Damping values for distribution systems are defined in Table 6-2. Those values are suitable for analysis purposes to define the applicable seismic response spectrum using linear elastic analysis methods. The values used for ductwork, conduit, and tubing are the same as for piping. Damping values used for raceways depend on the amount of wire (full percentage) in the raceway and the effect of applied fireproofing.

ASCE 43-18, Section 3.3.3, “Damping Values for SSCs,” states the following:

> The damping values to be used for determining seismic design loads by linear elastic analysis for SSCs are presented ASCE 4 Table 3-1 and Table 6.5-2 and are reproduced here in Table 3-1. These are presented as a function of the average Response Level determined by demand-to- capacity ratios. The De/C ratios are calculated on an element basis (C = code capacity, De = total elastically computed demand, including nonseismic loads). The appropriate Response Level can be estimated from Table 3-2. Values of damping for steel concrete (SC) composite elements and heating, ventilation, and air conditioning (HVAC) are not addressed in ASCE 4-14 but are included in Table 3-1.
>
> When performing the seismic analysis, consideration of the actual Response Level is required for generation of in-structure response spectra. In lieu of iterative analyses to determine the actual Response Level and associated damping value, Response Level 1 damping values are permitted for generation of in-structure spectra. Response Level 1 damping values shall be used if elastic buckling or brittle material failure control the design.
>
> Response Level 3 damping may be used for evaluating seismic-induced forces and moments in structural members by elastic analysis without consideration of the actual Response Level for Limit States A, B, or C. Response Level 2 damping may be used for Limit State D.
>
> Response Level 1 damping values shall be used when performing a nonlinear inelastic response analysis that explicitly incorporates hysteretic energy dissipation. A summary of the maximum Response Level that shall be used for selecting the damping value is shown in Table 3-3.

ASCE 43-18, Commentary Section C3.3.3, “Damping Values for SSCs,” states the following:

> Table 3-1 reproduces information presented in Chapter 3 of ASCE 4-16. Commentary is provided in ASCE 4-16 and is not repeated here.
>
> Table 3-1 includes additional information on damping values for SC walls and

HVAC.

Damping values for HVAC provided in Table 3-1 are from NUREG/CR-6919 (Ref. [C3-2)]. This reference also provides values of damping for supported SSCs not identified in Table 3-1.

Table 3-1 includes information on SC walls that was not available at the time ASCE 4-16 was being compiled and is included here for completeness. The values of damping assigned to SC walls are 3%, 5% and 10% at Response Levels 1, 2 and 3 as defined in ASCE 4-16 and Table 3-2. These are based on Akiyama et al. (Ref. [C3-3], Epackachi et al. (Ref. [C3-4]), and Seo et al. (Ref. [C3-5]).

Demands on structural components of the seismic framing system are determined using the provisions of ASCE 4-16. The numerical model of the seismic framing system should include all components that contribute mass and stiffness and materially affect the deformation of the seismic framing system. The numerical model of the framing system should include all sources of seismically reactive mass. A product of the analysis per ASCE 4-16 will be input motions to supported SSCs that might contribute mass but not stiffness to the seismic framing system.

The primary purpose of the data in Section 3.3.3 is to enable calculations of demand. Response Levels should be established for each supported SSC using the demand-capacity procedures of ASCE 4-16, which will require analysis of the supported SSC, a calculation of demand, and a calculation of the ratio of demand to capacity.

Alternately, damping associated with Response Level 1 per Table 3-1 may be used for calculation of demands on supported SSCs. In general when computing soil–structure interaction effects, De/C ratios for the best estimate case of soil properties may be used to determine Response Level.

Nonlinear analysis of supported SSCs may be performed but Response Level 1 values of damping should be used to avoid duplication of the hysteretic energy dissipation that is a product of nonlinear action.

TABLE 3-1. Specified Damping Values for Dynamic Analysis

	Damping (% of Critical)		
Type of Component	Response Level 1	Response Level 2	Response Level 3
Welded and friction-bolted metal structures	2	4	7
Bearing-bolted metal structures	4	7	10
Prestressed concrete structures (without complete loss of prestress)	2	5	7
Reinforced concrete structures	4	7	10
Steel-Plate Composite (SC) Walls	3	5	10
Reinforced masonry shear walls	4	7	10
Piping	5	5	5
HVAC			
• Pocket Lock	7	10	
• Companion Angle	5	7	*
• Welded	3	4	
Distribution systems:			
• Cable trays 50% or more full and in-structure response spectrum Zero Period Acceleration of 0.25 g or greater	5	10	15
• For other cable trays, cable trays with rigid fireproofing and conduits	5	7	7
Massive, low-stressed mechanical components (pumps, compressors, fans, motors, etc.)	2	3	*
Light welded instrument racks	2	3	*
Electrical cabinets and other equipment	3	4	5**
Liquid containing metal tanks:			
• Impulsive mode	2	3	4
• Sloshing mode	0.5	0.5	0.5

Notes:

* Should not be stressed to Response Level 3. Use damping for Response Level 2.

** May be used for anchorage and structural failure modes that are accompanied by at least some inelastic response. Response Level 1 damping values shall be used for functional failure modes such as relay chatter or relative displacement issues that may occur at a low cabinet stress level.

TABLE 3-2. Estimating Damping Response Level

Response Level*	D_e/C
1	≤ 0.5
2	≈ 0.5 to 1.0
3	≥ 1.0

Note:
* See ASCE 4 Chapter 3 for more information.

TABLE 3-3. Summary of Maximum Response Level for Damping

Elastic buckling conditions control design	Response Level 1
Generation of in-structure spectra	Response Level 1 (Response Level 2, if justified)
Limit State D	Response Level 2
Limit States A, B, or C:	
Elastic analysis	Response Level 3*
Inelastic time-history response analysis	Response Level 1 (Response Level 2, if justified)

Note:
* Only to be used with adequate ductile detailing. However, functionality of SSCs must be given due consideration. Response Level 3 corresponds to Limit State C; Response Level 3 may also be used for Limit States A and B.

ASCE 43-18, Commentary Section C5.1.3, "Inelastic Energy Absorption Factor, F_μ," states the following:

> Consideration of Response Level 3 Damping with F_μ.
>
> The nonlinear cyclic behavior of low-rise shear walls is dominated by pinched hysteresis loops. Pinched hysteresis loops dissipate less energy than full hysteresis loops encountered in special moment frames. Thus, shear walls are used in WSRC-TR-2002-00333 [C5-19] to study of the interaction between deformation limits, F_μ, damping, SSI response and weak/soft story effects. This study indicates that the F_μ factors in this Standard are conservative for structures with and without SSI, where the SSI response may include significant radiation damping. The study also indicates that concentrating the inelastic deformation in the shear wall, combined with SSI flexibility, does not require a weak/soft story reduction factor.
>
> Additionally, the study demonstrates that the F_μ factors are generally conservative when combined with Response Level 3 damping and SSI with significant radiation damping. Thus, it is the judgment of the Working Group that the F_μ factors in this Standard may be combined with Response Level 3 damping in accordance with Section 3.3.3 of this Standard and the F_μ factors may be used in structures with or without SSI effects.

BNL ASSESSMENT

BNL reviewed the structural and equipment damping values specified in ASCE 4-16 and ASCE 43-18 and compared them to the current guidance in NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition" (SRP). (See Regulatory Guide (RG) 1.61, Revision 1, "Damping Values for Seismic Design of Nuclear Power Plants," issued March 2007.) The specified Response Level 1 and Response Level 2 damping values are consistent with the operating-basis earthquake (OBE) and safe-shutdown earthquake (SSE) damping values in RG 1.61, Revision 1, with one notable exception.

For piping damping, ASCE has adopted the recommendation of the ASME, namely 5-percent damping for both Response Level 1 and Response Level 2. The current staff position in RG 1.61, Revision 1, for piping damping is 3 percent for OBE and 4 percent for SSE. The latter values have been confirmed in a study performed by BNL for the staff, following release of RG 1.61, Revision 1. (See PVP2010-25465, "Assessing Equivalent Viscous Damping Using Piping System Test Results," issued 2010.) Consequently, BNL recommends taking an exception to the Response Level 1 and 2 piping damping values listed in ASCE 4-16 and ASCE 43-18, replacing them, respectively, with the OBE and SSE piping damping values in RG 1.61, Revision 1.

ASCE 4-16 and ASCE 43-18 indicate the use of Response Level 1 (comparable to OBE) damping values when explicit nonlinear inelastic analysis is performed. This is consistent with the current staff position in RG 1.61, Revision 1.

ASCE 4-16 and ASCE 43-18 indicate the use of Response Level 1 damping in building analyses intended to generate in-structure response spectra for subsequent analysis of attached SSCs. This is also consistent with the current staff position in RG 1.61, Revision 1.

The review also considered the correct use of Response Level 3 damping in analyses for Limit States C, B, and A (limited permanent distortion to very large permanent distortion). BNL finds the Response Level 3 damping values specified in ASCE 4-16 and ASCE 43-18 to be acceptable for use with linear elastic analysis.

However, the significant technical issue is whether Response Level 3 damping values are appropriate when inelastic energy absorption factors are also applied to linear elastic results. This represents a double reduction of demand (i.e., energy dissipation by use of the inelastic energy absorption factor and energy dissipation by use of the Response Level 3 damping).

BNL's review of prior research in this area (e.g., NUREG/CR-3805, "Engineering Characterization of Ground Motion—Task 1: Effects of Characteristics of Free-Field Motion on Structural Response," issued May 1984) does not appear to support the guidance in the Commentary to ASCE 43-18, namely that Response Level 3 damping can be used in conjunction with inelastic energy absorption factors. The technical basis cited in the Commentary is extremely limited in scope. BNL considers that a more comprehensive technical basis is needed to support the use of Response Level 3 damping in conjunction with inelastic energy absorption factors.

Until such a technical basis is developed, BNL recommends that Response Level 2 damping values be used in conjunction with inelastic energy absorption factors.

Modeling of Mass

ASCE 4-16, Section 3.4.1, "Discretization of Mass," states the following:

> (a) The inertial mass properties of a structure may be modeled by assuming that the structural mass and associated rotational inertia are discretized and lumped at node points of the model. Alternatively, the consistent mass formulation may be used.
>
> (b) When appropriate, three translational and three rotational degrees of freedom shall be used at each node point. Mass for some degrees of freedom, such as rotational degrees of freedom, may be neglected, provided that their exclusion does not unconservatively affect the response parameters of interest by more than 10% and the torsional response is not affected. The following conditions shall be met:
>
> > 1. Structural mass shall be distributed or lumped so that the total mass and the center of gravity are preserved, both for the total structure and for any of its major components that respond in the direction of motion.
> >
> > 2. The number of dynamic degrees of freedom, and hence the number of lumped masses, shall be selected so that all significant vibration modes (at least 90% effective mass participation) of the structure can be evaluated. For a structure with distributed mass, the number of degrees of freedom in a given direction shall be equal to at least twice the number of significant modes in that direction.

In particular, Section 3.4.1(b) states, "Mass for some degrees of freedom, such as rotational degrees of freedom, may be neglected, provided that their exclusion does not unconservatively affect the response parameters of interest by more than 10% and the torsional response is not affected." This guidance is considered reasonable, because the current norm is to develop three-dimensional models with structure mass distributed throughout the model. In such models, mass is typically not assigned to rotational degrees of freedom, and this has minimal impact on the solution. For stick models, however, rotational degrees of freedom may be significant and should not be neglected.

The guidance provided in paragraph 3.4.1(b)(2) is historical, providing no improvement on earlier dynamic analysis models. On the other hand, the current SRP Section 3.7.2, "Seismic System Analysis," provides the following detailed guidance:

> The adequacy of the number of discrete mass degrees of freedom can be confirmed by (1) preliminary modal analysis, and (2) correlation between static analysis results using the dynamic model and static analysis results using a distributed mass representation.
>
> (1) It is important to ensure that, for each excitation direction (2 horizontal and one vertical), all modes with frequencies less than the ZPA (or PGA) frequency of the corresponding spectrum are adequately represented in the dynamic solution. Preliminary modal analysis should be performed to establish that a sufficient number of discrete mass degrees of freedom have been included in the dynamic model to (a) predict a sufficient number of modes, and (b) produce mode shapes

that are reasonably smooth. If a mode shape exhibits rapid change in modal displacement between adjacent mass degrees of freedom, additional mass degrees of freedom should be added until reasonably smooth mode shapes are obtained for all modes to be included in the dynamic analysis.

(2) After completion of (1), simple 1g static analyses of the dynamic model should be performed for each of the three (3) excitation directions and compared to the corresponding results obtained from static analyses that utilize a distributed mass representation. Lack of correlation, particularly in the vicinity of and at support locations, is indicative of an insufficient number of discrete mass degrees of freedom.

SRP Section 3.7.2 provides sensible guidance that allows the analyst to assess when a sufficiently accurate model of mass has been developed for dynamic analysis. BNL recommends this guidance over the traditional 90-percent effective mass participation rule of thumb.

Dynamic Coupling Criteria

ASCE 4-16, Section 3.7.1, “General Requirements,” provides the general criteria for coupling of a primary structure and secondary system. This section states the following:

(a) Coupled analysis of a primary structure and secondary system shall be performed when the effects of dynamic response interaction are significant according to the criteria of Sections 3.7.2 and 3.7.3.

(b) If a coupled analysis will not increase the response of key design parameters of the primary system over that of a decoupled analysis by more than 10%, then a coupled analysis is not required. However, the requirements of Section 3.7.3 regarding the static constraint shall be considered.

(c) In applying Sections 3.7.2 and 3.7.3, one subsystem at a time may be considered, unless the subsystems are essentially identical (uncoupled dominating frequencies within +/-10%) and located together, in which case the subsystem masses shall be lumped together.

(d) When coupling is required, a detailed model of the secondary system is not required for global response of the primary structure, provided that the simple model adequately represents the major effects of interaction between the two parts. When a simple model is used, the secondary system shall be reanalyzed in appropriate detail using the output motions from the first analysis as input at the points of connectivity.

(e) All combinations of the dominant secondary system modes and the dominant primary structure modes (considering the response forces and displacements at the interfaces) must be considered, and the most restrictive combination shall govern. The dominant frequency has a modal mass greater than 20% of the total system mass.

ASCE 4-16, Section 3.7.2, “Single Point Attachment,” provides the coupling criteria for secondary systems with a single-point attachment to the primary system. This section states the following:

> (a) To determine if coupled analyses are required owing to dynamic interaction, the criteria shown in Fig. 3-2 shall be used. The mass ratio in Fig. 3-2 is the modal mass ratio computed from Eq. (3-13), and the frequency ratio is the ratio of the dominant uncoupled modal frequencies of the secondary and primary systems.
>
> (b) For a secondary system dominant mode and the primary system mode *i*, the modal mass ratio can be estimated by
>
> $$\Lambda_i \approx \frac{M_s}{M_{pi}} \qquad (3\text{-}13)$$
>
> where
>
> M_{pi} = $(1/\varphi_{ci})^2$;
> φ_{ci} = mode vector value from the primary system's modal displacement at the location where the secondary system is connected, from the *i*th normalized modal vector, $\{\varphi_{pi}\}$, $\{\varphi_{pi}\}^T[M_p]\{\varphi_{pi}\}=1$;
> $[M_p]$ = mass matrix of the primary system; and
> M_s = total mass of the secondary system.

ASCE 4-16, Section 3.7.3, “Multipoint Attachment and Static Constraint,” provides the coupling criteria for a subsystem supported to the primary system with multiple attachment points. This section states the following:

> (a) The stiffness of a subsystem supported at two or more points may restrict movement of the primary system. In addition to mass and frequency ratio consideration, the relative stiffness of the subsystem to structure shall be investigated to determine when coupling is required. Coupling is required when the values of key design parameters from the coupled model are more than 10% higher than those from an uncoupled model.

(b) A coupled analysis of the primary-secondary system shall be performed if the static constraints cause significant load redistribution in the primary system.

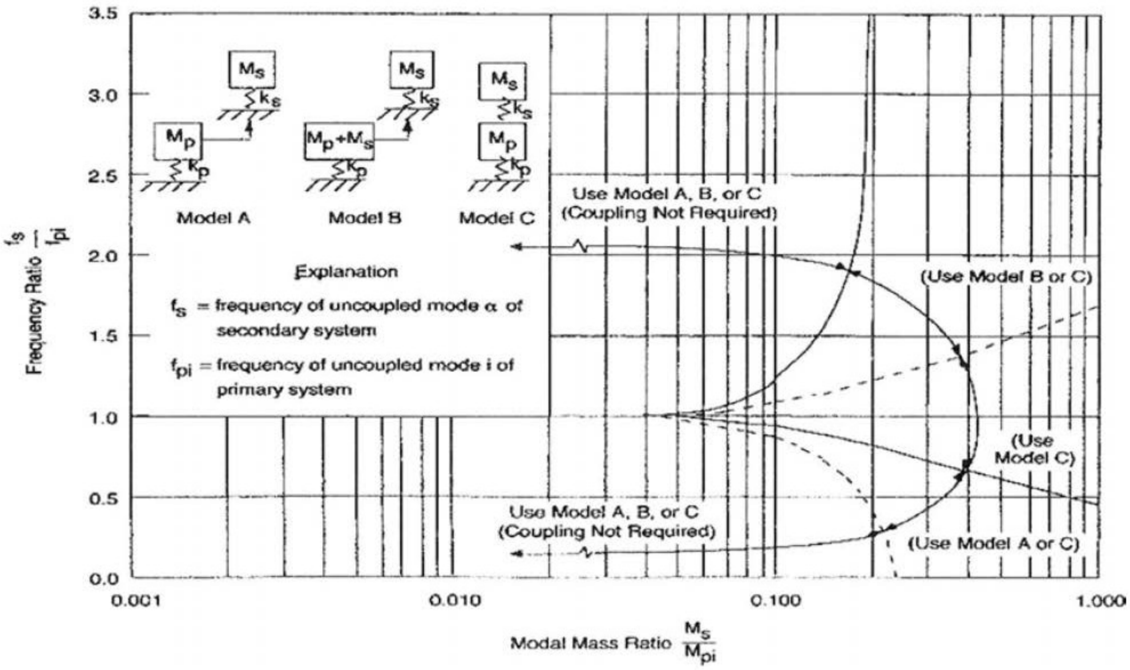

FIGURE 3-2. Decoupling Criteria for Secondary Systems with Single-Point Attachment to the Primary System

ASCE 4-16, Commentary Section C3.7, provides further information on the approach presented in Section 3.7 for coupling of single- point and multipoint attachments. Commentary Section C3.7 states the following:

C3.7.1 General Requirements

Coupled analysis generally alters the subsystem dynamic response and may also alter the primary system dynamic response. The purpose of the coupled analysis is to determine if a decoupled model sufficiently captures the response parameters of interest.

C3.7.2 Single-Point Attachment

The decoupling criteria in Fig. 3-2 are based on (1) a single primary system, (2) a single secondary system, (3) the secondary system consisting of a single degree of freedom system, (4) the primary system consisting of a single degree of freedom system, (5) the secondary system being connected to the primary system at a single point, (6) the evaluation of a single quantity of response (i.e., force in the spring or displacement of the mass etc., as opposed to multiple quantities of response such as three forces and three moments, etc.), and (7) the application of a single-dimensional earthquake as opposed to the three-dimensional earthquake. Therefore, the application of the criteria in Fig. 3-2 to practical situations such as multi-degrees-of-freedom secondary systems multiply connected to several multi-degrees-of-freedom primary systems requires judgment, caution, and additional considerations. Fig. 3-2 allows up to 10% error in the coupled modal frequency. The structural response error may be larger but is always on the conservative side (Hadjian and Ellison 1986).

The expression for the modal mass ratio in Eq. (3-13) assumes the mode of the secondary system is dominant and uses the total secondary system mass. The dominant modal masses of the primary system are used.

C3.7.3 Multipoint Attachment and Static Constraint

The stiffness of a subsystem supported at two or more points may alter the structural dynamic properties and thus constrain or amplify movement of the primary system. In addition to mass and frequency ratio consideration, the relative stiffness of the subsystem to primary structure shall be investigated to determine when coupling is required. For configurations with multipoint attachment of the secondary system, using the secondary system dominant mode and the primary system mode *i*, the modal mass ratio is defined by the following general equation (Gupta 1990):

$$r_{i\alpha} = ([\Gamma_{c\alpha}]\{\varphi_{ci}\})^2 \qquad \text{(C3-13)}$$

where

$\{\varphi_{ci}\}$ = subvector of the uncoupled primary system's i^{th} normalized modal vector, $\{\varphi_{pi}\}$, consisting of connecting degrees of freedom only, $\{\varphi_{pi}\}^T[M_p]\{\varphi_{pi}\} = 1$;
$[M_p]$ = mass matrix of the primary system;
$\Gamma_{c\alpha}$= row of secondary system participation factors, consisting of one term for each connecting degree of freedom= $\{\varphi_{s\alpha}\}^T[M_S]\{U_{SC}\}$;
$\{\varphi_{s\alpha}\}$= αth normalized modal vector of the secondary system, $\{\varphi_{s\alpha}\}^T[M_S]\{\varphi_{s\alpha}\} = 1$; and
$\{U_{SC}\}$ = secondary system influence matrix consisting of one influence vector for each connecting degree of freedom, c. The influence vector for a connecting degree of freedom is the displacement vector of the secondary system, when the particular degree of freedom undergoes a unit displacement.

This mass ratio, $r_{i\alpha}$ is the one to be used in Fig, 3-2. All combinations of modes of the primary and secondary systems must be considered and the most restrictive combination will govern.

Static constraint applied by the multipoint attachment of a secondary system on the primary system may increase the primary system modal frequency (Gupta 1990). The value of the uncoupled *i*th primary system modal frequency, ω_{pi} (rad/s) is increased to $(\omega_{pi}^2 + \Delta\omega_{pi}^2)^{0.5}$ in which

$$\Delta\omega_{pi}^2 = \{\varphi_{ci}\}^T[K_{CC}^s]\{\varphi_{ci}\} - \textstyle\sum_\alpha r_{i\alpha}\omega_{s\alpha}^2 \qquad \text{(C3-14)}$$

where
$[K_{CC}^S]$ = square matrix representing the stiff contribution of the secondary system to the

stiffness matrix of the coupled primary-secondary system for the connecting degrees of freedom; and

$\omega_{s\alpha}$ = circular frequency (rad/s) of the *i*th uncoupled secondary system mode.

The summation in Eq. (C3-13) is on all the significant secondary system modes. A coupled analysis of the primary-secondary system shall be performed when the ratio of the increased primary system frequency to the uncoupled frequency, $\left(1 + \frac{\Delta\omega_{pi}^2}{\omega_{pi}^2}\right)^{0.5}$ is greater than 1.1.

SRP Section 3.7.2 provides the criteria when coupling of a subsystem and primary system is needed. SRP Section 3.7.2II.3.B states the following:

<u>Decoupling Criteria for Subsystems.</u>

It can be shown, in general, that frequencies of systems and subsystems have a negligible effect on the error due to decoupling. It can be shown that the mass ratio, R_m, and the frequency ratio, R_f, govern the results where R_m and R_f are defined as:

R_m = Total mass of supported subsystem/Total mass of the supporting system

R_f = Fundamental frequency of the supported subsystem/Dominant frequency of the support system

The following criteria are acceptable:

i. If $R_m < 0.01$, decoupling can be done for any R_f.

ii. If $0.01 < R_m < 0.1$, decoupling can be done if $0.8 > R_f > 1.25$.

iii. If $R_m > 0.1$, a subsystem model should be included in the primary system model.

If the subsystem is rigid compared to the supporting system and also is rigidly connected to the supporting system, it is sufficient to include only the mass of the subsystem at the support point in the primary system model. On the other hand, in the case of a subsystem supported by very flexible connections, e.g., pipe supported by hangers, the subsystem need not be included in the primary model. In most cases, the equipment and components, which come under the definition of subsystems, are analyzed (or tested) as a decoupled system from the primary structure and the seismic input for the former is obtained by the analysis of the latter. One important exception to this procedure is the reactor coolant system, which is considered a subsystem but is usually analyzed using a coupled model of the reactor coolant system and primary structure.

The above information shows that implementing the provisions of ASCE 4-16, Section 3.7, may involve a substantial calculational effort to satisfy decoupling criteria for multi-degree-of-freedom (MDOF) mathematical models typical of those developed for seismic/structural analysis of nuclear power plant SSCs. In many cases, supported components may be amenable to modeling with a SDOF but are attached to supporting MDOF structure or system models. Therefore, the decoupling criteria in ASCE 4-16, Section 3.7.2(a), Figure 3-2, applicable to an SDOF supporting element and SDOF supported element, must be implemented with caution. Commentary Section C3.7.2 states, “Therefore, the application of the criteria in Fig. 3-2 to practical situations such as multi-degrees-of-freedom secondary systems multiply connected to several multi-degrees-of-freedom primary systems requires judgment, caution, and additional considerations.” Consequently, implementations of the criteria in Figure 3-2 for MDOF supporting elements and either MDOF or SDOF supported elements should be peer-reviewed for technical adequacy.

Commentary Section C3.7.2 also states, “Fig. 3-2 allows up to 10% error in the coupled modal frequency. The structural response error may be larger but is always on the conservative side.” This is apparently more generous than the current SRP guidance. Applying SRP criteria to Figure 3-2, a vertical line would be drawn at a mass ratio of 0.10; above 0.10, coupling is required regardless of the frequency ratio.

Decoupling is intended to simplify the analysis process without compromising the validity of the results; it can be implemented if it is demonstrated to produce results close to, or conservative relative to, the coupled response. BNL considers a difference of up to 10 percent between coupled response and uncoupled response to be acceptable if it is on the conservative side. Excessive conservatism may indicate improper modeling. BNL further recommends the use of a coupled model if the uncoupled response is unconservative by more than 5 percent relative to the coupled response. This ensures that decoupling will not contribute significantly to underprediction of the seismic response.

The straightforward decoupling criteria in SRP Section 3.7.2 have been in use for many years and are relatively easy to implement. They should be considered as an alternative to those in ASCE 4-16, Section 3.7.

Combination of Multiple Response Parameters

ASCE 4-16, Section 4.2.2, “Combination of Spatial Components,” states the following:

> (c) If linear response-history analyses are performed separately for each component of ground motion, the combined response for all three spatial components shall be obtained by one of the following:
>
> 1. Use the square-root-of-the-sum-of-squares (SRSS) rule to combine the maximum responses from each earthquake component.
>
> 2. Algebraically combine the individual component responses at each time step to obtain the combined response history. The maximum combined response shall be recorded for design.
>
> 3. Use the 100-40-40 rule to combine the maximum responses from

each component of seismic input. The responses are combined directly, using the assumption that when the maximum response from one component of seismic input occurs, the responses from the other two components of input are 40% of the maximum. In this method, all possible combinations of the responses to the three components of seismic input shall be evaluated and the most critical response used. In the following equations, R represents the total response of the parameter of interest (in a fixed direction), and R_i represents the contribution to the response parameter of interest caused by the ith component of seismic input.

$$R = \pm[|R_1| + 0.4|R_2| + 0.4|R_3|]$$

or

$$R = \pm[0.4|R_1| + |R_2| + 0.4|R_3|] \qquad (4\text{-}1)$$

or

$$R = \pm[0.4|R_1| + 0.4|R_2| + |R_3|]$$

This rule is for combining response quantities of similar type and generated along the same direction due to different components of earthquake motion.

ASCE 4-16, Section 4.3.3, “Combination of Spatial Components,” states the following:

Modal analysis will generally be performed in two orthogonal horizontal and one vertical direction. Component responses will be computed for each axis of excitation. These component responses shall be spatially combined by either the SRSS method or the 100-40-40 rule.

ASCE 4-16, Section 4.3.4, “Combination of Multiple Response Parameters,” states the following:

(a) When more than one response parameter exists, such as column axial force and moment, in the design calculation, the combined value of each response shall be calculated by SRSS or 100-40-40, including the effects of rigid body response. In the subsequent design calculations, all possible combinations of these values shall be considered. For M response parameters of interest, 2^M sets of response combinations exist to be considered.

(b) Alternatively, simultaneous variation in the responses of interest, R^r, $r = 1$ to M, given by the following equation may be used:

$$\sum_r \sum_S H^{rS} R^r R^S = 1 \qquad (4\text{-}3)$$

where

H^{rS} = two-dimensional array, $(M \times M)_s$, which is the inverse of another array, G^{rS}, given by

$$G^{rS} = \sum_I \sum_i \sum_j \varepsilon_{ij} R^r_{Ii} R^S_{Ij} \qquad (4\text{-}4)$$

where

R^r_{Ii}, R^S_{Ij} = value of the response, $R^r(R^S)$, in the ith, jth mode of vibration under Ith earthquake component.

A sufficient number of R^r values satisfying Eq. (4-3) shall be considered in design, so that all the possibilities reflected by the equation are included.

ASCE 4-16, Commentary Section C4.2.2, “Combination of Spatial Components,” states the following:

> The analysis of recorded earthquake ground motions indicates that the two horizontal components and the vertical components are substantially independent (Hadjian 1981; Huang et al. 2011). Accordingly, Section 2.6.2 requires that the orthogonal components of the input motions be statistically independent.
>
> For linear response-history analysis, the three components of motion may be input to the mathematical model simultaneously or separately. The mathematical model should include the soil surrounding the structure if the effect of SSI is likely significant.
>
> The 100-40-40 rule proposed by Newmark (Newmark 1975; Newmark and Hall 1978) was based on the observation that the maximum increase in the resultant for two orthogonal forces occurs when these forces are equal. The maximum value is 1.4 times one component. As a consequence, this rule is an acceptable alternative to the SRSS rule and is a reasonable procedure to use given the basic uncertainties involved.

ASCE 4-16, Commentary Section C4.3.3, “Combination of Spatial Components,” states the following:

Studies have shown that the two horizontal and one vertical component of actual recorded earthquake ground motions are substantially independent in a statistical sense (Hadjian 1981; Huang et al. 2011). Consequently, only a small probability exists that the peak response in a structural member due to each of the three components will occur at the same time. Two methods of combining spatial components of ground motion are recommended: (1) the SRSS method per Clough and Penzien (2003) and Chopra (2012) and (2) the 100-40-40 method as proposed by Newmark and Hall (1978).

When a single design parameter is dominant, both methods will give similar results. When multiple design parameters are involved, the SRSS method is generally more conservative. In applying the 100-40-40 method, the design values for multiple parameters should be calculated using consistent component factors.

In spatial combinations, the goal is to use realistic multiple responses that are representative of maximum values that could occur simultaneously. This principle is implemented by considering the maximum value of each design parameter together with the values of the other parameters that correspond to the same directional combination. By permutation, each maximum design parameter is considered in design with the values of the other parameters corresponding to the same spatial combination.

The appropriate use of the 100-40-40 method is illustrated in the following example:

Consider a shear wall oriented in the north–south direction with seismic responses for four design parameters: P = axial load, V = shear, M_{ip} = in-plane moment, and M_{op} = out-of-plane moment. Assume that responses for these parameters have been obtained from dynamic analysis as shown in Table C4-1. (Note: for simplicity the signs are ignored in this example.) The first three rows in the table show the calculated responses due to seismic input in each direction. The fourth row is the SRSS combination. The next three rows give the design values using the 100-40-40 method. All three rows obtained from the 100-40-40 method would be used in design as individual seismic load combinations.

As can be seen from the table, the most severe design condition is produced by the SRSS method. This is the case in which each response parameter is dominated by a particular earthquake direction. However, the design resulting from each of the three 100-40-40 method combinations will be less demanding, and the final design will be more realistic as each of these spatial response combinations are likely to occur, but at different points in time.

Using the maximums of each parameter from the three factored combinations yields results similar to SRSS but negates any benefit of 100-40-40 and is inconsistent with the goal of determining seismic design parameters that are most likely to occur simultaneously.

Table C4-1. Application of the SRSS and 100-40-40 Methods

Seismic Load	P, kip	V, kip	M_{ip}, kip-ft	M_{op}, kip-ft
N-S earthquake	0	500	10,000	100
E-W earthquake	0	30	500	500
Vert. earthquake	400	0	0	0
SRSS	400	501	10,012	510
Factored 1: 100 + 40 + 40	160	**512**	**10,200**	300
Factored 2: 40 + 100 + 40	160	230	4,500	**540**
Factored 3: 40 + 40 + 100	**400**	212	4,200	240

,

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ASCE 4-16, Section 4.3.4(a), addresses implementation of the 100-40-40 method in design when more than one response parameter exists (e.g., column axial force and moment). It is noted that the Commentary to Section 4.3.3 (not Section 4.3.4) presents an example that appears to be inconsistent with the criteria in Section 4.3.4(a). In the past, the criteria in Section 4.3.4(a) have been subject to differing interpretations by BNL and by industry (see, for example, “On the Correct Application of the 100-40-40 Rule for Combining Responses Due to Three Directions of Earthquake Loading,” issued 2010).

The method presented in ASCE 4-16, Section 4.3.4(b), has been carried over from ASCE 4-98. BNL is unaware of any actual implementation of this method for multiple response parameters and has not evaluated its acceptability. Any implementations of it should be peer-reviewed for technical adequacy.

To provide a reference point for its recommendation, BNL has evaluated the textbook, but feasible, case of an axis-vertical, square hollow tube beam/column subject to identical north-south and east-west horizontal inputs oriented in the beam major axis directions, as well as an arbitrary vertical input V. Linear elastic theory permits each input to be analyzed separately for the undeformed shape of the beam.

The design of the beam involves five quantities of interest: axial force, transverse shear forces in the north-south and east-west directions, and moments about the north-south and east-west axes. In this case, north-south input produces moment M_{ns} and transverse shear force S_{ns}, east-west input produces moment M_{ew} and transverse shear force S_{ew}, and vertical input V produces axial force P. (Note that $M_{ns} = M_{ew}$, $S_{ns} = S_{ew}$ in this example.)

SRSS combination of the three directions produces the following load set for design of the beam:

$$P, M_{ns}, M_{ew}, S_{ns}, S_{ew}.$$

This is equivalent to the absolute-sum (ABSUM) combination of the three directions, which implies all response quantities reach their peak value at the same instant in time.

In contrast, 100-40-40 produces the following three load sets for design of the beam:

$$P,\ 0.4M_{ns},\ 0.4M_{ew},\ 0.4S_{ns},\ 0.4S_{ew};$$
$$0.4P,\ M_{ns},\ 0.4M_{ew},\ S_{ns},\ 0.4S_{ew};$$
$$0.4P,\ 0.4M_{ns},\ M_{ew},\ 0.4S_{ns},\ S_{ew}.$$

These load sets reflect the expectation that the response quantities do not all reach their peak value at the same instant in time.

The over conservatism of the SRSS spatial combination method in this example supports a conclusion that the 100-40-40 spatial combination method provides a more realistic design basis. Therefore, BNL considers the implementation of 100-40-40 in accordance with the sample problem in Commentary Section C4.3.3, Table C4-1 (and the above example), to be acceptable. This applies to time history, response spectrum, and equivalent static analyses.

As further demonstration, a comparison is made between SRSS and 100-40-40 if the single parameter of axial stress is combined for the three directions of loading. Assume S_b is the maximum extreme fiber axial stress produced by north-south loading; S_b is also the maximum extreme fiber axial stress produced by east-west loading. These stress values coincide at the four corners of the square hollow section. S_v is assumed to be the uniform axial stress in the cross section produced by vertical loading.

Using SSRS, the expected maximum axial stress at the four corners is $(S_b^2 + S_b^2 + S_v^2)^{1/2}$.

For $S_v = 0$,	SRSS gives $1.414 \times S_b$;	100-40-40 gives $1.4 \times S_b$.
For $S_v = S_b$,	SRSS gives $1.732 \times S_b$;	100-40-40 gives $1.8 \times S_b$.
For $S_v = 1.414 \times S_b$,	SRSS gives $2 \times S_b$;	100-40-40 gives $2.214 \times S_b$.

As expected, for a single parameter, the 100-40-40 method is essentially equal to or conservative compared to SRSS. However, the multiparameter SRSS combination, which reduces to ABSUM, produces very conservative predictions for combined axial stress. In contrast, the multiparameter 100-40-40 combination produces the same prediction for combined axial stress as the 100-40-40 single-parameter case.

These results support the conclusion that the 100-40-40 combination method is more realistic when there are multiple response parameters; BNL thus recommends 100-40-40 over SRSS. However, SRSS is still acceptable, because it produces more conservative results when there are multiple response parameters.

A.2 ASCE 43-18

Seismic Design Criteria (ASCE 43-18, Section 1.1, "Seismic Design Criteria")

The seismic design criteria in ASCE 43-18 are based on two key assumptions:

(1) ASCE 4-16 predicts seismic demands for DBE at the 80th-percentile nonexceedance probability.

(2) Material standards such as American Concrete Institute (ACI) 349-13, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," and the American Institute of Steel Construction (AISC) N690, "Specification for Safety-Related Steel Structures for Nuclear Facilities," deliver capacities at the 98th-percentile exceedance probability (it is customary to refer to this value as the 2-percent exclusion limit, a term that will be used subsequently in this review).

These assumptions did not appear in ASCE 43-05.[1] Used together, they are intended to ensure (1) a 1-percent (or smaller) annual frequency of unacceptable performance, conditioned on DBE shaking, and (2) a 10-percent annual frequency of unacceptable performance, conditioned on 150 percent of DBE shaking. The use of inelastic energy absorption factors in demonstrating compliance is permitted. These two objectives are comparable if the overall uncertainty, β, defined as the SRSS of the log-standard deviations in demand, β_R, and capacity, β_C, is approximately 0.39. For larger β, the objective (1) controls, while for smaller β, (2) controls.

Section 2.1 of this report discusses the adequacy of the first assumption, describing the review of ASCE 4-16. The current section of the appendix discusses the adequacy of the second assumption.

The assertion that material standards deliver capacities at the 2-percent exclusion limit has its roots in the seismic margins studies of the 1990s, where it appears in a number of conference papers and reports. ASCE 4-16, Section 1A.2.2.1, "Median Strength Conservatism Ratio," states the following:

> According to a review of median capacities from past seismic probabilistic risk assessment studies versus U.S. code-specified ultimate strengths for several failure modes, the determination is that for ductile failure modes when the conservatism of material strengths, code strength equations, and seismic strain-rate effects are considered, at least a 98% probability exists that the actual strength will exceed the code strength. For low ductility failure modes, an additional factor of conservatism of about 1.33 is typically introduced.

Commentary Sections C1.3.1.2 and C1.3.1.3 of ASCE 43-18 provide a series of calculations aimed at showing that the two assumptions enable the two performance requirements to be

[1] Commentary Sections C1.3.1.2.1 and C1.3.1.2.2 of ASCE 43-05 assert that code ultimate strengths have at least a 98-percent probability of exceedance, while ASCE 4-16 is aimed at achieving about a 10-percent probability that the actual seismic response will exceed the computed response, given the occurrence of the DBE divided by 1.22. In ASCE standards, the commentary is considered nonmandatory. The net results in ASCE 43-05 and ASCE 43-18 are within a few percentage points of one another, depending on the logarithmic standard deviation in seismic demand.

met. As part of this demonstration, the same assumptions are made in Commentary Section C1.3.1.2.2.

The construction provided by these standards "should ensure seismic ruggedness with a high degree of reliability" (Commentary Section C1.1). Although the second assumption appears to be reasonable in most cases, it is nearly three decades old; design and construction practices have changed since its introduction. Furthermore, it is not supported by any peer-reviewed references.

The table below summarizes some common structural components and limit states and associated probabilities, $P[R < \varphi R_n]$, under the assumption that the strengths can be described by log-normal distributions. The cases considered represent ordinary building construction. The values in this table are based on the strengths calculated in ACI 318-14, "Building Code Requirements for Structural Concrete and Commentary," ACI 349-13, and AISC N690-18; they have not been adjusted for ductility.

	Strength limit state	Mean	COV[2]	Resistance factor, φ	$P[R < \varphi R_n]$
1	RC[3], one-way slab in flexure	$1.12\ M_n$	0.14	0.90	0.059
2	RC, shear wall	$1.20\ V_n$	0.18	0.75	0.005
3	RC short column, compression failure	$0.98\ R_n$	0.14	0.70	0.008
4	RC short column, tension failure	$1.05\ R_n$	0.12	0.90	0.100
5	Simple steel beam, flexure	$1.05\ M_n$	0.11	0.90	0.081
6	Continuous steel beam	$1.08\ M_n$	0.11	0.90	0.049
7	Steel brace, tension yielding	$1.05\ P_n$	0.08	0.90	0.027
8	Steel brace, compression instability	$1.05P_n$	0.13	0.90	0.117

Reinforced concrete components perform differently from steel components in part because ACI 318-14 and ACI 349-13 concrete mix design procedures are targeted at achieving approximately a 10-percent exclusion limit concrete compression strength. In contrast, the American Society for Testing Materials (ASTM) standards governing strength of common construction-grade steels (e.g., ASTM A36, ASTM A572, ASTM A992) and fasteners (e.g., ASTM A325, ASTM A449, ASTM A490) are not based on a target exclusion limit.

Many of the limit state probabilities in column (5) ($P[R < \varphi R_n)$ may be lower for nuclear power plant structures for three reasons:

(1) Quality assurance in construction of nuclear power plant structures typically is higher than in ordinary building construction.

[2] COV- Coefficient of Variation
[3] RC- Reinforced Concrete

- Consider, for example, tension yielding of a steel brace. If the mean is $1.06P_n$ rather than $1.05P_n$, the probability decreases from 0.027 to 0.02; if the coefficient of variation is 0.07 rather than 0.08, the probability is less than 0.02.

(2) The probabilities in column (5) ($P[R < \varphi R_n)$ do not reflect the effect of ductility, which may significantly increase the capacity for deformation-controlled structural actions such as those due to earthquakes. ASCE 43-18, Section 5.1.2, "Seismic Load Combinations," permits the use of an inelastic energy absorption factor F_μ for strength-based acceptance criteria (5.1.2.1) and the use of nonlinear seismic analysis for deformation-based acceptance criteria (5.1.2.2). The inelastic energy absorption factor comprises several components; of interest here are the components $F_{\mu C}$ (5.1.3.1), listed in Table 5-1, which are used to reduce the seismic demand. These reduction factors are stipulated for Limit States A, B and C; for Limit State D, $F_{\mu C} = 1.0$. The factors are intended for use in design and are conservative.

- A properly designed and detailed one-way reinforced concrete beam or slab or a compact laterally supported steel beam will typically sustain deformations that are at least four times the elastic limit deformations. If post-yield strength or deformation for the flexural members in rows (1), (4), (5), or (6) is within 10 percent of the elastic limit, the probabilities will decrease to less than 0.02.

- The steel brace in row (8) would have to sustain a load 12 percent in excess of the buckling load; the presence of lateral bracing would determine whether this increase is plausible.

(3) The strengths are reported for static rates of load, which are lower than strengths for dynamic loads.

- If the reduction due to static rate of load in row (3) were disregarded, the mean strength would be approximately $1.02P_n$, and the probability would decrease to 0.004.

Based on the analysis above, the assumption that current material codes and specifications deliver capacities at the 2-percent (or less) exclusion limit appears to be reasonable, for the most part; Section 1.1 is therefore judged to be acceptable. However, it is strongly recommended that a study be performed to provide further data to support this judgment. BLM also recommends that, during the design phase, a peer review be performed for the 2-percent exclusion limit on design strength for the SSCs and the corresponding codes and standards being used. Section 10.1 of ASCE 43-18 provides guidance for performing an independent peer review.

Damping Values (ASCE 43-18, Section 3.3.3, "Damping Values for SSCs")

Table 3-1, "Specified Damping Values for Dynamic Analysis," of ASCE 43-18 presents damping values for SSCs. Section 2.1.3 of this report summarizes BNL's evaluation of these damping values, except those for steel-plate concrete (SC) structures. The evaluation of damping values for SC walls is described below.

For SC walls, Table 3-1 of ASCE 43-18 indicates that damping values should be 3 percent for

Response Level 1, 5 percent for Response Level 2, and 10 percent for Response Level 3. The paper "1/10th Scale Model Test of Inner Concrete Structure Composed of Concrete Filled Steel Bearing Wall" (H. Akiyama et al., *Transactions of SMiRT-10*, August 22–27, 1989) reports on a test performed on a scaled specimen of containment internal structures. These consisted of the inner concrete structure of a pressurized-water reactor nuclear power plant containing SC wall members to represent the primary shield wall, which supports and surrounds the reactor vessel, and a secondary shield wall, which supports and surrounds the steam generators, a pressurizer, and a fuel transfer canal. On damping, the paper states the following:

> Fig. 6 shows the equivalent viscous damping factor obtained from the loop of the load to relative rotation angle. In the tested SC structure, the equivalent viscous damping factor was about five percent before the steel yielded while it increased dramatically after the steel yielded. It is supposed that this is because the viscous damping governed the damping mechanism before the steel yielded while the hysteresis damping did so after the steel yielded.

Another reference on damping for SC members is a paper titled "Experimental Behavior of Flexural-Critical Steel-Plate Composite Structural Walls" (S. Epackachi et al., *Transactions of SMiRT-23*, August 10–14, 2015). Here, four large-scale specimens were tested under displacement-controlled cyclic loading. The paper concluded, "The equivalent viscous damping for flexure-critical SC walls can be assumed to be 5% at displacements less than that at peak strength and 10% for greater displacements."

Based on the above discussion, for Response Level 2 (SSE level), the use of 5-percent damping indicated in Table 3-1 of ASCE 43-18 is acceptable.

For Response Level 3, in addition to the above papers, which suggest that 10-percent damping is acceptable, another paper was identified: "Investigation of Damping Ratio of Steel Plate Concrete (SC) Shear Wall by Lateral Loading Test & Impact Test" (S.G. Cho et al., *Journal of the Earthquake Engineering Society of Korea*, 17(2):79–88, 2013). Although most of the paper is in Korean, BNL identified useful information from the abstract, figures, and a table. The abstract states, "The experimental results show that the damping ratios increased from about 6% to about 20% by increasing the load from the safe shutdown earthquake level to the ultimate strength level." In this case, for SSE (Response Level 2), the paper identifies the damping value of 6 percent. For levels beyond Response Level 2, Table 6 and Figure 18 of the paper show that for all three specimens under cycle #3 (when peak loading was reached), the damping values ranged from 13.3 percent to 14.5 percent. In later cycles, the damping values rose higher, to about 20 percent in cycle #5.

It has been generally recognized that SC members behave comparably to, and often better than, reinforced concrete members. Even though the reinforced concrete damping values for Response Levels 2 and 3 are 7 percent and 10 percent, respectively, BNL judges the corresponding damping values of 5 percent and 10 percent for SC members to be reasonable, on the basis of the above references. Therefore, the damping value of 10 percent given in Table 3-1 for SC members at Response Level 3 is also judged to be acceptable.

Editorial and Typographical Errors

<u>Section 1.1, "Seismic Design Criteria"</u>

The last paragraph states, "Requirements for Quality Assurance (QA) and independent peer review are described in Chapter 9." However, Section 9.4, "Peer Review," discusses only peer review of the isolation system and the related test programs, whereas Chapter 10, "Quality Assurance Provisions," Section 10.1.2, describes an independent seismic peer review for the entire seismic analysis/design effort. The reference should be to Chapter 10, not Chapter 9.

Commentary Section C101, "Design Verification and Independent Peer Review"

The number of this section should be corrected to be C10.1.

Section 3.3, "Modeling Input Parameters"

Section 3.3.1 is titled "Effective Stiffness of Reinforced Concrete and Steel-Plate Composite (SC) Members." This section states, "Table 3-2 in ASCE 4-14 enables calculation of effective stiffness of reinforced concrete." The reference should be to ASCE 4-16. The same error occurs in Section 3.3.3 and Commentary Section C3.3.3.

Furthermore, Section 3.3.1 states, "ANSI/AISC N690-12 provides demand for steel-plate composite (SC) and concrete elements." Since the topic of this section is the stiffness of members, the statement should read, "...provides the stiffness for steel-plate composite...." It should also be noted that ANSI/AISC N690 does not apply to SC members in general or to concrete elements, but only to SC walls (which do contain infill concrete between the steel faceplates). It would be more appropriate to use the phrase "stiffness for steel-plate composite walls."

Section 4.2.2.1, "Shear Strength of Reinforced Concrete Capacities"

This section states, "The shear strength provided by concrete for non-prestressed members, Vc, calculated in accordance with Section 11-2 of ACI 349-13, shall be multiplied by a size correction factor κ." There appears to be a typo; the reference should be to Section 11.2 (not 11-2) of ACI 349-13.

Commentary Section C4.2.2.1, "Shear Strength of Reinforced Concrete Capacities"

Commentary Section C4.2.2.1 states, "A detailed evaluation of the shear strength of reinforced concrete beams constructed without shear reinforcement, calculated in accordance with Equation 11-3 of ACI 349-13 shows clearly that...." There is no Equation 11-3 in ACI 349-13, but there is an Equation 11-3 in ACI 318-11, to which Section 11.2 of ACI 349- 13 refers.

Section 5.1, "Load Combinations"

Section 5.1.2.1 defines D_{NS} as follows:

> Non-seismic demand acting on an element. Non-seismic demand shall include the mean effects of dead, live, equipment, fluid, snow, and lateral soil loads. The non-seismic demands shall be consistent with the mass defined in Section 3.4.2 of this Standard and the mass defined in ASCE 4.

The first sentence is essentially the same as that appearing in ASCE 43-05, which was reviewed previously and is acceptable. In the last sentence, it appears that the reference to Section 3.4.2 should be to Section 3.3.2, and the rest of the sentence should read, "…and the mass defined in Section 3.4.2 of ASCE 4."

Appendix A, "Alternate Method to Meet ASCE Standard 43 Performance Goals When Seismic

Capabilities Are Defined at the 50% Probability of Failure Level"

The next-to-last sentence in Section A3 indicates that A_R is defined in Section 2.2.1 of ASCE 43, which does not exist. It appears that this should refer to Section 2.2.

The first sentence in Commentary Section CA3 refers to Commentary Section C2.2.1.3, which does not exist. It appears that this reference should be to Commentary Section C2.2.

Appendix B, "Alternate Method to Meet ASCE Standard 43 Performance Goals When Seismic Capabilities Are Defined at the 10% Probability of Failure Level"

The last sentence of Section B2 refers to Section C2.2.1, which does not exist. It appears that this should refer to Section 2.2.

The sentence following Equation CB3-1 refers to Commentary Section C2.2.1.3, which does not exist. It appears that this should refer to Commentary Section C2.2.

Equation CB3-5 does not define the symbol CH. It appears that this symbol should be C_{HP}; otherwise Equation CB3-7 does not follow in the derivation.