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Evaluation of the Seismic Design Criteria in ASCE/SEI Standard 43-05 for Application to Nuclear Power Plants

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ABSTRACT

This report describes the results of the review and evaluation of ASCE/SEI Standard 43-05, Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, to determine the applicability of this standard to the seismic design of nuclear power plants (NPPs). This effort included the review of this Standard, references cited in the Standard and other supporting documents. As a result of this review, technical and regulatory issues that might need to be addressed are identified and a comparison is made between the criteria presented in the Standard and the criteria provided in NRC regulatory guidance documents.

Throughout this report, observations are made where the provisions of the Standard do not appear to be consistent with current NRC regulatory guidance documents. These observations are followed by recommendations for alternative approaches, further justifications, or the need to adhere to the currently accepted methods in NRC regulatory guidance documents. In some cases, recommendations are made where further detailed review would help resolve the identified question or concern.

The overall conclusion from this review effort is that with properly stipulated Performance Goals and supporting criteria, the approach presented in ASCE/SEI Standard 43-05 can provide acceptable levels of protection against severe low-probability earthquakes. However, some limitations, as defined in this report, are needed to achieve the goals of public safety and environmental protection.

FOREWORD

This report presents the results of a review of the American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities." As its title implies, this standard provides seismic design criteria for safety-related structures, systems, and components (SSCs) in a broad spectrum of nuclear facilities.

This review of ASCE/SEI 43-05 was undertaken by Brookhaven National Laboratory (BNL) at the request of the U.S. Nuclear Regulatory Commission (NRC), Office of Nuclear Regulatory Research. In sponsoring this review, the NRC's primary purpose was to assess the use of the standard's risk-informed and performance-based approach to determine seismic ground motion for use in designing commercial nuclear power plants (NPPs). In addition, the NRC's secondary interest was to evaluate the design criteria for safety-significant SSCs.

As part of this study, BNL staff reviewed references cited in ASCE/SEI 43-05, as well as other supporting documents, to determine the conditions under which it would be appropriate to use the standard in the seismic design of SSCs at NPPs. In so doing, BNL identified and evaluated the strengths and weaknesses of the standard, and identified technical and regulatory issues that may need to be addressed if ASCE/SEI 43-05 were to be used. In addition, BNL compared the seismic design criteria presented in ASCE/SEI 43-05 against those provided in the NRC's current regulatory guidance documents, such as regulatory guides, generic communications, staff positions, and the "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants" (NUREG-0800).

The main conclusion of this report is that the standard's performance-based approach can provide acceptable levels of protection against low-probability earthquakes, given proper stipulation of performance goals and supporting criteria. As a result, the NRC staff used insights gained from this review in developing Regulatory Guide 1.208, "A Performance-Based Approach To Define the Site-Specific Earthquake Ground Motion," which the staff issued for public comment as Draft Regulatory Guide DG-1146 on October 30, 2006. This new regulatory guide will provide guidance to applicants for early site permits (ESPs) and combined operating licenses (COLs). Insights gained from this evaluation also assisted the NRC staff in developing the technical bases for implementing the performance-based approach, and will inform staff efforts in updating other regulatory guidance documents, such as NUREG-0800. In addition, a number of findings were related to structural design and seismic qualification of equipment. While these findings are not directly related to Regulatory Guide 1.208, they were informative for related staff efforts to update the relevant sections of the NUREG-0800.

Brian W. Sheron, Director
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EXECUTIVE SUMMARY

The American Society of Civil Engineers recently published a new standard, ASCE/SEI Standard 43-05, Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities. This Standard provides seismic design criteria for safety-related structures, systems and components (SSCs) in a broad spectrum of nuclear facilities. In view of the current trend to utilize risk-informed and performance-based methods in various codes and standards in other industries and the interest expressed by the nuclear power industry in applying these methods to nuclear power plants (NPPs), a review of the new ASCE/SEI Standard 43-05 for potential application to NPPs is warranted.

This report presents the results of a review and evaluation of ASCE/SEI Standard 43-05 with regard to its potential application to the design of NPPs. As part of the review of the Standard, references cited in the Standard, and other supporting documents, were also reviewed to determine the conditions under which it would be appropriate to use this Standard in the seismic design of SSCs at NPPs. This effort identified technical and regulatory issues that might need to be addressed if the Standard were to be used. The strengths and weaknesses of the Standard are described from the perspective of the reviewers. A comparison also was made between the criteria presented in the Standard and the criteria provided in the NRC regulatory guidance documents such as the NRC Standard Review Plan (NUREG-0800), Regulatory Guides, Generic Communications, and current staff positions.

This report is organized to provide the results of the review following the same general topics presented in ASCE/SEI Standard 43-05. In each section, observations are made where the provisions of the Standard do not appear to be consistent with NRC regulatory guidance documents. In addition, recommendations for alternative approaches, further justifications, or the need to adhere to the currently accepted methods in NRC regulatory guidance documents are provided.

Section 2 of this report summarizes and evaluates the seismic design basis and the use of other codes and standards with ASCE/SEI Standard 43-05. The evaluation of the seismic design basis includes the definition of the Seismic Design Category and Limit State which then define the appropriate seismic design criteria.

Section 3 reviews the development of the design earthquake ground motion. This includes the overview of current regulatory practice, performance-based approach for developing design ground motions, use of a quantitative performance target rather than an acceptable seismic hazard level, development of the safe shutdown earthquake (SSE) from the performance goal, and factors of safety to achieve the target performance goals. Also evaluated are whether the use of ASCE/SEI Standard 43-05 will achieve the seismic margin requirement in SRM/SECY-93-087, the definition of SSE at varying depths, and the criteria for computing synthetic time histories.

Section 4 reviews the seismic demand, structural capacity, acceptance criteria and special requirements such as ductile detailing, rocking and sliding, and seismic separation. The evaluation of the seismic demand includes the appropriate use of industry codes and standards, various seismic analysis methods, and modeling/input parameters. The evaluation of structural capacities discusses the use of the strength design approach or the allowable stress design approach based on existing industry codes and standards. The evaluation of the acceptance criteria reviews the requirements specified for load combinations and acceptance criteria for strength and deformation.

Section 5 reviews the requirements for qualification of equipment and distribution systems (e.g., cable trays, conduit, and piping). The methods reviewed for seismic qualification consist of equivalent static and dynamic analysis methods, testing, past earthquake experience, and generic test data.

The conclusions and recommendations reached from the review and evaluation of ASCE/SEI Standard 43-05 are summarized in Section 6. The general conclusion reached is that, subject to the limitations identified in this report, there exists a technical engineering basis for NRC to endorse portions of the Standard for application to NPP design and construction. The major findings and recommendations are as follows:

- The Seismic Design Basis for nuclear power plant design and construction should be stipulated as SDB-5D, with a Target Performance Goal (limit state probability) of 10^{-5}/yr.

- The Target Performance Goal shall be met by ensuring that there is less than 1% mean probability of unacceptable performance for the SSE ground motion.

- The SRM/SECY 93-087 requirement - that the HCLPF shall be greater than or equal to 1.67 times the SSE in a margin assessment of seismic events - should be uncoupled from the seismic design criteria in ASCE/SEI Standard 43-05. However, SRM/SECY 93-087 may be applied, as circumstances warrant, as an independent verification that safety goals related to core damage are met.

- The specific dates or editions of codes and standards referenced in Section 1.2 of ASCE/SEI Standard 43-05 should be identified to allow confirmation that these codes and standards have been endorsed by the NRC for the seismic design of NPPs.

- The NRC should update its regulatory guidance documents to reflect the current generation of codes and standards and to develop one source of regulatory guidance that identifies the endorsement and any special regulatory positions that may apply to codes and standards.

- The use of the alternative methods in Section 4.2.2 of the Standard, in lieu of ACI Standard 349 for the design of low-rise shear walls, should not be endorsed at this time for design purposes.

- When performing qualification by analysis of equipment and distribution systems, the capacity for equipment and distribution systems should be determined using the current accepted methods defined by NRC SRP, RGs, and technical positions, rather than the provisions of Section 8.2.3 of the Standard.

- The NRC should complete the revision of RG 1.100 to present the regulatory positions on the use of IEEE Standard 344-2004, including the use of test experience data and earthquake experience data.

- The NRC should develop a technical position on the use of ASCE Standard 4, because this standard is integrated with ASCE/SEI Standard 43-05 to a considerable degree.

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Thanks are also given to the NRC Seismic Issues Technical Advisory Group (SITAG) for their technical review of this report and the suggestions and comments which greatly enhanced the technical content and results presented in this report.

1 INTRODUCTION

1.1 Background

ASCE/SEI Standard 43-05, entitled Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, (hereinafter referred to as the Standard or ASCE 43-05 in this report) has been recently published by the American Society of Civil Engineers. This Standard utilizes performance-based and risk-consistent methods in defining seismic design criteria for safety-related structures, systems and components (SSCs) in nuclear facilities. The goal of the Standard is to provide seismic design criteria to make SSCs sufficiently robust against earthquake effects so that the accidental release of radioactive materials is precluded. This goal is achieved by requiring that the nuclear facilities be designed to achieve quantitative probabilistic Target Performance Goals.

The Standard is intended to be used in the design of new nuclear facilities. It was written to utilize other nationally recognized codes, consensus standards, and guidance documents wherever appropriate. Its focus is on steel or reinforced concrete structures; however it also addresses plant systems and equipment. Criteria for development of appropriate design earthquake ground motion, evaluation of seismic demand and capacity, load combinations, detailing for ductility, and quality assurance are provided. A commentary[1] is also included in the Standard which provides an explanation and supplementary information to assist in the application and understanding of the recommended requirements. The Standard was developed by a number of contributors who are experienced in the design of nuclear facilities and was reviewed by a number of outside agencies and personnel. It is promulgated by the American Society of Civil Engineers (ASCE) as a national voluntary consensus standard.

1.2 Objective

The objective of the study reported herein was to evaluate ASCE/SEI Standard 43-05 with regard to its potential application to the design of nuclear power plants (NPPs). This objective was achieved by:

- Thoroughly reviewing the Standard and its Commentary, as well as references cited and other supporting documentation that provide its technical basis, to determine whether it would be appropriate for use in the seismic design of structures, systems, and components in NPPs.

- Identifying and evaluating the strengths and weaknesses of the Standard in terms of their technical and regulatory significance.

- Comparing the criteria presented in the Standard and the criteria contained in NRC regulatory guidance documents such as NRC Standard Review Plan (NUREG-0800), Regulatory Guides, Generic Communications, and current staff positions.

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[1] It should be noted that a commentary to an ASCE Standard is not part of the standard. Rather it is intended to provide explanatory and supplementary material that is intended to assist engineers and regulatory authorities in interpreting and applying the standard provisions.

1.3 Scope

As stated in the Foreword of ASCE 43-05, the Standard is applicable to facilities that process, store, or handle radioactive materials in a form and quantity that pose potential nuclear hazard to the workers, the public, or the environment. These facilities include both reactor and nonreactor facilities. The objective of the study described in this report, however, was specifically to evaluate the applicability of the Standard to the design of NPPs. Therefore, this study focused only on those criteria presented in the Standard that apply to NPPs and the type of SSCs that are normally found in NPP construction.

It appears that the Standard focuses primarily on structures constructed from steel and reinforced concrete members; however, Section 8 of the Standard is applicable to the design of equipment and distribution systems. Equipment items listed include vessels, heat exchangers, coolers, tanks, pumps, fans, valves, dampers, and electrical racks and cabinets. Distribution systems include pipe, conduit, cable tray, and HVAC ducts. In addition, criteria for designing supports to the various equipment items are covered in Section 8. Therefore, it appears that most SSCs found at NPPs are covered in the Standard. However, certain components do not appear to be addressed by the Standard such as containments, reactors, and earthen structures. These should be reviewed on a case-by-case basis until an acceptable approach is developed and reviewed by the NRC.

1.4 Evaluation Approach

The approach utilized to evaluate ASCE 43-05 was to review the design criteria contained in the Standard and the technical basis provided in other sources that led to these criteria. These other sources include the Commentary to the Standard and other applicable references cited in the Standard and Commentary. In addition, documents related to the early site permit (ESP) application for the Clinton NPP site were reviewed because the approach used in this ESP application is based on ASCE 43-05. The specific technical documents reviewed in this study are identified throughout this report where the particular topic was evaluated, and are provided in the Reference section (Section 7) as well.

All sections contained in ASCE 43-05 applicable to NPPs were evaluated. The critical elements in the criteria were identified and evaluated. Observations have been made where the provisions of the Standard do not appear to be consistent with NRC regulatory guidance documents. These observations are followed by recommendations for alternative approaches, further justifications, or the need to adhere to the currently accepted methods in NRC regulatory guidance documents. In some cases, recommendations have been made where further detailed review would help resolve the identified question or concern.

2 SUMMARY OF SEISMIC DESIGN CRITERIA IN ASCE/SEI STANDARD 43-05

2.1 Introduction

ASCE/SEI Standard 43-05 was developed to provide criteria for the seismic design of safety-related structures, systems, and components (SSCs) for use in nuclear facilities. These nuclear facilities can cover a broad range of facilities that process, store, or handle radioactive materials. The goal of the Standard is to present seismic design criteria which will ensure that nuclear facilities can withstand the effects of earthquakes with desired performance, expressed as probabilistic Target Performance Goals. This goal is achieved using a functionally graded approach in which the design criteria are commensurate with the relative importance to safety, safeguards, and security; magnitude of hazard; importance of radiological hazards; and other relevant factors.

Using this graded approach, a total of 20 Seismic Design Bases (SDBs) have been defined in Table 1-1 of the Standard (reproduced, for convenience, as Table 2-1 of this report). These SDBs represent a combination of five seismic design categories (SDCs) and four Limit States. Each SDC has a specified quantitative probabilistic Target Performance Goal. The appropriate SDB is selected on the basis of two parameters: the seismic design category and the Limit State. The Standard indicates that ANSI/ANS 2.26 provides criteria for selecting the SDC and Limit State which then define the SDB for the SSCs housed within the facility.

Following the definition of the SDB, the Standard indicates that ANSI/ANS 2.27 and 2.29 are to be used to characterize the site and to determine the appropriate ground motion, defined in terms of a design response spectra that has been developed from a probabilistic seismic hazard analysis (PSHA). Then, the criteria presented in Sections 1 and 2 of the Standard are used to develop the design basis earthquake (DBE) ground motion for the particular SDC. The calculation that leads to the development of the DBE considers the quantitative Target Performance Goal and the level of conservatism inherent in the seismic design criteria used in the evaluation of the seismic demand and structural capacity of the SSCs. Two probability goals are provided which define this level of conservatism.

1. Less than about a 1% probability of unacceptable performance for the DBE ground motion, and

2. Less than about a 10% probability of unacceptable performance for a ground motion equal to 150% of the DBE ground motion.

Condition (1) governs for “high-variability” situations, while condition (2) governs for “low-variability” cases[2], where the use of condition (1) might result in undue conservatism. The DBE in the Standard is comparable, for purposes of designing SSCs, to the safe shutdown earthquake (SSE) in Regulatory Guide 1.165, Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion. Once the DBE is determined, Sections 3 through 8 of the Standard provide specific design criteria for SSCs

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[2] Terms "high-variability" and "low-variability" relate to the magnitude of the logarithmic standard deviation, ß that describes the uncertainty in seismic capacity. These are defined quantitatively in Section 3. The breakpoint between "small" and "large" is at ß approximately equal to 0.40.

which include seismic demand, structural capacity, load combinations and acceptance criteria for structures, ductile detailing requirements, special considerations, and requirements for equipment and distribution systems. Section 9 of the Standard addresses seismic quality provisions including quality assurance. All topics that are applicable to NPPs are reviewed and evaluated in the appropriate sections of this report.

The Standard also provides a Commentary to the Standard. However, as indicated in the Commentary to the Standard, it is "not a part of the ASCE Standard *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*. It is included for information purposes." The Commentary is also reviewed as part of this study because it provides further explanations and supplementary material for interpreting the provisions and for ensuring that the seismic criteria are properly applied.

It should be noted that the Standard is applicable only to the design of new facilities. It provides specific guidance for addressing ground motion issues. It does not address other well known issues associated with earthquake effects such as differential fault displacement, seismic slope instability, and liquefaction. The methodology in the Standard is comparable to the seismic design approach presented in the U.S. Department of Energy (DOE) Standard DOE-STD-1020-2002, Natural Phenomena Hazards (NPH) Design and Evaluation Criteria for Department of Energy Facilities, for critical facilities identified as being in SDCs 3, 4, and 5.

As noted earlier, the Standard covers nuclear facilities that process, store, or handle radioactive materials in a form and quantity that pose potential nuclear hazard to the workers, the public, or the environment. Such facilities include both reactor and nonreactor facilities. While the Standard was not developed specifically for nuclear power plant SSCs, there is no indication in the Standard that NPPs are excluded from its scope of application. In fact, there are many instances in the Standard where reference is made to NPPs and to NRC criteria. For example, Section 1.2 of the Standard indicates that conventional buildings may be assigned to SDC-1, whereas nuclear power plants may be assigned to SDC-5. In the Commentary, Table C1-1 presents representative applications of the graded approach. For SDC-5 and Limit State D, the table entry states "similar to modern NRC NPP." Also, in Section 9 of the Standard, which addresses seismic quality provisions, a statement is made that the "seismic analysis and design of nuclear facilities specified in this Standard will be performed under the purview of the US Department of Energy (DOE) or the US Nuclear Regulatory Commission (NRC)." Based on these representative examples, one may conclude that the developers of ASCE 43-05 intended that it may be applied to NPPs as well as to other nuclear facilities.

2.2 Seismic Design Basis

As noted in the previous section, each Seismic Design Basis represents a combination of a Seismic Design Category and a Limit State. The SDCs are used to define the probability levels for design earthquakes and structural performance (acceptance criteria). The SDCs range from 1 for conventional buildings to 5 for facilities considered as hazardous such as nuclear power plants. SSCs are placed, for design purposes, into one of the five SDCs based on their importance and the inherent hazards associated with their failure. The four Limit States are used to define the analysis methodology, design procedures, and acceptance criteria. The Limit States range from Limit State A, representing large permanent distortion associated with near-collapse conditions, to Limit State D, representing deformations that remain essentially within the elastic range (Table 1-4 of the Standard). It should be noted that Limit State D associated with “essentially elastic deformation” does not preclude consideration of damage potential, since elastic design stress levels may still be associated with cracking of concrete confinement systems where flow paths for radiation through these cracks is a possibility.

The developers of ASCE 43-05 have presumed that all design bases falling within SDC-1 and SDC-2 are covered by the provisions in ASCE Standard 7-02 (recently revised as ASCE Standard 7-05) and the International Building Code (IBC 2003 edition) code requirements. Thus, the provisions in ASCE 43-05 specifically address the remaining SDCs 3, 4, and 5, with 12 identifiable SDBs. Quantitative hazard levels and target performance goals, expressed in terms of annual probabilities of exceeding the hazard or acceptable design Limit State, respectively, are stipulated for each SDC. Specific performance goals for structural acceptance criteria (P_F), seismic hazard levels (H_D), and probability ratios ($R_P = H_D / P_F$) are provided for each SDC, as shown in Table 1-2 of the Standard (reproduced, for convenience, as Table 2-2 of this report). Increasingly critical SSCs are designed to increasingly more severe hazard demands and lower Target Performance Goals (annual probabilities).

While the Standard does not stipulate Limit States and SDCs specifically for commercial nuclear reactor facilities, it may be inferred from Section 1.1 and from Tables 1-2 through 1-4 of the Standard that the standard-writers intended that SSCs found in NPPs would fall, almost exclusively, in a Seismic Design Basis consisting of SDC-5 and Limit State D (denoted SDB-5D). Section 1.1 of the Standard indicates that nuclear power plant requirements correspond to SDC-5. Tables 1-1 and 1-4 of the Standard (and Table 2-1 of this report) indicate that Limit State D is associated with “essentially elastic behavior” which is consistent with NRC regulatory guidance documents. Furthermore, Table C1-1 of the Standard indicates that for SDC-5 and Limit State D, the approach is “similar to modern NRC NPPs.”

For SDB-5D, the probability targets for seismic hazard and structural performance are $H_D = 1 \text{ X } 10^{-4}$/yr and $P_F = 1 \text{ X } 10^{-5}$/yr, respectively. For comparison, US Department of Energy (DOE) 5480.28 has stipulated Performance Categories ranging from 0 to 4 to define the seismic hazard and design criteria for structures, systems and components (SSC) in DOE facilities. The goal for performance category 4 is "occupant safety, continued operation, confidence of hazard confinement." The hazard and performance probability levels associated with DOE category PC 4 are identical to those associated with SDB-5 in the Standard.

A perspective on these annual probabilities may be obtained by restating their equivalents with respect to a 50-yr reference period, the latter being the basis for probability-based codified design for natural hazards in ASCE Standard 7-05, and the Model Codes (IBC 2003 and NFPA 5000 (2003)) that reference that standard. The annual probabilities of the Target Performance Goals found in Tables 1-2 and 1-3 of the Standard are equivalent, on a 50-yr basis and effective return period basis, to the values shown in Table 2-3 of this report. The seismic hazard maps in ASCE Standard 7-05 are based on spectral accelerations with 2%/50 yr probability (the maximum considered earthquake, or MCE), which are multiplied by the factor 2/3 to obtain the design earthquake spectral response accelerations (DS) used in structural design. At sites in seismically active regions in the Western United States (WUS), the corresponding DS hazard is approximately 10%/50 yr (return period of 475 yr), while in the Central and Eastern United States (CEUS) this hazard is approximately 4%/50 yr (return period of approximately 1,200 yr), due to differences in the typical slopes of seismic hazard curves in the WUS and CEUS. For structures in Seismic Use Group I (SUG I in ASCE Standard 7-05) designed in conformance with ACI Standard 318-02 or ANSI/AISC 360-05, the corresponding probability of incipient collapse is less than 2%/50 years, or 4 x 10^{-4}/yr. On the other hand, substantial nonlinear action is permitted for building structures falling within ASCE Standard 7 - SUG I (manifested in the R and C_d factors tabulated for different construction in Table 12.2-1 of ASCE Standard 7-05), while ASCE/SEI Standard 43-05 - SDB-5D mandates essentially elastic behavior with no damage.

Thus, the requirements in ASCE 43-05 are conservative, by any objective measure, with respect to customary building code requirements for critical facilities (identified as Occupancy Category IV in ASCE Standard 7-05, Table 1-1). The adequacy of the requirements in the Standard for the development of a seismic design ground motion for NPPs which meets an appropriate safety goal is discussed later in Section 3 of this report.

2.3 Use of Other Codes and Standards with ASCE/SEI Standard 43-05

Throughout the Standard, there are extensive cross-references to other standards and guidelines for the development of the design-basis seismic ground motion and the design acceptance criteria covered in Sections 3 through 9.

To classify the seismic design basis of the SSC at a given facility, reference is made to ANSI/ANS 2.26. This standard provides guidance to categorize the SSCs into the appropriate SDC and Limit State. ANSI/ANS 2.27 is referenced in the Standard to provide guidance on scope and methodology for site characterization requirements and ANSI/ANS 2.29 is referenced to provide guidance on procedures for performing probabilistic seismic hazard analysis, the fundamental input for definition of the design ground motion.

It should be noted that ANSI/ANS 2.26 was published in 2005, with the intention that it be utilized with ANSI/ANS 2.27, ANSI/ANS 2.29, and ASCE 43-05 to establish the design response spectra and the design and construction practices of the SSCs in a nuclear facility. ANSI/ANS 2.26 was developed based on the methods used by the US DOE for performance categorization, design criteria, and design procedures for SSCs in nuclear facilities. This guidance is contained in DOE standards DOE-STD-1020-2002, DOE-STD-1021-93 (reaffirmed in 2002), DOE-STD-1022-94 (reaffirmed in 2002), and DOE-STD-1023-95 (reaffirmed in 2002). ANSI/ANS 2.26 states that it is applicable to nuclear facilities *other than commercial power reactors* since such facilities are not under the direct jurisdiction of the DOE, the prime motivator of the Standard. Therefore, ANSI/ANS 2.26 is not applicable and should not be utilized if ASCE 43-05 is accepted for use in the seismic design of SSCs at NPPs. Instead, it is recommended that all safety-related SSCs at NPPs be classified as SDC-5 and Limit State D (SDB-5D), as described in Section 2.2 above.

Section 1.2 of ASCE 43-05 lists a number of consensus codes and standards that are to be used with the Standard. None of these codes and standards is accompanied with dates/editions. The Commentary does identify the dates or editions of some codes and standards; however, the Commentary cannot be relied upon to ensure the use of the appropriate codes and standards contained in the provisions of ASCE 43-05 because it is not part of the Standard and because in many cases the editions are not compatible with the current requirements of the NRC. Therefore, the specific dates or editions of codes and standards referenced in Section 1.2 of ASCE 43-05 should be identified to allow confirmation that these codes and standards have been endorsed by the NRC for the seismic design of NPPs. Otherwise, each use of this Standard will need to be reviewed on a case-by-case basis to ensure the acceptability of the codes and standards utilized.

When specifying the appropriate codes and standards, it would be acceptable to use the versions already endorsed or accepted by the NRC in their regulatory guidance documents. For example, Regulatory Guide 1.142 accepts the use of American Concrete Institute Standard ACI 349-97, subject to 15 regulatory positions contained in that Regulatory Guide. Further examples of codes and standards currently endorsed by the NRC are contained in the various sections of this report that deal with the particular SSC being evaluated.

There are cases where the current NRC regulatory guidance documents endorse an old version of a code or standard. This does not imply that updated versions are somehow deficient and should not be utilized. According to SRM SECY 93-087, the Commission approved the staff's position that "consistent with past practice, the staff will review both evolutionary and passive plant design applications using the newest codes and standards that have been endorsed by the NRC. Unapproved revisions to codes and standards will be reviewed on a case-by-case basis." Thus, until the NRC updates the SRP and Regulatory Guides to reflect more current industry codes and standards, licensees can utilize more modern editions of the codes and standards; however, they will be subject to review on a case-by-case basis. It is recommended that the NRC consider updating its regulatory guidance documents (e.g., 10CFRs, SRPs, and RGs) to reflect the current generation of codes and standards and to develop one source of regulatory guidance that identifies the endorsement and any special regulatory positions that may apply to codes and standards. This would result in an efficient and productive utilization of up-to-date knowledge, would avoid having to repeatedly review each licensing application for its use of recent codes and standards, and would ensure consistency in the use of referenced codes and standards across the inventory of NPPs.[3]

[3] National voluntary consensus standard-writing organizations require that once a new edition of a standard has been processed and is ready for publication and promulgation, the existing edition be formally withdrawn. By endorsing provisions in old rather than current documents, the NRC regulatory guidance documents are in effect endorsing provisions that the standard-developing organizations no longer recognize or support with interpretations.

Table 2-1 Seismic Design Basis (SDB)

	Limit State			
SDC	A Large Permanent Distortion (Short of Collapse)	B Moderate Permanent Distortion	C Limited Permanent Distortion	D Essentially Elastic
1	SDB-1A	SDB-1B	SDB-1C	SDB-1D
2	SDB-2A	SDB-2B	SDB-2C	SDB-2D
3	SDB-3A	SDB-3B	SDB-3C	SDB-3D
4	SDB-4A	SDB-4B	SDB-4C	SDB-4D
5	SDB-5A	SDB-5B	SDB-5C	SDB-5D

SDC is the Seismic Design Category

Table 2-2 Earthquake Design Parameters for SDC 3, 4 & 5

	SDC		
	3	4	5
Target Performance Goal (P_F)	1×10^{-4}	4×10^{-5}	1×10^{-5}
Probability Ratio (R_P)	4	10	10
Hazard Exceedance Probability (H_D) $H_D = R_P \times P_F$	4×10^{-4}	4×10^{-4}	1×10^{-4}

Table 2-3 Target Performance Goal - Annual Probability, Probability of Exceedance, and Approximate Return Period

SDC	Annual Probability	Probability of Exceedance	Approximate Return Period
1	1×10^{-3}	5%/50 yr	1,000 yrs
2	4×10^{-4}	2%/50 yr	2,500 yrs
3	1×10^{-4}	0.5%/50 yr	10,000 yrs
4	4×10^{-5}	0.2%/50 yr	25,000 yrs
5	1×10^{-5}	0.05%/50 yr	100,000yrs

3 Earthquake Ground Motion

This section provides a description of the risk-consistent and performance-based methodologies used in ASCE/SEI Standard 43-05 for the development of the Design Response Spectrum (DRS). The DRS serves the same purpose as the safe shutdown earthquake (SSE) for the design of structures, systems and components (SSCs) in nuclear power plants (NPPs). This description is based on the categorization of SSCs within Seismic Design Category 5 and Limit State D (SDB-5D) as discussed in Section 2.2 of this report. A review and assessment are provided with respect to the selection of the performance target, the procedure for SSE development, and the factors-of-safety to achieve the probability goals. The SRM/SECY-93-087 seismic margin requirement and its relevance to ASCE 43-05 seismic design criteria are also evaluated. In addition, the ASCE 43-05 requirements are evaluated with regard to the definition of SSE at varying depths and the development of synthetic or modified recorded time histories.

To place the assessment of ASCE 43-05 criteria for the ground motion development in a proper context, Regulatory Guide (RG) 1.165 is also summarized. This Regulatory Guide outlines a procedure that currently is acceptable to the NRC staff for determining the SSE for new NPPs using hazard-consistent probabilistic seismic hazard analysis (PSHA) methodologies.

3.1 Overview of Current Regulatory Practice

Prior to the 1970s, as the licensing of the earlier generation of commercial nuclear power plants was in progress, the seismic design philosophy was evolving towards the use of design seismic loads based on the site-specific investigation of local and regional seismology, geology and geotechnical engineering. The concept of the safe shutdown earthquake (SSE) emerged in the early 1970's and was codified into the federal regulations with the publication, in December 1973, of Appendix A, “Seismic and Geologic Siting Criteria for Nuclear Power Plants,” to 10 CFR Part 100, “Reactor Site Criteria.” The NRC subsequently published a series of Regulatory Guides in support of Appendix A, including RG 1.60, “Design Response Spectra of Nuclear Power Reactors.”

RG 1.60 provides ground design response spectral shapes for horizontal and vertical directions that are considered acceptable to the NRC staff. The RG 1.60 spectra were developed from a statistical analysis of response spectra of past WUS strong-motion earthquakes collected from a variety of different site conditions, primarily at deep soil sites. However, the procedure for defining the SSE by RG 1.60 is deterministic, and therefore may not address adequately the uncertainties inherent in estimates of the SSE that are associated with the seismological and geological evaluations arising from site locations (WUS vs. CEUS), fault geometry, rupture characteristics, seismicity, source-to-site characteristics and ground motion attenuation. For some sites, particularly hard rock sites in the CEUS, the RG 1.60 spectral shape is not considered appropriate, particularly at frequency ranges above 10 Hz. Recommended spectral shapes for such sites (e.g., NUREG/CR-6728) indicate significant power in spectra above 10 Hz that is not captured in the RG 1.60 spectral shape. Furthermore, the SSE response spectra computed using the RG 1.60 spectral shape do not achieve a uniform probability of earthquake hazard exceedance across the entire frequency range. To achieve an SSE with a uniform probability of exceedance across the frequency range and to fully account for the uncertainties in earthquake ground motion, a probabilistic seismic hazard analysis (PSHA) or a suitable sensitivity analysis must be conducted.

During the late 1980s and early 1990s, both the nuclear industry through EPRI (EPRI NP-4726) and the NRC through LLNL (NUREG-1488) conducted large scale programs to systematically investigate the seismic hazard and to apply state-of-the-art PSHA methodologies to obtain seismic hazard estimates for nuclear power plant sites in the Central and Eastern United States. To reflect the state of knowledge and to incorporate the latest advances into seismic hazard estimates, the NRC published RG 1.165 in March 1997. This guide provides procedures acceptable to the NRC staff for 1) conducting geological, geophysical, seismological, and geotechnical investigations, 2) identifying and characterizing seismic sources, 3) conducting PSHAs, and 4) determining the SSE for satisfying the requirements of 10 CFR 100.23.

NRC Standard Review Plan (SRP) 2.5.2 states that the determination of the SSE, including supporting probabilistic seismic hazard analysis and derivation of controlling earthquakes, is acceptable if it follows the procedures in RG 1.165. RG 1.165 stipulates that a PSHA can be used as a means to determine the SSE and to account for uncertainties in the seismological and geological evaluations arising from fault geometry, rupture characteristics, seismicity and ground motion modeling. The RG identifies a "reference probability" as that probability of exceeding, on an annual basis, the SSE at future NPP sites. This reference probability, based on the distribution of median probabilities of exceeding the SSE at 29 sites in the Eastern United States (EUS) (NUREG-1488), is 10^{-5}/yr. Figure B.2 of RG 1.165 shows that the median probability of exceeding the SSE for 29 operating NPP sites east of the Rocky Mountains using the LLNL hazard curves is approximately 10^{-5}/yr.[4] The deaggregation of the PSHA hazard curves are used to select the low-frequency (1-2.5 Hz) and high-frequency (5-10Hz) controlling earthquakes at the reference probability. The ground motion estimates are made for rock conditions in the free-field or by assuming hypothetical rock conditions for a nonrock site. Using the controlling earthquakes, response spectral shapes for the actual or assumed rock conditions are determined. Then, the response spectrum shapes, corresponding to the low-frequency and high-frequency controlling earthquakes, are scaled to arrive at the SSE response spectra corresponding to the two frequencies. The smooth or broad-band SSE spectra to be used in design shall then envelope the two SSE spectra corresponding to the low and high frequencies. For nonrock sites, the soil surface SSE response spectra can be obtained from two (low and high frequency controlling earthquakes) site-specific soil amplification analyses consistent with the rock SSE.

Subsequent to the trial use of RG 1.165 by the nuclear industry in early site permit (ESP) applications, the industry expressed concerns that the use of the guideline, especially the selection of the reference probability, may lead to regulatory and technical instability in the licensing of future plants. As mentioned above, the reference probability is computed as the median probability obtained from the distribution of median probabilities of exceeding the SSE at 29 sites in the EUS. The sites selected were intended to represent relatively recent designs that used RG 1.60 spectra or similar as their design bases. The purpose is to ensure an adequate level of conservatism in determining the SSE consistent with recent licensing decisions. However, RG 1.165 also requires that PSHAs for the 29 sites need to be updated at periodic intervals; as a result, the computed reference probability may differ from 10^{-5}/yr if

[4] The mean probability of exceeding the SSE is estimated as 1.8 x 10^{-5}/yr, occurring at the 64th percentile of the distribution in Figure B.2 of RG 1.165; the 10th percentile to 90th percentile range being 0.3 to 7.0 x 10^{-5}/yr

substantial changes to only a few PSHAs occur.[5] Therefore, seismic activity in the vicinity of one (or a few sites) will result in changes in the SSE estimates at a site which may be located at a remote distance from the affected site (or sites). This could result in additional expenditure and resource spending for a future site for potential changes to the SSE, even though no changes of seismic activity at the site has been identified.

3.2 Risk-Consistent Approach to Development of Design Ground Motions

In contrast to RG 1.165, ASCE 43-05 presents a performance-based approach for determining design ground motions. In this approach, the seismic hazard is convolved with an SSC fragility to arrive at a probability, P_F, of unacceptable seismic performance. The design ground motion is then back-calculated to be consistent with a specified level of seismic hazard, reflected in the ratio R_P identified in Table 1-2 of the Standard (Table 2-2 of this report). The final result is a site-specific design-basis ground motion that is essentially risk-consistent for all SSCs in the NPP.

Section 2 and its Commentary of ASCE 43-05 establish the procedure for determining the design basis earthquake in terms of a risk-consistent design response spectrum (DRS). Since the DRS is equivalent to the SSE spectrum for design purposes, the term SSE (the word "spectrum" is understood, and for brevity, is dropped) is used throughout this report to avoid any ambiguity in the use of the terminology.

Section 2 of ASCE 43-05 requires that the SSE be based on a probabilistic seismic hazard analysis (PSHA). Such an analysis customarily yields a *mean* seismic hazard curve representing the probabilities that specific levels of spectral accelerations of a 5% elastic damped oscillator in a frequency range of approximately 0.1 to 50 Hz are exceeded, and a set of uniform hazard response spectra (UHRS) constructed from the hazard analysis at different probability levels. These UHRS are interpreted as *mean* response spectra.

The provisions in Section 2 of ASCE 43-05 are based on structural reliability analysis. This analysis required for developing the risk-consistent SSE is summarized below to provide the basis for examining the risk implications of ASCE 43-05 and for evaluating the applicability of the Standard to the design of SSCs in NPPs.

The Limit State probability P_F of an SSC can be determined as the convolution of the seismic hazard curve, H(a) and the mean fragility curve, F_C(a) of that SSC over the ground motion level *a* by either of the following two mathematically equivalent equations:

$$P_F = \int_0^\infty H(a)\frac{dF_C(a)}{da}da \qquad (3.1a)$$

$$P_F = -\int_0^\infty F_C(a)\frac{dH(a)}{da}da \qquad (3.1b)$$

[5] The instability would be worse if the *mean* rather than the median probability were to be chosen for the reference probability, since the mean is more sensitive to small changes in the sample than the median.

Since the seismic hazard curve is assumed to be known, Equation (3.1a) is a more useful representation of risk, and will be used to develop a simplified risk equation in this section.

The Limit State probability P_F is defined in ASCE 43-05 as the target annual mean frequency of exceeding a specified Limit State (or target performance goal). The hazard curve, H(a), defines the mean seismic hazard curve, which is expressed as a Cauchy-Pareto complementary cumulative distribution function (CCDF). Since H(a) is approximately linear in log-log scale over a ten-fold reduction in annual frequency, it can be approximated as,

$$H(a) = k_1 a^{-K_H} \tag{3.2}$$

in which k_1 and K_H are constants. Parameter K_H represents the slope of the mean seismic hazard curve when plotted on log-log scale. If A_R represents the ratio of spectral accelerations at probability levels $0.1H_D$ and H_D, in which H_D = probability of exceedance at which the UHRS is defined, then

$$K_H = \frac{1}{\log(A_R)} \tag{3.3}$$

Seismic hazard analysis indicates that when the seismic hazard is expressed in terms of spectral acceleration, K_H also is frequency-dependent (K_H increases slightly as the natural frequency at which spectral acceleration S_a is determined increases from 1 Hz to 10 Hz). Moreover, typical slopes of mean seismic hazard curves in regions of high seismicity are markedly different from those in regions of low-to-moderate seismicity. Typical values of K_H in the Western United States (WUS) would be in the range of 3 to 6 (A_R between 1.6 and 2.2), while in the Central and Eastern United States (CEUS), K_H is typically 2.5 or less (A_R greater than 2.5).

The mean fragility is defined by the lognormal cumulative distribution function (CDF),

$$F_C(a) = \Phi\,[\ln(a/a_C)/ß] \tag{3.4}$$

in which a_C is the median capacity, ß is the composite logarithmic standard deviation, which represents contributions of both aleatoric ($ß_R$) and epistemic uncertainty ($ß_U$) in seismic demand and capacity[6], in that $ß = (ß_R^2 + ß_U^2)^{1/2}$, and Φ [ ] is the standard normal probability distribution. Based on past seismic probabilistic risk assessments (Park et al., 1998 and NUREG/CR-6728 (McGuire, et al., 2001)), for most SSCs, ß lies in the range of 0.3 to 0.6. In NUREG/CR-6728, it has been suggested that a typical value of ß would be approximately 0.45. This value is used in the calculations that follow to illustrate the assessment of the provisions in ASCE 43-05.

[6] Earlier seismic margin studies tracked the aleatoric uncertainties (those due to inherent randomness) and epistemic uncertainties (due to limitations in knowledge) separately. Current practice combines them in fragility assessment, leading, in a Bayesian sense, to a mean fragility.

Substituting Eqs. (3.2) and (3.4) into Eq. (3.1a), one obtains the so-called "risk equation,"

$$P_F = H(a_C)\exp\left[(K_H\beta)^2 / 2\right] \tag{3.5}$$

which states, in words, that the limit state probability is equal to the seismic hazard, evaluated at the median capacity of the SSC, multiplied by a correction factor that depends on the slope of the seismic hazard curve and the logarithmic standard deviation, ß. For example, with K_H = 2.5 and ß = 0.45, this correction factor is approximately 1.88.

For purposes of implementation in ASCE 43-05, the terms in this risk equation are restated in terms of design quantities that are familiar to the regulatory authority and to the structural engineer. First, suppose that the p-percentile value of the mean fragility in Eq. (3.4), a_p, is used for the design capacity of SSCs (*p* will be defined shortly). Then,

$$a_P = a_C \exp\left[- X_P\beta\right] \tag{3.6}$$

in which the factor X_p is the (1-p)-percentile of the standard normal deviate. Substituting the value of a_C from Eq. (3.6) into Eq. (3.5), we obtain,

$$P_F = k_1(a_P)^{-K_H} \exp\left[\left(K_H\beta^2\right) / 2 - X_P K_H \beta\right] \tag{3.7}$$

Second, we assume that a_p is related to the DRS (or SSE) by a seismic margin factor, F_P:

$$a_P = F_P \times DRS = F_P \times DF \times UHRS \tag{3.8}$$

in which F_p is a factor of safety keyed to the p-percentile value, a_p, and DF is a seismic Design Factor that is used to scale the UHRS (specified for SDC-5 in Table 1-2 of ASCE 43-05 at a mean annual frequency of exceedance of 10^{-4}/yr) to obtain the design-basis earthquake, as stipulated in Equation 2-1 of the Standard. Third, we define H_D as the annual probability of exceeding the UHRS, or

$$H_D = H(UHRS) = k_1(UHRS)^{-K_H} \tag{3.9}$$

Substituting Eq. (3.8) into Eq. (3.7) and making use of Eq. (3.9) yields,

$$P_F = H_D(F_P DF)^{-K_H} \exp\left[(K_H\beta)^2 / 2 - X_P K_H \beta\right] \tag{3.10}$$

Solving Eq. (3.10) for the required design factor yields,

$$DF = (F_P)^{-1}\left[R_P \exp\left((K_H\beta)^2 / 2 - X_P K_H \beta\right)\right]^{1/K_H} \tag{3.11}$$

in which $R_P = H_D / P_F$ [7] according to Tables 1-2 and 2-1 in the Standard for SDB-5D. It may be observed that DF is an increasing function of probability ratio R_P and a decreasing function of the seismic margin factor, F_P. It also decreases as K_H increases (thus, DF is an increasing function of A_R).

The NRC has accepted the notion of a high-confidence of low probability of failure (HCLPF) value as the 1-percentile value (1% exclusion limit) of the capacity described by the mean fragility in Eq. (3.4), making $X_p = 2.326$ in Eq. (3.6). If the seismic margin factor on $a_{1\%}$ = HCLPF is set equal to 1.0 for design of SSCs (i.e., $F_{1\%} = 1.0$ in Eq. (3.11), the design factor becomes,

$$DF = \left[R_p \exp\left(\left(K_H \beta \right)^2 / 2 - 2.326 K_H \beta \right) \right]^{1/K_H} \tag{3.12}$$

This DF in Eq. (3.12) is tantamount to the seismic margin above the UHRS that is required to achieve a factor of safety ≥ 1.0 against a 1% conditional probability of failure for a stipulated value of R_P when the HCLPF and UHRS are specified as above. This margin is equivalent to the first of the two seismic acceptance criteria in Section 1.3 of ASCE 43-05. This DF also is equivalent to the safety factor, F_P, in the Standard Commentary Eq. (C2-9).

To avoid the above probabilistic formulations for practical design purposes, the design-basis ground motion in Section 2.2.1 of ASCE 43-05 simply is represented by a design response spectrum (DRS), defined from the UHRS by,

DRS = DF x UHRS (Eq. 2-1)[8]

in which the Design Factor, DF, is defined for SDB-5D as

$DF = \max [1.0, 0.6 A_R^{0.8}]$ (Eq. 2-3)

As noted above, the purpose of the DF is to scale the UHRS upward for purposes of achieving a risk-consistent design response spectrum. The design factor DF in Eq. (2-3) is an approximation to Eq. (3.12). It should be noted that Eq. (3.12) is a function of both K_H and ß, while Eq. (2-3) in the Standard depends only on K_H. The approximation of Eq. (3.12) by Eq. (2-3) simplifies the application of the Standard with little sacrifice in reliability.

A comparison of DF-values from Eq. (2-3) and Eq. (3.12) is shown in Table 3-1. The exact DF is calculated using $R_P = 10$ for SDB-5D in ASCE 43-05, ß, values of $F_{1\%}$ taken from the commentary to Section C1.3, and ground motion ratio A_R in a range from 1.5 to 6.0. As shown in Table 3-1, the required DF compares well with the DF estimated using the ASCE 43-05 formula for the practical range of the parameter variations. The ASCE DF is generally higher

[7] Throughout this report, it will be assumed that $R_P = 10$ for SDB-5D, unless otherwise noted.

[8] Where referring specifically to an equation appearing in the Standard, the equation number from the standard is used, rather than the sequential numbering otherwise followed in this Report.

than the exact DF for ß > 0.40. Conversely, if ß < 0.4, Eq. (2-3) may underestimate the exact value, especially for small values of K_H. This non-conservatism is manifested in a slightly higher P_F. For example, if ß = 0.3 and A_R = 4, then P_F from Eq. (3.10) is $0.13H_D$ rather than $0.10H_D$; this 30% difference is not significant in view of the epistemic uncertainties involved in the analysis of seismic demand and SSC fragility. Note that Eq. (2-3) also assures that an SSE is always equal to or greater than the mean UHRS, indicating that it provides an acceptable DF estimate for practical purposes.

Using the ASCE design factors defined by Eq. (2-3), the performance goal (expressed as limit state probability, P_F) can be determined. Table 3-2 presents the performance goal calculated using Eq. (2-3) for DF and using Eq. (3.10) for P_F, in which F_P = 1.0 and X_P = 2.326 (the first acceptance criterion in Section 3.1 of the Standard). Table 3-2 presents the calculated performance goal P_F for A_R in a range of 1.5 to 6.0 and variability ß in a range of 0.3 to 0.6. As indicated in Table 3-2, the performance goal target of 1 x 10^{-5} is generally achieved over a small range. For the range of A_R and ß indicated in the table, 0.5 x 10^{-5} < P_F < 1.2 x 10^{-5}, again a range that is acceptable in view of the uncertainties in seismic hazard and structural response analysis.

In summarizing the ASCE 43-05 approach for determining the SSE to be used for design of an NPP, a mean target performance goal $P_F = 10^{-5}$/yr is selected according to the assigned seismic design category and the limit state: SDB-5D. With a probability ratio R_P = 10, the mean seismic hazard is calculated by the relation: $H_D = P_F \times R_P$, leading to $H_D = 10^{-4}$/yr. The ground motion ratio A_R is determined from the mean UHRS corresponding to the seismic hazard $0.1H_D$ and H_D levels. The risk-consistent SSE is calculated using Eqs. (2-1) and (2-3) across the entire frequency range of interest. We emphasize that the ASCE approach begins with the limit state probability target, P_F, and derives a design factor DF to arrive at an SSE that achieves the target performance goal with a factor of approximately 10 separating H_D and P_F. This approach is fundamentally different from the approach taken in the current RG 1.165, as discussed subsequently.

Although the ratio $R_P = H_D/P_F$ has been fixed at 10 in the above discussion, with the Standard stipulating $H_D = 10^{-4}$ for SDB-5D, many combinations of UHRS and DF could be selected to achieve the same probabilistic performance goal. In particular, RG1.165 states that a reference probability of exceeding the SSE equal to 10^{-5}/yr is acceptable to the NRC staff. Suppose that ß = 0.45 and A_R = 4; if R_P were set equal to 1.0 (corresponding to $P_F = H_D = 10^{-5}$/yr) rather than 10, the DF from Eq. (3.12) would be 0.42. Conversely, if the DF were to be calculated from Eq. (2-3) under these same conditions, $P_F = 0.0058\ H_D$, substantially less than the target of $0.1H_D$ stipulated in Table 2-1. From such analyses, it becomes apparent that the target performance goal (P_F) and hazard exceedance probability (H_D) appear to have been selected appropriately, in a relative sense, for design and regulatory purposes in ASCE 43-05.

3.3 Quantitative Performance Target P_F

While the goal of both the Nuclear Regulatory Commission and ASCE 43-05 is to control the design process so that performance of structures, systems and components with regard to safety and environmental protection is acceptable, RG 1.165 and ASCE 43-05 take a somewhat different approach to achieving this goal in performance prediction and evaluation for earthquakes.

According to RG 1.165, a PSHA can be used as a means to determine the SSE and to account for uncertainties in the seismological and geological evaluations arising from fault geometry, rupture characteristics, seismicity and ground motion modeling. Specific steps are described in RG 1.165 for performing a PSHA that is acceptable to the NRC. RG 1.165 identifies a "reference probability" as that probability of exceeding, on an annual basis, the SSE at future NPP sites. This reference probability, based on the distribution of median probabilities of exceeding the SSE at 29 NPP sites in the EUS (NUREG-1488), is 10^{-5}/yr. The ground motions developed according to RG 1.165 are used along with the criteria in SRP 3.7 and 3.8 to design steel and reinforced concrete structural components and systems in the NPP. These criteria include nominal material strengths and factors of safety (resistance factors or allowable stress factors), which are consistent, for the most part, with those found in national consensus standards and specifications. Because of the inherent conservatism in these nominal design parameters and the factors of safety employed in national consensus standards and specifications, the limit state probability of a typical properly designed NPP structure will almost certainly be less than 10^{-5}/yr, but to an unknown degree.

One might argue that since the current inventory of operating plants is considered safe, a seismic hazard at median 10^{-5}/yr is a suitable basis for the SSE. However, the limit state probability for plant structures remains undetermined and unspecified in this approach. Because this approach cannot address the impact of differences in K_H (or A_R), and the role of structural fragilities (manifested by the median capacity a_C and composite uncertainty ß) on risk, the RG1.165 approach cannot lead to uniform seismic risk across SCCs in the inventory of NPPs. Furthermore, as noted in Section 3.1 of this report, the requirements of RG 1.165 may result in changes to the reference probability of 10^{-5}/yr in the future due to the required periodic updates of PSHAs for the 29 NPP sites. If this were to occur, future modifications of the SSE may be required for a particular site, even though the seismic hazard for that site remains unchanged. This situation conceivably could require periodic changes to the plant licensing basis, which is neither desirable nor practical.

ASCE 43-05 takes a different approach, beginning with the stipulation of an acceptable Performance Goal rather than a seismic hazard. The fundamental performance goal for SSCs categorized as SDC-5 (the appropriate category for NPPs) is expressed in terms of an annual probability of failure to meet the performance goal, or limit state probability (ASCE 43-05, Table 1-3). For structures falling within SDB-5D, which includes NPP structures, this performance goal might be stated simply as "The probability of the onset of nonlinear behavior (or permanent deformation) shall be less than 10^{-5}/yr." This is often alternatively referred to as "Frequency of Onset of Significant Inelastic Deformation (FOSID)." The point at which Onset of Significant Inelastic Deformation (OSID) occurs is clearly depicted in the Commentary of ASCE 43-05 (Figure C5-4, p. 68). The selection of the target probability $P_F = 10^{-5}$/yr for SDB-5D in ASCE 43-05 is explained by a finding (NUREG-1742) that the median seismic core damage frequency (CDF) of 25 plants that had undergone a seismic probabilistic risk assessment was 1.2×10^{-5} and a judgment that the core damage frequency would be substantially less than that value if the performance goal had been set equal to that value when the SSCs originally were designed. (The 10th percentile value of median core damage frequency for this same group of plants is approximately 1.0×10^{-6}/yr, while the 90th percentile is close to 10^{-4}/yr, indicating the substantial range in CDF for these plants.) However, the context and specific aims of the IPEEE should be considered in interpreting this target probability goal.

NUREG-1742 is an insights report for the IPEEE, a study that was aimed at identifying plant vulnerabilities to seismic events and may not have been subjected to the same level of rigor that is required for licensing applications. The objective of the IPEEE was to gain a qualitative understanding (versus the quantitative insights required by the IPE program) of the overall likelihood of core damage due to external events. Therefore, the IPEEE submittals were reviewed to ensure that the intent of the Generic Letter 88-20, Supplement 4, was met, and the CDF or LERF (Large Early Release Frequency) frequencies submitted by the licensees were not validated by the NRC staff. The quantitative CDF data for each of the plants in NUREG-1742 were intended to be used to provide a comparison of the relative risk values across different plants.

The level of rigor in the seismic PRAs (SPRAs) performed for the IPEEE program was not the same for all plants studied. NUREG-1742 has identified a number of weaknesses which may potentially impact some SPRA results. A majority of the SPRAs performed for the IPEEE program used the margin screening criteria as provided in EPRI-6041; in some cases, the screening thresholds were set equal to the review level earthquake assigned to the plant, which may be too low for developing PRA insights.

About 40% of the plants performed SPRAs and the rest of the plants performed seismic margin assessments (SMAs). Therefore, it is not clear to what extent the use of the median CDF from these 25 SPRAs is representative of the entire US NPP inventory.

The use of an average (or median) CDF from a group of NPP sites as a criterion for selecting the target performance goal raises a similar concern as that identified for the seismic hazard target in RG 1.165. In particular, because of the concerns raised in NUREG-1742 about the SPRAs from the IPEEE program, future improvements in these SPRAs are anticipated, and these improvements may affect the computed average CDF. The American Nuclear Society (ANS) has recently developed a guidance document for assessing the quality of the performance of external-events PRA methodology (ANSI/ANS-58.21-2003).

An approach for the long term would be to stipulate the target performance goal to be consistent with the risk goal prescribed by the 1986 Commission's policy statement on the Safety Goals for the Operations of Nuclear Power Plants (51 FR 28044). Under Section V - Guidelines for Regulatory Implementation, the Commission indicated that the overall mean Large Release Frequency (LRF) should be less than 10^{-6}/yr. It is believed that the LRF is a good risk measure for advanced reactors of all types, while the CDF is a LWR-specific risk measure that may not apply to all advanced reactor designs. However, since the CDF is a surrogate for the latent cancer QHO (Quantitative Health Objective) and the LRF is a surrogate for the prompt fatality QHO, *both* CDF and LRF may need to be considered in establishing the target performance goal. Relating such risk goals to the structural criteria in ASCE 43-05 would require a major research effort, and is not recommended for short-term implementation of the Standard.

Taking into consideration the above review, the Target Performance Goal of 10^{-5}/yr (mean) for SSCs in SDB-5D is considered to be reasonable. Since the onset of significant inelastic structural deformations would occur at seismic demands lower than those that would threaten core damage, it is probable that structural design of SSCs based on the SDB-5D performance objective would lead to estimated core damage frequencies that tend to be lower than those resulting from the past SPRA evaluations, although again by an unknown degree. It is understood that the safety goal is an objective and not a requirement; however, subsidiary safety objectives in terms of CDF or LRF ultimately might be considered in the selection of the target performance goal.

3.4 Development of Safe Shutdown Earthquake (SSE) Spectrum

The seismic risk P_F is determined by the convolution of seismic hazard curve H(a) and structural fragility F_C(a). The SSE then is "back-calculated" from the Performance Goal (acceptable seismic risk P_F) so as to ensure that it is met with an appropriate level of conservatism (F_P) in the structural design process. F_P is the seismic margin factor at the conditional failure probability level P.

Since the seismic hazard curves at plant sites in the United States differ by slope, the probability of exceeding the SSE may vary from site to site. While the uniform hazard spectrum, which is the starting point, is specified at mean 10^{-4}/yr (ASCE 43-05, Table 2-1), the design response spectrum (SSE) is defined by Eq. (2-1). As described in Section 3.2 of this report, the design factor DF accounts for the characteristics of the seismic hazard in an exact manner in Eq. (3.11). An approximate simplified relation for DF is proposed by ASCE 43-05 as indicated by Eq. (2-3). The uncertainties in the structural capacities (reflected in the composite logarithmic standard deviation, ß, in structural fragility) are reflected indirectly in Eq. (2-3) through the (generally) conservative approximation that it provides to Eq. (3.11).

3.5 Factors of Safety to Achieve Target Performance Goals

As indicated in Section 3.2, the target performance goal is achieved by requiring an appropriate level of conservatism in the structural design process defined in terms of the seismic margin factor F_P as expressed in Eq. (3.8). ASCE 43-05 stipulates two criteria for the seismic margin factor F_P that must be achieved:

(1) less than 1% probability of unacceptable performance for the SSE, and

(2) less than 10% probability of unacceptable performance for 1.5 SSE.

These two conditions (expressed in mathematical form as: $F_{1\%}$ = 1.0 and $F_{10\%}$ = 1.5) result in nominal factors of safety that are tabulated in the Commentary and in Table 3-3. Condition (1) governs for high-variability" situation (greater than 0.39), while condition (2) controls for "low-variability" cases. The Commentary of ASCE 43-05 demonstrates that meeting the deterministic seismic design criteria as outlined in the Standard will achieve both conditions (1) and (2) for the seismic margin factors. The two criteria were developed to make the Standard broadly applicable to nuclear facilities within the purview of several agencies. Criterion (1) is likely to govern design of SSCs in NPP systems because their ß's typically are larger than 0.4 and significant inelastic action is essentially precluded in NPP design (see Section 4.1.5). However, for completeness, both criteria are evaluated below.

The conservatism inherent in the ASCE 43-05 seismic criteria results from three elements: 1) prescribed strengths, 2) seismic demand analysis, and 3) ductility estimate.

- The seismic strengths are prescribed in ASCE 43-05 in terms of the ACI code ultimate strengths, the AISC code LRFD limit state strengths including the code specified strength reduction factors, and the ASME code service level D strengths. These code strengths are stated to have at least a 98% probability of exceedance. For low ductility failure modes, an additional factor of conservatism of 1.33 is typically introduced (ASCE 43-05, Commentary C1.3.1.2.1).

- Seismic demand analysis in Section 3 of ASCE 43-05 is performed in accordance with the requirements of ASCE Standard 4 except that median spectral amplification factors are used instead of median-plus-one-standard deviation amplification factors. In addition, ASCE Standard 4 aims at 10% failure probability given the SSE.

- The ductility achieved in ASCE 43-05 corresponds to 5% non-exceedance probability (NEP) level (ASCE 43-05, Commentary C1.3.1.2.3).

All three elements are described by lognormal distributions, and the associated variability is expressed in terms of lognormal standard deviations varying in a range of 0.2 to 0.4.

Combining these three elements of conservatism according to Section C1.3.1 of the Commentary of ASCE 43-05, the following nominal factors of safety are derived:

$$F_{N1\%} = \frac{C_{1\%}}{C_{std}} = R_C e^{-2.326\beta} \tag{3.13a}$$

$$F_{N10\%} = \frac{C_{10\%}}{C_{std}} = R_C e^{-1.282\beta} \tag{3.13b}$$

where, $C_{1\%}$ and $C_{10\%}$ are the seismic capacities corresponding to 1% and 10% conditional probability of failure, and R_C and ß are given by:

$$R_C = 0.82 e^{2.054\beta_S + 1.28\beta_D + 1.645\beta_N} \quad \text{(Ductile failures)} \tag{3.14a}$$

$$R_C = 1.09 e^{2.054\beta_S + 1.282\beta_D} \quad \text{(Low ductility)} \tag{3.14b}$$

and

$$ß = (ß_S^2 + ß_D^2 + ß_N^2)^{½} \tag{3.14c}$$

in which $ß_S$, $ß_D$ and $ß_N$ represent the variability associated with strengths, demands and ductility estimates, and typically vary between 0.2 and 0.4, resulting in the variability *ß* approximately between 0.3 and 0.6.

Table 3-3 presents $F_{N1\%}$ and $F_{N10\%}$ for values of $ß_S$, $ß_D$ and $ß_N$ in the range of 0.2 to 0.4. The table shows that the seismic margin factors $F_{N1\%} = 1.0$ and $F_{N10\%} = 1.5$ are essentially achieved over the range of values for $ß_S$, $ß_D$ and $ß_N$. Therefore, based on the above assumptions and methodology, it may be concluded that the seismic design criteria in ASCE 43-05 satisfy the two conditions for the seismic margin factors.[9]

3.6 SRM/SECY-93-087 Seismic Margin Requirement

In SECY-93-087, the staff examined the PRA-based seismic margin analyses and made certain recommendations to the Commission. In the subsequent Staff Requirement Memorandum (SRM) SECY-93-087, the Commission approved the use of a PRA-based seismic margins analysis which considers sequence-level HCLPFs and fragilities for all sequences leading to CDF up to approximately 1.67 times the SSE. The staff anticipated that fully developed seismic PRAs will be performed for new NPP designs, and sufficient margin shown in the plant level HCLPF will effectively identify any seismic significant contributors to risk and capture potential design-specific seismic vulnerabilities.

The SRM/SECY-93-087 requirement for plant-level HCLPF margins appears to apply to the seismic PRA analyses and insights, and is endorsed by the Commission as a regulatory requirement. ASCE 43-05 is a consensus document for seismic design criteria and does not address issues related to PRA-based seismic margin assessments. However, one issue which may need to be considered is whether a NPP designed using ASCE 43-05 will achieve a plant HCLPF equal to 1.67 x SSE if evaluated using a PRA-based margin analysis.

An SSC HCLPF value is equivalent to setting the seismic margin factor in Eq. (3.8) to $F_{1\%}$, which is the 1-percentile on the mean fragility curve. In other words, the HCLPF would be defined as:

$$\text{HCLPF} = 1.67 \times \text{DF} \times \text{UHRS} \qquad (3.15)$$

Note that $F_{1\%}$ required in ASCE 43-05 is based on a set of seismic design criteria which are presumed to be more stringent than the criteria for computing a seismic margin HCLPF value. The Commentary of ASCE 43-05 (summarized in Table 3-3, as discussed previously) has demonstrated that the ASCE seismic criteria achieve the seismic margin factors $F_{1\%} = 1.0$ and $F_{10\%} = 1.5$. However, whether an $F_{1\%} = 1.67$ at the plant level can be achieved, based on a PRA margin evaluation, remains to be determined.

There is uncertainty surrounding the calculation of HCLPFs. Past margins studies have shown that a wide range of HCLPF capacity estimates can be developed when calculations are performed by different analysts, even with the same problem definition. These differences occur because substantial subjective inputs are required for HCLPF calculations, especially with respect to the selection of median-centered input parameters such as material strengths, damping, ductility, etc. Although the general methodology for computing a HCLPF is well established, a guideline which leads to consistent results from independent analysts for median-centered input parameters has yet to be developed.

However, once the issues regarding the plant level HCLPF calculation are resolved, the plant CDF can be readily determined from Eq. (3.10), with $X_p = 2.326$ and $F_{1\%} = 1.67$:

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[9] It should be noted that current design provisions for NPPs do not permit inelastic structural behavior to be considered explicitly, so the columns labeled "ductile" in Table 3-3 are not applicable to NPP design.

$$CDF = P_F = H_D\left(1.67DF\right)^{-K_H}\ \exp\left[\left(K_H\beta\right)^2\ /\ 2 - 2.326K_H\beta\right] \tag{3.16}$$

To put this CDF in perspective with the performance goal for SDB-5D in ASCE 43-05, Table 3-4 presents the CDF calculated using Eq. (3.16), with DF determined by Eqs. (2-3(a) and 2-3(b)) and $H_D = 10^{-4}$/yr (as per SDC-5). The CDF values are calculated considering A_R in the range of 1.5 to 6.0 and variability ß in the range of 0.3 to 0.6. If $F_{1\%} = 1.67$ can be achieved (which has yet to be demonstrated), then the performance goal target of 10^{-5}/yr for SSCs would translate into a plant CDF in the range of 0.36 x 10^{-6}/yr to 6.95 x 10^{-6}/yr, with values tending to increase for sites where A_R is larger (K_H smaller). Such sites would tend to be in the Central and Eastern United States.

3.7 Definition of SSE at Varying Depths

Given the definition of a UHRS for horizontal motion at a given control location, the motion at the other depths of the overburden soil/rock profile is typically determined via equivalent-linear convolution analyses based on assumed upward propagating shear waves in which nonlinear soil/rock properties are approximately accounted for by using equivalent linear iteration in shear strain along with appropriate strain-dependent modulus reduction and damping amplification models for the individual layer site materials. These degradation models are best determined from dynamic laboratory tests conducted on undisturbed samples taken of each layer of the site profile. Generic degradation models may be used where appropriate or where sampling is difficult.

ASCE 43-05 recommends that a Monte Carlo procedure be used in which a number of such convolution analyses are performed to adequately capture the effect of variability or uncertainty in soil properties. Enough realizations of the convolution calculations need to be performed from which best estimate (BE), upper bound (UB) and lower bound (LB) profiles are determined at the mean and mean plus/minus one standard deviation of the iterated shear moduli. The iterated shear moduli include the effects of nonlinear behavior of the foundation material in the site response calculation. The Standard indicates that enough site property data must be developed to be able to properly capture the mean and variability of the properties in the site response calculations. The final response spectrum at the output depth (e.g., ground surface, foundation depth) in the profile is then determined as the mean of the spectra obtained from each site convolution realization.

To avoid the issue of convolving up the soil column with artificial time histories generated from large broad-banded response spectra, the Standard recommends that convolution sets be performed using input motions associated with appropriate seismic events (defined by their magnitude and distance from the site) selected from the deaggregated sources associated with the seismic hazard for the site. The input motion spectra are recommended to be defined at low frequency (about 1 Hz) and high frequency (10Hz) to ensure that the soil column will not be “over-driven” into unrealistically high nonlinear behavior by motions associated with large broad-banded input spectra.

Since a typical site may not have uniform soil properties across the entire zone of interest, more than one initial base-case soil profile may need to be selected to represent the site condition, and the probabilistic convolutions are performed for each base-case profile following the ASCE procedure. The final output spectrum should then be determined by enveloping the mean spectra computed from the different sets of convolution analyses. The mean spectrum computed in this manner should have a broader bandwidth than those computed with a single initial base-case profile.

To convert the site UHRS to the DRS, ASCE 43-05 indicates that the UHRS needs to be defined at $0.1H_D$ and H_D; the slope factor A_R then is determined and the corresponding design factor DF is calculated from Eq. (2-3). If convolution analyses are performed to determine the UHRS at another output depth (e.g., ground surface, foundation depth), this will require that two sets of convolution analyses are performed at the two annual probability of exceedance (APE) levels defined above. The final DRS is determined using Eq. (2-1) with the convolved UHRS and DF.

A deterministic approach is outlined in Section 3.7.2.II.4 of the SRP, which requires similar convolution or deconvolution analyses to be performed for use in soil structure interaction (SSI) analyses. However, in this approach, only three such analyses need to be performed for site shear wave velocity and damping properties set at their BE, LB and UB values. For sites that are not well investigated, the SRP indicates that three analyses be performed at shear wave velocities corresponding to the BE, one-half the BE, and twice the BE shear moduli of the soil foundation materials. The final response spectrum at the output depth in the profile is then obtained by enveloping the spectra from the three individual convolution analyses. The SRP does not discuss issues associated with over-driving the soil profile by using large broad-banded input motions or the need to develop risk-consistent spectra.

The description of the recommended procedures described above refers to convolution of horizontal ground motions. ASCE 43-05 indicates that vertical motions should be developed following the requirements of ASCE 4. However, as described in the Commentary to ASCE 43-05, similar one-dimensional computational procedures assuming upward propagating P-waves are not recommended to determine corresponding vertical motions. This is due to uncertainty in the adequacy of performing the one-dimensional convolution process to predict vertical motions. Significant contribution to vertical motions is developed from converted P-SV motions as well as from the direct P-wave. Therefore, ASCE 43-05 recommends that vertical motions should be generated from the computed horizontal motions using frequency-dependent V/H ratios generated from the empirical data-base. Based on the current state-of-the-art, this approach is considered to be reasonable.

3.8 Synthetic or Modified Recorded Time Histories

ASCE 43-05 criteria for computing synthetic time histories are generally consistent with, and in some aspects more detailed than those in, the SRP. However, three requirements, related to the damping selection, Power Spectral Density (PSD) check, and the correlation coefficient for demonstration of statistical independence, are not consistent. These are discussed separately below:

<u>Damping requirement:</u>

ASCE 43-05 requires that the response spectrum from the synthetic time history envelop the DRS with 5% damping, while SRP Section 3.7.1 II.1.b requires that the response spectrum from the synthetic time history envelop the DRS for all damping values actually used in the response analysis. Since methods for generating synthetic time histories compatible with multiple damping ratios are abundant in the literature, it should not cause any undue hardship to meet the SRP requirement.

Power Spectral Density (PSD) requirement:

ASCE 43-05 does not require a check of the PSD unless the computed spectrum for the synthetic time history exceeds the DRS by more than 30% at any frequency in the frequency range between 0.2 Hz and 25 Hz. SRP Section 3.7.1 II.1.b (Option 1) requires that the synthetic time history must in general satisfy requirements for both enveloping the DRS as well as adequately matching a target power spectral density function which is compatible with the DRS.

In NUREG/CR-6728, it is indicated that the ground motion fitting process is designed to provide time histories whose spectra closely match the SSE and have characteristics that are appropriate for the defined characteristic events (such as strong motion duration). If the matching criteria are satisfied, there is no need to develop a PSD.

It is recommended that the criteria for ground motion fitting described in NUREG/CR-6728 and ASCE 43-05 be implemented, and if the criteria are satisfied there is no need to check that the synthetic time history matches a target power spectral density function which is compatible with the DRS. If the criteria cannot be satisfied, then the additional requirement for matching the target power spectral density function should be performed.

Note, however, that Appendix A to SRP Section 3.7.1 presents the definition for the target PSD as a function of Fourier spectral amplitudes that is appropriate for application to artificial time histories developed for use with RG 1.60 spectral shapes. No other definitions of target PSD functions are currently available for application to other target spectral shapes, and in particular, to CEUS shapes with significant high frequency content. Both the SRP and ASCE 43-05 should provide guidance on development of such targets.

Demonstration of Statistical Independence:

ASCE 43-05 indicates that to be considered statistically independent, the directional correlation coefficients between pairs of records shall not exceed a value of 0.30. The current NRC staff position is contained in Regulatory Guide 1.92, Rev. 2, which refers to a Journal Article by C. Chen (1975) for guidance on demonstrating statistical independent time histories. This study (Chen, 1975), stated “it is recommended that the statistically independent artificial time histories should be those with absolute correlation coefficients less than or equal to 0.16.” This criterion has been utilized in past licensing applications for NPPs. A current study being performed in support of the SRP 3.7 update has confirmed the appropriateness of the 0.16 criterion. Therefore, the 0.16 criterion should continue to be utilized for demonstrating statistical independent time histories for analysis of SSCs at NPPs, rather than the 0.30 specified in ASCE 43-05.

Table 3-1 Comparison of Exact *DF* with ASCE Estimate

Exact *DF*							ASCE *DF*
A_R	K_H	ß=0.3 $F_{1\%}$=1.1	ß=0.4 $F_{1\%}$=1.0	ß=0.45 $F_{1\%}$=1.0	ß=0.5 $F_{1\%}$=1.0	ß=0.6 $F_{1\%}$=1.0	
1.50	5.68	0.88	0.93	0.94	0.95	1.03	1.00
1.75	4.11	0.95	0.96	0.93	0.91	0.91	1.00
2.00	3.32	1.05	1.03	0.98	0.95	0.90	1.04
2.25	2.84	1.16	1.11	1.05	1.00	0.93	1.15
2.50	2.51	1.27	1.21	1.13	1.07	0.97	1.25
2.75	2.28	1.38	1.30	1.22	1.14	1.03	1.35
3.00	2.10	1.49	1.40	1.30	1.22	1.08	1.44
3.25	1.95	1.61	1.50	1.39	1.30	1.14	1.54
3.50	1.84	1.72	1.60	1.48	1.38	1.21	1.63
3.75	1.74	1.83	1.70	1.57	1.46	1.27	1.73
4.00	1.66	1.95	1.80	1.66	1.54	1.34	1.82
4.25	1.59	2.07	1.90	1.75	1.62	1.40	1.91
4.50	1.53	2.18	2.01	1.84	1.70	1.47	2.00
4.75	1.48	2.30	2.11	1.94	1.79	1.54	2.09
5.00	1.43	2.41	2.21	2.03	1.87	1.60	2.17
5.25	1.39	2.53	2.31	2.12	1.95	1.67	2.26
5.50	1.35	2.64	2.42	2.21	2.04	1.74	2.35
5.75	1.32	2.76	2.52	2.31	2.12	1.80	2.43
6.00	1.29	2.88	2.62	2.40	2.20	1.87	2.52

Table 3-2 Performance Goal (x 10^{-5}) Achieved Using ASCE/SEI Standard 43-05 DF

A_R	K_H	DF	ß = 0.3 $F_{1\%}$ = 1.1	ß = 0.4 $F_{1\%}$ = 1	ß = 0.45 $F_{1\%}$ = 1	ß = 0.5 $F_{1\%}$ = 1	ß = 0.6 $F_{1\%}$ = 1
1.50	5.68	1.00	0.47	0.67	0.69	0.76	1.20
1.75	4.11	1.00	0.82	0.84	0.75	0.69	0.68
2.00	3.32	1.04	1.02	0.95	0.82	0.72	0.61
2.25	2.84	1.15	1.02	0.92	0.78	0.68	0.55
2.50	2.51	1.25	1.04	0.92	0.78	0.68	0.53
2.75	2.28	1.35	1.05	0.92	0.79	0.69	0.54
3.00	2.10	1.44	1.07	0.93	0.80	0.70	0.55
3.25	1.95	1.54	1.08	0.95	0.82	0.71	0.56
3.50	1.84	1.63	1.10	0.96	0.83	0.73	0.57
3.75	1.74	1.73	1.11	0.97	0.85	0.74	0.59
4.00	1.66	1.82	1.12	0.98	0.86	0.76	0.60
4.25	1.59	1.91	1.13	1.00	0.87	0.77	0.61
4.50	1.53	2.00	1.14	1.01	0.88	0.78	0.62
4.75	1.48	2.09	1.15	1.02	0.90	0.79	0.64
5.00	1.43	2.17	1.16	1.02	0.91	0.81	0.65
5.25	1.39	2.26	1.17	1.03	0.92	0.82	0.66
5.50	1.35	2.35	1.18	1.04	0.92	0.83	0.67
5.75	1.32	2.43	1.18	1.05	0.93	0.83	0.68
6.00	1.29	2.52	1.19	1.05	0.94	0.84	0.68

Table 3-3 Nominal Factors of Safety Achieved with ASCE/SEI Standard 43-05

$ß_S$	$ß_D$	$ß_N$	Low ductile R_C	Ductile R_C	Low ductile $ß$	Ductile $ß$	Low ductile $F_{N1\%}$	Ductile $F_{N1\%}$	Low ductile $F_{N10\%}$	Ductile $F_{N10\%}$
0.20	0.20	0.20	2.12	2.22	0.28	0.35	1.10	0.99	1.48	1.42
0.20	0.20	0.30	2.12	2.62	0.28	0.41	1.10	1.00	1.48	1.54
0.20	0.20	0.40	2.12	3.09	0.28	0.49	1.10	0.99	1.48	1.65
0.20	0.30	0.20	2.41	2.52	0.36	0.41	1.04	0.97	1.52	1.49
0.20	0.30	0.30	2.41	2.98	0.36	0.47	1.04	1.00	1.52	1.63
0.20	0.30	0.40	2.41	3.51	0.36	0.54	1.04	1.00	1.52	1.76
0.20	0.40	0.20	2.74	2.87	0.45	0.49	0.97	0.92	1.55	1.53
0.20	0.40	0.30	2.74	3.38	0.45	0.54	0.97	0.97	1.55	1.70
0.20	0.40	0.40	2.74	3.99	0.45	0.60	0.97	0.99	1.55	1.85
0.30	0.20	0.20	2.61	2.73	0.36	0.41	1.13	1.05	1.64	1.61
0.30	0.20	0.30	2.61	3.21	0.36	0.47	1.13	1.08	1.64	1.76
0.30	0.20	0.40	2.61	3.79	0.36	0.54	1.13	1.08	1.64	1.90
0.30	0.30	0.20	2.97	3.10	0.42	0.47	1.11	1.04	1.72	1.70
0.30	0.30	0.30	2.97	3.65	0.42	0.52	1.11	1.09	1.72	1.88
0.30	0.30	0.40	2.97	4.31	0.42	0.58	1.11	1.11	1.72	2.04
0.30	0.40	0.20	3.37	3.52	0.50	0.54	1.05	1.01	1.78	1.77
0.30	0.40	0.30	3.37	4.15	0.50	0.58	1.05	1.07	1.78	1.97
0.30	0.40	0.40	3.37	4.90	0.50	0.64	1.05	1.10	1.78	2.15
0.40	0.20	0.20	3.20	3.35	0.45	0.49	1.13	1.07	1.81	1.79
0.40	0.20	0.30	3.20	3.95	0.45	0.54	1.13	1.13	1.81	1.98
0.40	0.20	0.40	3.20	4.65	0.45	0.60	1.13	1.15	1.81	2.16
0.40	0.30	0.20	3.64	3.81	0.50	0.54	1.14	1.09	1.92	1.91
0.40	0.30	0.30	3.64	4.49	0.50	0.58	1.14	1.16	1.92	2.12
0.40	0.30	0.40	3.64	5.29	0.50	0.64	1.14	1.19	1.92	2.33
0.40	0.40	0.20	4.14	4.33	0.57	0.60	1.11	1.07	2.00	2.01
0.40	0.40	0.30	4.14	5.10	0.57	0.64	1.11	1.15	2.00	2.24
0.40	0.40	0.40	4.14	6.01	0.57	0.69	1.11	1.20	2.00	2.47

Table 3-4 Estimate of Plant CDF (x10^{-6}/yr), Given $F_{1\%}$ = 1.67 Is Achieved at Plant Level

A_R	K_H	DF	CORE DAMAGE FREQUENCY (CDF) X 10^{-6}/yr				
			ß = 0.3	ß = 0.4	ß = 0.45	ß = 0.5	ß = 0.6
1.50	5.68	1.00	0.44	0.36	0.37	0.41	0.65
1.75	4.11	1.00	1.47	1.02	0.91	0.84	0.82
2.00	3.32	1.04	2.55	1.73	1.49	1.31	1.11
2.25	2.84	1.15	3.12	2.14	1.83	1.59	1.28
2.50	2.51	1.25	3.63	2.52	2.15	1.87	1.47
2.75	2.28	1.35	4.07	2.87	2.46	2.14	1.67
3.00	2.10	1.44	4.46	3.19	2.75	2.39	1.87
3.25	1.95	1.54	4.80	3.48	3.01	2.62	2.05
3.50	1.84	1.63	5.10	3.74	3.25	2.84	2.23
3.75	1.74	1.73	5.37	3.98	3.47	3.04	2.40
4.00	1.66	1.82	5.61	4.20	3.67	3.23	2.56
4.25	1.59	1.91	5.83	4.40	3.86	3.41	2.70
4.50	1.53	2.00	6.03	4.59	4.03	3.57	2.84
4.75	1.48	2.09	6.22	4.76	4.20	3.72	2.98
5.00	1.43	2.17	6.39	4.92	4.35	3.87	3.10
5.25	1.39	2.26	6.54	5.07	4.49	4.00	3.22
5.50	1.35	2.35	6.69	5.21	4.62	4.13	3.33
5.75	1.32	2.43	6.82	5.34	4.75	4.25	3.44
6.00	1.29	2.52	6.95	5.46	4.87	4.36	3.54

4 Seismic Demand, Capacity, and Acceptance Criteria

4.1 Seismic Demand

Section 3.1 of ASCE/SEI Standard 43-05 stipulates that seismic demand for SSCs shall be computed in accordance with the requirements of ASCE Standard 4, Seismic Analysis of Safety-Related Nuclear Structures. In applying ASCE Standard 4, ASCE 43-05 allows the use of linear equivalent static analysis, linear dynamic analysis, complex frequency response methods, or nonlinear analysis methods.

4.1.1 Acceptable Editions of Industry Codes and Standards

As indicated in Section 2.3 of this report, in most cases the Standard does not identify the date or edition of the applicable code or standard (such as ASCE Standard 4 in this case)[10]. This can lead to potential problems in the future when newer editions of these codes/standard are utilized for design without the opportunity of the NRC to review their acceptance. Therefore, the specific dates or editions of codes and standards referenced in Section 1.2 of ASCE 43-05 should be identified to allow confirmation that these codes and standards have been endorsed by the NRC for the seismic design of NPPs.

4.1.2 Technical Adequacy of ASCE Standard 4

Most of the analysis methods described in ASCE Standard 4 (the latest version is 1998) are consistent with the accepted methods in NRC regulatory guidance documents such as the Standard Review Plan (NUREG-0800) (SRP) and Regulatory Guides. However, the NRC has not officially endorsed or generically accepted the use of ASCE Standard 4. NRC is currently updating Regulatory Guides such as RG 1.92 and RG 1.61, and NRC is also undertaking an update of the SRP, in order to have updated guidance in place prior to the anticipated submittal of new reactor applications. However, a review by the NRC for generic acceptance of ASCE Standard 4 at this time is not anticipated, and a complete technical review of ASCE Standard 4 is beyond the scope of this study. Therefore, for purposes of this study, a number of areas where the Standard specifically references ASCE Standard 4 for analysis methods have been reviewed and compared with the guidance presented in NRC regulatory documents. These differences are discussed in this report. The NRC may want to consider a future activity to review the current ASCE Standard 4 for generic acceptance. This would avoid the need to review selected applications of the analysis methods contained in ASCE Standard 4 on a case-by-case basis each time it is referenced in licensing applications.

4.1.3 Linear Equivalent Static Analysis Methods

Section 3.2.1 of the Standard describes an equivalent static analysis method to evaluate single-point-of-attachment cantilever models with essentially uniform mass distribution, or other simple structures that can be represented as single degree-of-freedom systems. The equivalent static load base shear is determined by multiplying the cantilevered structure, equipment, or

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[10] Page iii of ASCE 43-05 lists current ASCE Standards with their dates. However, this cannot be considered an endorsement since (a) this page is not part of Standard 43-05, and (b) all ASCE standards are listed, including those that are irrelevant to NPP construction.

component masses by an acceleration equal to the peak of the input response spectrum. The base moment is determined by using an acceleration equal to 1.1 times the peak of the response spectrum and applying the resulting load at the center of gravity of the structure. Reviewing NRC regulatory guidance documents, the equivalent static load method is described in Section 3.7.2, II, 1, b of the SRP for seismic analysis of SSCs. However, this section of the SRP requires that a factor of 1.5 be applied to the peak acceleration of the applicable floor response spectrum. According to the SRP, a factor of less than 1.5 may be used if adequate justification is provided. In SRP 3.9.2, II, 2, a, (2), which addresses dynamic testing and analysis of systems, components, and equipment, the same guidance is provided as in Section 3.7.2, II, 1, b of the SRP. However, SRP 3.9.2, II, 2, a, (2), also indicates that for equipment which can be modeled adequately as one-degree-of-freedom systems, the use of a static load equivalent to the peak of the floor response spectra is acceptable. Thus, it can be concluded that the equivalent static approach contained in the Standard is acceptable; however, adequate justification needs to be provided that the SSC can be modeled as a one-degree-of-freedom system or that a factor of less than 1.5 is still conservative.

Section 3.2.1 of the Standard also describes the use of an equivalent static analysis method for cantilevers with nonuniform mass distribution and other simple multiple-degree-of-freedom structures. If the predominant or fundamental mode shape of the structure has a curvature in one direction only (e.g., cantilever mode), the equivalent static load is determined as the product of the structure, equipment, or component masses by an acceleration equal to 1.5 times the peak acceleration of the response spectrum. The Standard also indicates that a smaller factor may be used, if justified. This approach is consistent with the guidance provided in Sections 3.7.2, II, 1, b and 3.9.2, II, 2, a, (2) of the SRP which were discussed above. Also, IEEE Std 344-1987 (formally accepted by RG 1.100, Rev.2) and the 2004 edition (not yet accepted by NRC) basically describe the same approach for seismic qualification of Class 1E equipment for NPPs. IEEE Std 344 states that the 1.5 static coefficient factor was determined from experience to take into account the effects of multifrequency excitation and multimode response for linear frame-type structures (similar to beams and columns), which can be represented by a simple model. Based on the above discussion, this second equivalent static method is considered acceptable.

A third equivalent static method is described in Section 3.2.1 of the Standard. If a modal solution has been performed in accordance with ASCE Standard 4, then the spectral acceleration value at the fundamental frequency of the structure may be used. The 1.1 factor for the base moment or the 1.5 factor defined for the nonuniform mass distribution and other simple multiple-degree-of-freedom structures shall be applied to the acceleration value determined at the fundamental frequency. This approach is not described in ASCE Standard 4, SRP 3.7 and 3.9, and in IEEE Std 344-1987 or -2004. In some cases this may be an acceptable approach; however without further guidance, this approach could under-predict the true response. For example, if the fundamental frequency of the SSC were to fall on the soft side of any peaks in the spectra, this approach could lead to unconservative member forces if it turns out that the SSC had a somewhat higher fundamental frequency or the SSC has higher modes with frequencies corresponding to accelerations greater than that at the fundamental frequency. Also, the use of the 1.1 factor rather than 1.5, for base moment, would have to be justified. Therefore, it is recommended that this approach not be generically accepted without further guidance; its application should be reviewed on a case-by-case basis.

4.1.4 Linear Dynamic Analysis Methods

Linear dynamic analysis methods are described in Section 3.2.2 of the Standard. These methods consist of the response spectrum and time history methods of analysis. Analysis may be performed by either time history integration or by modal superposition methods in accordance with Section 3.2.2 of ASCE Standard 4. The Standard also requires consideration of P-delta (P-Δ) effects if they result in greater than a 10% increase in the imposed moment demand of a structural member.

Seismic dynamic analysis methods using the response spectrum and time history methods of analysis are described in SRP Section 3.7.2, II, 1, a. Specific items to consider in these analysis methods are also described: effects of soil-structure interaction; torsional, rocking, and translational responses of the structures, and their foundations; use of an adequate number of masses or degrees-of-freedom in the dynamic model; use of sufficient number of modes; maximum relative displacements at the supports; and significant effects such as piping interactions, externally applied structural restraints, hydrodynamic loads, and nonlinear responses. Therefore, the use of the linear dynamic analysis methods identified in ASCE 43-05 and the reference to Section 3.2.2 of ASCE Standard 4 are acceptable provided that the specific provisions in ASCE Standard 4 that are followed are consistent or conservative when compared to the requirements in the SRP and applicable RGs. Any deviations from the NRC SRP or RGs must be reviewed on a case-by-case basis.

It should be noted that there are several provisions in ASCE Standard 4 that do not agree with current NRC regulatory documents and staff positions. As an example, the procedure described in Section 3.2.2.2.1(f) of ASCE Standard 4-98, which discusses an alternate method for considering the number of modes in a modal superposition analysis, states that the number of modes included shall be sufficient to ensure that inclusion of all remaining modes does not result in more than a 10% increase in the total response of interest. The current NRC technical position, as described in RG 1.92, Rev. 2 (Prepublication), is that this approach is "non-conservative and should not be used." Instead, two methods are described in RG 1.92, Rev. 2 (Prepublication) which the staff finds acceptable for considering the missing mass in a response spectrum analysis and mode superposition time history analysis.

P-Δ effects are not expected to be a significant concern in most cases where primary structures are being evaluated but may be of concern for evaluation of secondary systems. Structures at NPPs are generally quite stiff because they are relatively short, utilize reinforced concrete shear walls to resist lateral loads, and are designed to be stiff in order to prevent the detrimental effects of deformation to attached systems (e.g., piping). This would mitigate the additional loading due to P-Δ effects. However, if the P-Δ effects become significant, then they should be considered in the analysis.

4.1.5 Nonlinear Analysis Methods

Nonlinear analysis methods are described in Section 3.3 of the Standard. Guidance is provided for nonlinear static analysis and nonlinear dynamic analysis methods. For nonlinear equivalent static methods, the Standard states that the guidance in FEMA-356 for the target displacement method or in ATC-40 for the capacity spectrum method shall be followed. For nonlinear dynamic analysis methods, the Standard states that the procedures of Section 3.2 of ASCE Standard 4 shall be followed.

As described in Sections 2.2 and 2.3 of this report, the application of this Standard to NPPs should be limited to SDC-5 and Limit State D (essentially elastic response). Therefore, the provisions in Section 3.3 are not germane to the design of NPP SSCs. As stated in Section 3.7.2, II, 1 of the SRP, "The SRP criteria generally deal with linear elastic analysis coupled with allowable stresses near elastic limits of the structures. However, for certain special cases (e.g., evaluation of as-built structures), the staff has accepted the concept of limited inelastic/nonlinear behavior when appropriate. The actual analysis, incorporating inelastic/nonlinear considerations, is reviewed on a case-by-case basis." Another example where case-specific nonlinear analysis methods are acceptable is in Section 3.5.3 of the SRP when analyzing Category I structures, shields, and barriers for missile impact (e.g., tornado generated, pipe break). When geometric nonlinearities (e.g., consideration of impacts due to gaps between SSCs or liftoff due to rocking of structures) can occur, nonlinear analyses are appropriate. However, as before, these are reviewed on a case-by-case basis. Therefore, if certain nonlinear analysis methods as described in the Standard are utilized, they must be reviewed on a case-by-case basis.

4.1.6 Modeling and Input Parameters

The modeling of SSCs for seismic analysis is described in Section 3.4 of the Standard. According to the Standard, the modeling of SSCs for seismic analysis shall follow Section 3.1 of ASCE Standard 4. In addition, the Standard provides procedures for determining effective stiffness of reinforced concrete members, modeling of mass, and damping values for the SSCs.

<u>ASCE Standard 4</u>

Section 3.1 of ASCE Standard 4 provides procedures for modeling structures at NPPs. Topics covered include general requirements, developing models for horizontal and vertical motions, multistep and one-step methods of seismic response analysis, finite element discretization considerations, structural material properties, modeling of stiffness, modeling of mass, modeling of damping, modeling of hydrodynamic effects, dynamic coupling criteria, and requirements for modeling specific types of structures (e.g., frame type, shear wall type, plate and shell type). Section 3.1 of ASCE Standard 4 also indicates that requirements for modeling of soil-structure interaction are given in Section 3.3 of ASCE Standard 4.

As indicated earlier, the staff has not reviewed or formally accepted the use of ASCE Standard 4 for the seismic design of SSCs at NPPs. While most of the procedures in ASCE Standard 4-98 are consistent with SRP and RGs, there are some provisions that may not be acceptable. As an example, Section 3.3.1.10 of ASCE Standard 4-98 states that in the absence of analyses to establish the reduction in responses caused by the wave incoherence, it is conservative to assume the reductions to the ground response spectra as shown in Table 3.3-2 of ASCE Standard 4-98. This table provides reduction factors for ground response spectra at different frequencies as a function of the structure plan dimensions. For frequencies greater than 25 Hz, the reduction factors are 0.8 and 0.6 for plan dimensions of 150 ft (45.7 m) and 300 ft (91.4 m), respectively. The approach in Section 3.3.1.10 of ASCE Standard 4 has not been formally accepted; in fact, the subject of wave incoherency currently is being studied by the nuclear industry to develop an approach that would be acceptable to the NRC. Therefore, the use of procedures described in ASCE Standard 4 is acceptable provided that those specific procedures are consistent with or conservative with respect to the requirements in the SRP and applicable RGs. Any deviations from the NRC SRP or RGs must be reviewed on a case-by-case basis.

Effective Stiffness of Reinforced Concrete Members

As an alternative to a detailed stiffness calculation, Section 3.4.1 of ASCE 43-05 provides an approach for determining the effective stiffness values for reinforced concrete members. Table 3-1 of the Standard presents simple expressions for calculating the flexural rigidity, shear rigidity, and axial rigidity for reinforced concrete beams, columns, and walls. Although the technical basis for these expressions is not provided in the Standard, it appears that they are very similar to those given in Table 6-5 of FEMA 356 (2000). FEMA 356 (2000) is a pre-standard for the seismic rehabilitation of buildings. In most cases, the expressions in Table 3-1 of ASCE 43-05 and Table 6-5 of FEMA 356 are the same; however, there are some differences. As an example, for "Walls and diaphragms - uncracked," under the heading "Flexural Rigidity," Table 3-1 indicates using E_cI_g when $f_b<f_{cr}$, while Table 6-5 shows 0.8 E_cI_g. The basis for the tabulated expressions in FEMA 356 could not be identified, and thus their applicability to NPPs could not be assessed. This could be an activity for consideration in the future.

One of the stated goals of FEMA 356 was to encourage wider application of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA 273, by converting it into mandatory language. The commentary on the NEHRP guideline for rehabilitation of buildings is FEMA 274. A review of the older FEMA 274 standard provides some insight into the consideration of the effective stiffness of concrete members. FEMA 274 at first indicates that the various parameters (e.g., component dimensions, reinforcement quantities, boundary conditions, and stress levels) should be considered and verified when defining effective stiffnesses. It also acknowledges that a range of stiffnesses is possible for any set of nominal conditions, and that such variations may have a "considerable impact on the final assessment." However, FEMA 274 later indicates that it may be impractical to calculate effective stiffnesses directly, and therefore, the effective stiffness for the "linear procedures of Chapter 3 may be based on the approximate values of Table 6-4." As in the case of FEMA 356, the basis for the tabulated expressions in FEMA 274 could not be identified.

In addition to the above FEMA reports, which address seismic rehabilitation of buildings, the BSSC/NEHRP recommended provisions for seismic regulations for new buildings and other structures were published as FEMA Report 450 in 2003[11]. A review of FEMA 450 did not reveal substantive guidance on how to consider the effective stiffness of concrete members in the development of analytical mathematical models. Section 5.3 of FEMA 450, only states that for the response spectrum procedure, the stiffness properties of concrete and masonry elements shall consider the effects of cracked sections.

A more useful source for guidance on the effective stiffness of concrete members is Section 3.1.3 of ASCE Standard 4-98 which discusses the modeling of stiffness of reinforced concrete elements for application to safety-related nuclear structures. It indicates that concrete elements may be modeled using either cracked or uncracked sections, depending on the level of stress,

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[11] These recommendations form the basis for the provisions for earthquake-resistant design found in ASCE Standard 7-05 and the International Building Code (2003).

for the most critical seismic load combination. The commentary to this section in ASCE Standard 4-98 indicates that typically nominal shear stresses are kept below 100 psi (0.689 MPa) in shear walls, and therefore, the use of uncracked section properties for the determination of wall design forces is generally acceptable. However, ASCE Standard 4-98 acknowledges that if seismic analyses show that elements crack significantly due to critical load combinations, cracking effects should be considered. Since cracking of concrete can change the stiffness and frequency, it can affect the design seismic loads. As noted in ASCE Standard 4-98, in these cases "it is beneficial to perform a preliminary analysis of critical structural components subjected to combined loadings and assign cracked or uncracked stiffness properties to various elements based on their anticipated stress state. These stress states then should be compared to the stress states obtained by combining the final seismic stresses with other loads. If the locations and/or the extent of the final cracks differ from those initially assumed such that the responses (loads, accelerations, and displacements) are judged to have been altered significantly, the analysis should be repeated using "best-estimate" crack locations and extent."

No specific guidance is provided in SRP or RGs on this subject except in the Draft (Rev. 3) of SRP 3.7.2, III, 9 - Effects of Parameter Variations on Floor Response Spectra, which states that "Among the various structural parameters analyzed, the effect of potential concrete cracking on structural stiffness should be addressed."

Based on the above factors and their considerations, the approach presented in ASCE Standard 4-98 is believed to be technically correct and appropriate. Therefore, the approach presented in ASCE Standard 4-98 to determine the effective stiffness of concrete members for use in seismic analysis of reinforced concrete members at NPPs should be followed.

<u>Modeling of Mass</u>

Section 3.4.1 of the Standard provides procedures for modeling mass in the mathematical model. It requires the consideration of the weight of the structure, weight of permanent equipment, and expected live load, not less than 25% of the specified design live loads. For snow load, the Standard indicates that design snow loads of 30 psf (1.44 kN/m^2) or less need not be included. The basis for permitting snow loads less than 30 psf (1.44 kN/m^2) to be ignored when computing seismic mass is not provided in the Standard. However, the consideration of snow loads greater than 30 psf (1.44 kN/m^2) is described in Section 12.7.2 of ASCE Standard 7-05, where it is stated that where the flat roof snow load exceeds 30 psf (1.44 kN/m^2), 20 percent of the uniform design snow load shall be included in calculating W, regardless of roof slope. W, which represents the total dead load and applicable portions of other loads, is used to determine the seismic base shear load. Section 12.14.8.1 of the simplified seismic design criteria in ASCE Standard 7-05 contains precisely the same requirement.

Per the International Building Code (IBC 2003), no discussion of the mass to be included in a mathematical model of a structure for seismic analysis could be identified. The subject of eliminating the snow load for 30 psf (1.44 kN/m^2) or less does appear in Section 1605.3; however, it is only discussed in the context of load combinations using the allowable stress design approach and not for developing a seismic mathematical model. For load combinations using the strength design or load and resistance factor design method, 20% of the snow load is considered along with the seismic load regardless of the magnitude of the snow load.

On the basis of the above discussion, it is recommended that the contribution of snow, to the total mass used in developing a mathematical model for seismic analysis, be based on 25% of the roof snow load, regardless of the magnitude of the snow load.

Damping Values for SSCs

Section 3.4.3 of the Standard provides the damping values to be used in linear elastic analyses for determining the seismic induced loads for SSCs. The damping values are tabulated in Table 3-2 of the Standard as a function of the Response Level of the seismic load resisting elements. The Response Level is defined in the Standard in terms of the demand-to-capacity ratio. For seismic loads corresponding to the design response spectra equivalent to the SSE, the resulting demand-to-capacity ratio of approximately 0.5 to 1.0 is expected for SDC-5 and Limit State D (essentially elastic response). For a demand-to-capacity ratio of 0.5 to 1.0, Table 3-3 of the ASCE Standard indicates that Response Level 2 should be utilized. The use of Response Level 2 is also confirmed by Table 3-4 of the Standard, which indicates that for Limit State D, the maximum Response Level to use is 2. Using Response Level 2, most of the damping values presented in Table 3-2 of the Standard are identical or very close to the values tabulated in NRC RG 1.61. Several exceptions exist for piping, cable trays, electrical cabinets and other equipment.

Since the publication of RG 1.61 in 1973, additional test data and updated information on damping values have been developed. The NRC is currently in the process of revising RG 1.61 to reflect the updated data. BNL has been supporting the NRC staff in preparing a draft of RG 1.61. Based on a recommendations made in a BNL Technical Report (No. L-1106-11/95) and subsequent discussions with the NRC, the only significant difference in damping values would occur for piping. ASCE 43-05 specifies uniform 5% damping, regardless of system frequency and method of analysis (response spectrum analysis (RSA) and time history analysis). BNL's recommendation was to include in RG 1.61 (a) ASME Code Case N-411 damping for RSA only, and (b) 3% for OBE >1/3 SSE / 4% for SSE, for time history analysis or RSA. In earlier revisions of RG 1.84, NRC accepted Code Case N-411 damping (5% up to 10 Hz, decreasing from 5% to 2% at 20 Hz, 2% above 20 Hz) for RSA only. This code case has been annulled by ASME, and consequently is not in the current RG 1.84, Rev. 33, Table 2, which lists acceptable code cases. However, Code Case N-411 does appear in RG 1.84, Rev. 33, Table 4 - Annulled Conditionally Acceptable Code Cases, where 5 conditions for its use are specified. Therefore, it is recommended that the BNL proposal described above be followed for piping. If the ASME Code Case N-411 damping for RSA is utilized, the conditions listed in Table 4 of RG 1.84, Rev. 33 must be satisfied.

For cable trays, Table 3-2 of the Standard permits the use of 10% damping for cable trays that are 50% or more full and 7% damping for other cable trays, cable trays with rigid fireproofing, and conduits. RG 1.61 does not specifically address cable trays or conduits. However, BNL Technical Report (No. L-1106-11/95) recommends that 10% damping be used for full cable trays (not 50% or more full as in the Standard) and 7% damping be used for empty and sprayed-on fire retardant cable trays. For conduit systems, the BNL Technical Report recommends the use of 7% for maximum cable fill and 5% for the empty condition. For cable trays and conduit that are full, the damping values in the Standard are the same as those recommended in the BNL Technical Report. For cable trays and conduits that are not full, the lower damping values recommended in the BNL Report should be used until the revised RG 1.61 is issued.

For electrical cabinets and other equipment, Table 3-2 of the Standard permits the use of 4% damping. RG 1.61 does not specifically address electrical equipment; instead it indicates that damping for equipment and large-diameter piping systems use 3% damping. The BNL Technical Report does not address electrical equipment or other equipment. Therefore, until the revised RG 1.61 is issued, it is recommended that the current RG 1.61 damping values be used.

One component that is not included in Table 3-2 and RG 1.61 is HVAC duct systems. Based on the BNL Technical Report, recommended damping values for duct systems is a function of the type of duct construction. Recommended values are 10% for pocket lock, 7% for companion angle, and 4% for welded type of duct construction.

4.2 Structural Capacity

Section 4.0 of the Standard provides procedures for evaluation of structural capacity. This section of the Standard begins with a description of structural systems and then identifies acceptable structural systems and prohibited structural systems at nuclear facilities. Section 4.1.3 of the Standard states that structural systems specifically prohibited for use in the design of the lateral-force-resisting system of nuclear facilities include, among others, unreinforced masonry systems. Although this statement is acceptable, the use of unreinforced masonry walls should be prohibited not only from use in lateral-force-resisting systems, but from use in all safety-related areas at NPPs. This recommendation is supported by NRC IE Bulletin No. 80-11 on masonry wall design and NRC Information Notice No. 87-67 which describes lessons learned from inspections of licensee actions in response to IE Bulletin No. 80-11.

For determining structural capacities, the Standard indicates that either the strength design approach or allowable stress design levels amplified to strength design amplitudes shall be used. The structural capacities are determined from ACI Standard 349, the AISC Specification, or AISC Standard N690 (and supplement). For steel structures based on allowable stress design, the allowable stresses must be scaled upward (Section 4.2.4 of the Standard) to allow strength-based criteria (AISC LRFD/AISC N690)[12] to be applied. This requirement is important to allow safety checking for seismic effects to be performed on a limit load basis which is fundamental to modern earthquake performance assessment.

The structural capacities for steel and reinforced concrete structures in Section 4 appear to be consistent, for the most part, with the requirements in Standard Review Plans 3.8.3 and 3.8.4, NRC RGs, and current staff positions, which rely on the ACI Standard 349 and AISC N690 Specification. However, there are a number of other codes and specifications that are referred to in the Standard which have not been officially endorsed by the NRC. These include the AISC Load and Resistance Factor Design (LRFD) Manual of Steel Construction, AISC 341-02 Seismic Provisions for Structural Steel Buildings, International Building Code (IBC), and ACI

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[12] The AISC has recently (2005) issued a new combined Specification that includes design criteria for both LRFD and ASD. The LRFD check presumes that the load analysis is performed using "factored" loads (consistent with ASCE Standard 7-05, § 2.3) while the ASD check presumes that the loads are unfactored (ASCE 7-05, § 2.4). The equations that define the nominal strength are the same in both cases. A new and consistent edition of AISC Specification N690 currently is being balloted, and will be issued later in 2006.

530 Building Code Requirements for Masonry Structures. The acceptance of these other referenced codes and specifications, as well as the appropriate editions, need to be reviewed on a case-by-case basis.

One exception to the use of an accepted industry code is found in Section 4.2.3 of the Standard related to capacity of low-rise reinforced concrete shear walls. This exception is discussed in detail in Section 4.3 and Appendix A of this report.

4.3 Comparison of ASCE/SEI Standard 43-05 and ACI Standard 349 for Design of Low-Rise Reinforced Concrete Shear Walls

The ACI provisions for determining reinforced concrete shear wall strength are known to be conservative. Therefore, Section 4.2.2 of the Standard states that in lieu of the ACI provisions, the low-rise shear wall capacity may be determined using the procedures and Eq. 4-3 presented in Section 4.2.3 of the Standard. Eq. 4-3 was developed in a research program (Barda, Hansen and Corley, 1977) to determine capacity of low-rise shear walls in which the height/length ratio $h_w/l_w \leq 1.0$. The Standard extends the applicability of this equation to situations in which $h_w/l_w \leq 2$, but limits the total nominal shear stress to $20\ \phi\ f'_c{}^{1/2}$, in which ϕ = 0.8. This extension is based on a study conducted for the Diablo Canyon NPP. The resistance factor ϕ = 0.8 is presumed to place the design strength at one standard deviation below the Barda, et al. equation. Other resistance factors in ACI Standards have not been selected on this basis; rather, they are based on a perception of the mode of failure (achieving the limit state) and the seriousness of its consequences. While it is reasonable to permit an increase above traditional ACI limits in determining the capacity of low-rise concrete shear walls, further investigation is required as to the use of Eq. 4-3 for this purpose.

In an effort to examine the differences in the procedures prescribed in ASCE 43-05 and the currently NRC accepted procedures, a study was performed for a representative low-rise shear wall. The procedures presented in ASCE 43-05 were compared to the provisions contained in ACI 349-01. Details of the study are provided in Appendix A of this report where the shear wall capacity is calculated using Chapters 11 and 21 of ACI 349-01 and is compared to the capacity determined using the procedures in Section 4.2.3 of the Standard. The results of this study show that the calculated nominal shear load capacity using ASCE 43-05 is about 45% higher than the capacity obtained using Chapters 11 and 21 of ACI Standard 349. While this study was performed for a single representative configuration and design parameters and, as such, is not sufficient to make generic conclusions, it does show that the use of ASCE 43-05 procedures can lead to much higher shear wall capacities. For reasons described above and as noted in Appendix A, it is recommended that the NRC should not generically endorse the (alternative) procedures presented in Section 4.2.3 of ASCE 43-05 for design of low-rise concrete shear walls until a consensus on this approach is developed in the concrete design community and the ACI Standard 349 is revised to reflect this change. This recommendation does not imply that this approach is unsuitable for seismic margins or fragility calculations.

4.4 Acceptance Criteria

Load Combinations

Section 5.1 of the Standard defines the load combinations containing seismic loads and makes reference to the project design criteria for other load combinations. Nuclear plant structures are designed using elastic analysis. For strength-based or deformation-based acceptance criteria verified by elastic analysis, the Standard indicates that the total demand is determined as the sum (combination) of non-seismic demand, D_{NS}, and seismic demand, D_S (Eqs. 5-1(a) and 5-1(b)). The non-seismic demand represents *mean* effects of dead, live, equipment, fluid, snow and at-rest lateral soil loads. The seismic demand is calculated from the DBE in Section 2 (of the Standard); the appropriate probability of exceedance is taken into account in developing the DBE, and thus the load factor is 1.0. The inelastic energy absorption factor, F_μ, in Table 5-1 would equal unity in all cases because inelastic response is essentially precluded in NPP design. The use of $F_\mu = 1.0$ is also stated in the footnote to Table 5-1 of the Standard for Limit State D (which would be applicable to NPPs).

The load combination requirements differ slightly from those in Standard Review Plan (SRP) sections 3.8.3 and 3.8.4. The governing load combinations for steel and reinforced concrete structures in the SRP are,

$$D + L + T_o + R_o + SSE$$

$$D + L + (T_a + R_a + P_a) + (Y_r + Y_j + Y_m) + SSE$$

where, D and L are dead and live loads, respectively; T_a, R_a, and P_a are thermal loads under thermal conditions, reaction loads under thermal conditions, and pressure load, respectively, all associated with postulated pipe break; Y_r is the reaction load generated by a high energy pipe break; and Y_j, and Y_m are jet impingement loads and missile impact loads generated by the pipe break.

In the Standard, the non-seismic demand, D_{NS}, represents *mean* effects (5.1.2.1, p. 16; italics my emphasis). While the means of the dead, temperature, pipe reaction and pressure terms in these equations are close to the nominal values, the mean live load is on the order of 70% of the nominal live load (Hwang, et al., 1983). Whether this difference is of any practical significance will depend on the portion of the structure being designed.

Acceptance Criteria

Acceptance criteria for strength requirements and deformation requirements are presented in Section 5.2 of the Standard. When performing a linear analysis (which is the case for NPPs), the strength acceptance criteria requires the total demand for a given element to be less than or equal to the capacity. For deformation acceptance criteria, the total story drift ratio for each story shall be less than the allowable values presented in Table 5-2 of the Standard.

Section C5.2.3.1 of the Standard indicates that these drift limits were based on FEMA 273 (1997) and NUREG/CR-6104. Since FEMA 273 has been superseded by FEMA 356 (2000), FEMA 356 was reviewed. For concrete walls under the highest structural performance level corresponding to "Immediate Occupancy S-1," FEMA 356 indicates that the drift is 0.005 for transient drift conditions and negligible permanent drift. For concrete frames, the drift is 0.01 transient and negligible permanent drift. For steel moment frames, the drift is 0.007 transient and negligible permanent. For braced steel frames, the drift is 0.005 transient and negligible permanent. Therefore, the drift limits specified in Table 5-2 of the Standard are equal to or somewhat less (i.e., more conservative) than the limits presented in FEMA 356. However, it should be noted that a footnote in the FEMA table indicates that the tabulated values are not intended to be used as acceptance criteria but rather they are indicative of the range of drift that typical structures may undergo when responding within the various structural performance levels. The footnote also states that "drift control of a rehabilitated structure may often be governed by the requirements to protect nonstructural components." This is particularly true for NPPs which have various systems (e.g., piping) attached to building structures which may be affected by significant building deformations. Also, large building deformations may exceed available gaps between adjacent buildings due to seismic excitation. Therefore, it is recommended that the deformation acceptance criteria in the Standard, which are used in addition to the strength acceptance criteria, be accepted; however, a review at each NPP should be made to verify that these deformation limits are sufficient to preclude the attached safety-related systems from meeting their intended function and also will not lead to impact loads between adjacent structures.

4.5 Special Requirements

<u>Ductile Detailing</u>

Section 6 of the Standard provides guidelines for providing sufficient ductility in the materials, connections, and anchorages, as well as consideration for irregularities, strength distribution, redundancy, and seismic interactions. More detailed provisions are specified in Section 6.1 of the Standard for steel structures and Section 6.2 for reinforced concrete structures. Some of the provisions provide measures to ensure sufficient ductility is present based on the new information obtained as a result of the Northridge, California earthquake of January, 1994, and so these provisions should provide an improvement from past design practices.

In the case of concrete structures, the Standard states that they shall meet the detailing requirements in ACI 349, including Chapter 21. Six specific requirements applicable to concrete and reinforcing steel are listed. The use of ACI 349 detailing requirements are acceptable on the basis of Regulatory Guide 1.199, Nov. 2003. This Regulatory Guide, however, endorses ACI 349-97 subject to the Regulatory Positions stated within the guide. Some of the six specific requirements listed in Section 6.2.1 have already been included in ACI 349 (Chapter 21 - Special Provisions for Seismic Design).

Section 6.3 of the Standard specifically addresses anchorages in concrete. Recommended anchorages are cast-in-place bolts or headed studs, undercut-type expansion anchors, or welding to embedded plates. For SDC-5, other types of expansion anchors are not recommended for supports subject to vibration, very heavy equipment, or for sustained tension supports. Epoxy grouted anchorages are not permitted in applications involving elevated temperature, radiation, or overhead installation; nor are they to be used without documentation following ACI 349. All of these provisions are considered to be acceptable and should be followed. However, based on prior studies and tests, some questions were raised regarding the design methodology used in versions of Appendix B to ACI Standard 349 prior to 2001. These concerns were addressed in Appendix B to ACI 349-01 (2001). Therefore, the current version endorsed by the NRC for design of anchorages is Appendix B to ACI 349-01, subject to the regulatory positions delineated in Regulatory Guide 1.199, Nov. 2003.

Rocking and Sliding

Section 7.1 of the Standard indicates that it is generally preferable to anchor components so as to prevent rocking and sliding. However, it does permit rocking and sliding of unanchored rigid bodies, and provides procedures for such analyses in Section 7.1 and Appendix A of the Standard. Section 7.2 of the Standard provides analytical methods for analyzing building sliding and overturning. With rare exceptions, the NRC regulatory guidance documents do not permit unanchored safety-related components at NPPs. Notable exceptions are certain fuel rack designs in spent fuel pools and building structures resting on soil. Since free standing fuel racks are immersed in water, the analytical methods in the Standard would not apply. Fuel racks are typically analyzed using nonlinear time history analyses which are always reviewed on a case-by-case basis by the NRC staff. For buildings, rocking and sliding criteria are provided in the NRC Standard Review Plan 3.8.5 which addresses building foundations. This SRP section provides the loads, load combinations, acceptance criteria and other guidance. However, it does not discuss simplified analytical methods for calculating rocking and sliding comparable to those in the Standard. Therefore, this may be an area that warrants further detailed review if the methods in Section 7.1 and Appendix A are to be used in future NPP designs. For the time being, the application of such methods should be reviewed on a case-by-case basis.

Seismic Separation

The provisions in Section 7.3 of the Standard specify that minimum separation between SSCs shall be two times the square root of the sum of the squares (SRSS) of the elastically calculated displacements. According to the Commentary of the Standard, the SRSS method is used assuming that the displacements of adjacent structures are randomly phased relative to each other. The factor of two is used to provide less than a 10% chance of significant impact between adjacent structures for an input ground motion of 1.5 times the DBE. This would preclude the need to use multiple nonlinear time-history evaluations for input ground motion of 1.5 times the DBE. No quantitative criteria are contained in the NRC SRP regarding seismic separation. However, for NPPs the use of two times the SRSS of the displacements of the adjacent structures is considered to be conservative when compared to the absolute sum of the two displacement values, and thus is acceptable.

5 QUALIFICATION OF EQUIPMENT AND DISTRIBUTION SYSTEMS

ASCE/SEI Standard 43-05 permits seismic qualification of equipment and distribution systems by equivalent static or dynamic analysis, testing, past earthquake experience, or generic test data. The seismic demand for equipment and distribution systems is defined by the seismic in-structure spectra or time history.

5.1 Qualification by Analysis

Analysis Methods

Seismic qualification by analysis is described in Section 8.2 of the Standard. According to the Standard, equivalent static methods shall utilize the peak of the in-structure response spectra. Seismic analysis procedures shall meet the requirements of Section 3.2.5 of ASCE Standard 4. For single point of attachment cantilever components, Section 3.2.1 of ASCE 43-05 also shall be met. For piping system analysis, paragraph N-1225 of the ASME BPVC, Section III, Div. I, Appendix N shall be followed. Effects of support displacements for multiple supported equipment or distribution systems, due to seismic anchor motions (SAM), shall meet Section 3.2.6 of ASCE Standard 4. For piping systems, the effects of SAM shall meet paragraph N-1225 of the ASME BPVC, Section III, Div. I, Appendix N.

The analysis methods described in Section 3.2.5 of ASCE Standard 4-98, which are referenced in the Standard, are basically the same as those described in Section 3.2.1 of the Standard. These analysis methods were evaluated and discussed in Section 4.1.3 of this report. In general, the analytical methods were found to be acceptable except for the third method evaluated in Section 4.1.3, where the natural frequency of the SSC is calculated and the spectral acceleration value at the fundamental frequency is used (times an appropriate factor).

For the case of piping systems, where the provisions in the Standard refer to Appendix N of the ASME Boiler and Pressure Vessel Code (B&PVC), the NRC has not officially accepted Appendix N in any regulatory guidance document. Therefore, it is recommended that the existing methods described in the NRC SRP and RGs be used; otherwise, the use of Appendix N by a licensee will need to be reviewed on a case-by-case-basis.

For dynamic analysis methods, the Standard refers to the following Codes or standards:

ASME B&PVC, Section III, Appendix N
ASCE Standard 4
IEEE Std 344
ASME QME-1

The use of the ASME B&PVC, Appendix N has already been discussed above, under the subject of equivalent static analysis methods, and the stated recommendation would also apply here. Dynamic analysis methods described in ASCE Standard 4 were previously evaluated in Section 4.1.4 of this report. The use of the linear dynamic analysis methods in ASCE Standard 4 are acceptable provided that the specific provisions in ASCE Standard 4 that are followed are consistent or conservative when compared to the requirements in the SRP and applicable RGs. Any deviations from the NRC SRP or RGs must be reviewed on a case-by-case basis.

Dynamic analysis methods performed in accordance with IEEE Std 344-1987 are acceptable as they have been endorsed by RG 1.100, Rev. 2 for electrical and mechanical equipment. RG 1.100 is being revised by the NRC to reference the upcoming revision to the IEEE Std 344-2004. Therefore, if the current IEEE Std 344-2004 is utilized, then its use should be reviewed on a case-by-case basis until the revised RG 1.100 is issued. ASME QME-1 (current version 2002) describes requirements for qualification of active mechanical equipment in NPPs such as active valves and pumps. This standard has not been endorsed by the NRC; however, RG 1.100 is also being revised by the NRC to reference the upcoming revision to the Standard QME-1. Therefore, until the revised RG 1.100 is issued, the use of QME-1 should be reviewed on a case-by-case basis.

Section 8.1 of the Standard indicates that when demonstrating that the equipment or distribution systems are capable of performing all of their specified safety functions, it must also be verified that the equipment and distribution systems do not interfere with the safety function of adjacent equipment or distribution systems per ANS 2.26. This system interaction effect, which is discussed in ANS 2.26, is also referred to as “two-over-one phenomenon” where an SSC may not perform a safety function by itself, but its failure may adversely affect the safety function of another SSC. As noted in Section 2.3 of this report, ANS 2.26 specifically indicates that it is applicable to nuclear facilities other than commercial power reactors. Therefore, ANS 2.26 is not applicable to NPPs and should not be utilized. Instead, guidance for ensuring interaction between adjacent SSCs are presented in SRP Section 3.7.2 and 3.7.3, subsections II.8. These subsections describe several acceptable methods for considering interaction between non-safety-related SSCs and safety-related SSCs.

Demand

The provisions in the Standard for determining the demand for equipment and distribution systems utilize the in-structure response spectra or time histories, analysis methods described above, damping values specified in Table 3-2 of the Standard, and inelastic energy absorption factor (F_μ) contained in Table 8-1 of the Standard. The evaluation of the damping values were already discussed in Section 4.1.6 of this report. The inelastic energy absorption factor, F_μ, must be set to 1.0 because essentially inelastic analysis is precluded for SSCs at NPPs.

Capacity

Although it appears that the emphasis of the Standard is on structures, Table 8-1 implies that the Standard can also be used for seismic qualification of many different types of equipment and distributions systems as well. These consist of vessels, heat exchangers, coolers, chillers, tanks, pumps, fans, valves, dampers, filters, electrical boards, electrical racks, electrical cabinets, piping systems, conduit systems, cable tray systems, HVAC duct systems, instrument tubing systems, and equipment supports. According to Section 8.2.3 of the Standard, the capacity (using qualification by analysis) for these equipment and distribution systems is based on the stress or load limits specified in the appropriate code or standard for the equipment or distribution system. Typical codes and standards used for mechanical and electrical equipment are tabulated in Table C8-1 of the Commentary to the Standard. While in principle, this approach makes sense, there does not appear to be sufficient information in the Standard to clearly identify how to determine the capacity when using the “appropriate code or standard.” There are many different industry codes and standards listed in Table C8-1, and a comprehensive review of their application is beyond the scope of this study. In addition, there are a number of unique requirements for equipment and distribution systems at NPPs that may not be captured based on only meeting the provisions in Section 8 for equipment and distribution systems and the corresponding industry code and standards. Examples include the requirements associated with the elimination of the operating basis earthquake (OBE), functionality requirements for active components, and fatigue. In view of the above, it is recommended that, the capacity for equipment and distribution systems be determined using the current accepted methods defined by NRC SRP, RGs, and technical positions, rather than the provisions of Section 8.2.3 of the Standard.

<u>Load Combination and Acceptance Criteria</u>

The load combination and acceptance criteria for equipment and distribution systems are basically the same as for structures. The total demand is determined as the sum (combination) of non-seismic demand, D_{NS}, and seismic demand, $D_{S,}$ as shown in Eq. 8-1 of the Standard. This equation is identical to Eq. 5-1 (of the Standard) for structures. The non-seismic demand represents dead weight, fluid pressure, and other loads as defined in the industry codes or standards. The inelastic energy absorption factor, F_{μ}, would equal unity in all cases because essentially inelastic analysis is precluded in NPP design. The acceptance criteria are defined as demonstrating that the demand is less than or equal to the capacity. These are considered to be acceptable.

5.2 Qualification by Testing and Experience Data

The scope of NPP mechanical and electrical equipment subject to seismic and dynamic qualification is defined in SRP Section 3.10 and the referenced RG 1.100. The April 1996 draft Rev.3 of SRP Section 3.10 II. - Acceptance Criteria, specifies ANSI/IEEE Std 344-1987, as endorsed by RG 1.100, for qualification of both electrical equipment and their supports, and mechanical equipment to the extent practical. This SRP 3.10 subsection contains very specific guidance related to test details, special considerations for NPP equipment (e.g., reference to IEEE Std 323-1974 for environmental aging requirements prior to dynamic testing), and documentation requirements. Seismic qualification test methods performed in accordance with IEEE Std 344-1987 are acceptable as they have been endorsed by RG 1.100, Rev. 2 for electrical and mechanical equipment. As noted earlier, RG 1.100 is being revised by the NRC to reference the upcoming revision to the IEEE Std 344-2004. Therefore, if the current IEEE Std 344-2004 is utilized, then its use should be reviewed on a case-by-case basis until the revised RG 1.100 is issued.

Section 8.0 of ASCE 43-05 provides a broad framework that for the most part is consistent with SRP 3.10, but lacks the NPP-specific guidance contained in SRP 3.10. The long history of seismic/dynamic equipment qualification by testing in the nuclear power industry places it at the forefront of knowledge and experience in this technical area. Section 8.0 of the Standard draws heavily on this knowledge and experience, but has homogenized it to fit the graded approach.

The approach for qualification by experience data described in the Standard appears to be similar to the approach developed under the Unresolved Safety Issue (USI) A-46 program, which was used to verify the seismic adequacy of existing equipment installed in older (construction permit docketed before 1972) NPPs. At this time, the NRC has not endorsed this approach for seismic qualification of equipment for use at new NPPs. Currently, SRP 3.10

does not address the use of test experience data and earthquake experience data. The recently released IEEE Std 344-2004 includes a provision (Clause 10) for qualification of new equipment installations based on test experience data or earthquake experience data. It is recommended that the new revision to RG 1.100 include regulatory positions on the use of IEEE Std 344-2004, including the use of test experience data and earthquake experience data. Meanwhile, it is recommended that the existing NRC regulatory guidance documents as described in SRP 3.10 and RG 1.100 be utilized.

When determining the demand for qualification by test, Section 8.3.2 provides the following expression for total demand:

$$D = D_{NS} + 1.4\ D_S \qquad \text{(Eq. 8-3 of the Standard)}$$

where D_{NS} and D_S are the non-seismic and seismic demand as explained earlier. The 1.4 factor is the equipment capacity factor for qualification by test that provides the margin to obtain the required confidence level of performance. According to the Commentary Section C8.0 of the Standard, this factor is based on a study by Salmon and Kennedy (1994). A review of that study, to determine the applicability of this factor to seismic qualification by tests for use at NPPs, is warranted if the Standard will be used to qualify equipment by testing.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The philosophy governing the seismic provisions in ASCE/SEI Standard 43-05 is different from that in the Standard Review Plan and RG 1.165. Both sets of provisions recognize the need to consider uncertainty in setting ground motion intensity and utilize probabilistic methods in addressing uncertainty. RG 1.165 stipulates a probabilistic measure for the seismic hazard intensity - median probability of 10^{-5}/yr on the SSE - but is silent on what is the probability of failing to achieve the design objective. In a sense, this is analogous to traditional practice in civil building construction, where the wind and earthquake demands on a structure have been specified in terms of a return period, but (until recently, with LRFD) the limit state probability of structural members and systems remains unknown. In contrast, the starting point with ASCE/SEI Standard 43-05 is a Target Performance Goal expressed as a mean limit state probability. Since the limit state probability depends on uncertainties in both the seismic hazard (demand) and fragility (capacity) of structures, systems and components in a fully coupled reliability analysis [Eq. (3.1)], one would need to stipulate both a design-basis demand and capacity to meet that performance goal. The Standard presumes that other nationally recognized codes, standards and guidance documents will be used in design; thus, emphasis was placed on determining the appropriate seismic demand. Through the convolution in Eq. (3.1), the performance-based approach reflects the potential impact of all seismic events, including those beyond the design basis, on the structural performance, something that the current RG 1.165 does not do.

With properly stipulated Performance Goals and supporting criteria, the approaches in ASCE 43-05 and in the SRP/RG 1.165 both can provide acceptable levels of protection against severe low-probability earthquakes. On the other hand, it is not possible to achieve consistent results in all cases using the two approaches in parallel because their fundamental bases are different.[13] Nor should such consistency be expected.

The goal of ASCE 43-05 - to provide seismic design criteria that ensure acceptable performance of structures, systems and components with regard to public safety and environmental protection - is aligned with the regulatory goal of the NRC. For NPP SSCs, the limit state is defined by the onset of inelastic deformation (or FOSID, as defined previously in Section 3.3) in Seismic Design Category 5. The Seismic Design Basis is 5D and Target Performance Goal (expressed in terms of annual limit state probability) is 10^{-5}/yr. The procedure in ASCE 43-05 is essentially the same as that taken in DOE Standard 1020-2002, although the numbers are different in some instances. Moreover, it is consistent with the approach being recommended for seismic performance assessment in Applied Technology Council Project 58[14] currently in progress under FEMA sponsorship. Finally, the fundamental reliability-based approach, in which the demands and capacities are fully coupled in the

[13] When the transition from allowable stress design to limit states design occurred, the resulting differences in structural designs were difficult for many engineers to understand or accept. Acceptance has grown with education and experience.

[14] ATC Project 58, Guidelines for Seismic Performance Assessment of Buildings, is aimed at developing procedures and criteria that can be used to predict the probable earthquake performance of buildings.

performance assessment, is identical to that taken in developing the first generation of probabilistic codified design procedures in the United States (Ellingwood, et al., 1982). In that process, a performance objective first was stated in probabilistic terms.[15] A set of load and resistance factors then was determined to meet the performance using a constrained optimization procedure. These load and resistance factors accounted for the fact that the nominal loads and resistances may (or may not) be specified probabilistically. The process leading to the Design Factor, DF for a stipulated $R_P = H_D/P_F$ in Section 2 of the Standard, as explained in the Commentary, is consistent with this approach.[16] This approach recognizes that it is virtually impossible to achieve a consistent level of risk across an inventory of structures using a design approach in which the stipulation of the hazard and structural capacity for design purposes effectively uncouples the two uncertain quantities.

The NRC has adopted qualitative and quantitative safety goals for nuclear power plants in its Policy Statement on "Safety Goals for the Operations of Nuclear Power Plants," (51 FR 28044, 8/4/86). SRM SECY 90-016 supports a subsidiary safety goal of 10^{-4}/yr or less on Core Damage Frequency (CDF) and 10^{-6}/yr on Large Release Frequency (LRF), considering both internal and external events. The SRM also indicates that the Commission strongly supports the use of the information and experience gained from the current generation of reactors as a basis for improving the safety performance of new designs. The relationship of the Performance Goals in ASCE 43-05 to the CDF and LRF subsidiary safety goals has not been established, and would require a major research effort. The fact that such a relationship has not been established is not viewed as a barrier to the use of ASCE 43-05 in seismic design. For one thing, the CDF (or LRF) goals involve non-seismic as well as seismic events, and the relative importance of seismic hazard to plant performance varies significantly from plant to plant. Adjusting the Performance Goals in ASCE 43-05 to CDF, specifically when CDF from internal events are considered, might lead to seismic criteria that would be suitable for some plants but not for others, leading to a difficult regulatory situation. For another, it is easier to scrutinize and adjust seismic design criteria that are based on FOSID because the limit states are well-defined, the risk analysis is auditable, and the probabilities can be compared and benchmarked against structural reliability assessments performed for other critical facilities. Finally, there is the issue of validation of the Performance Goal. Inevitably, there is epistemic uncertainty associated with whether goals defined by probabilities on the order of 10^{-4} - 10^{-6}/year are actually achieved. That uncertainty certainly will be less when the structural reliability calculations are based on well-defined principles of mechanics, stipulated probabilistic models of uncertainty, and independently validated finite element software. On the other hand, subsidiary target performance goals for seismic design of SSCs in NPPs in terms of CDF or LRF might be considered as a long-range research goal by the NRC.

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[15] That performance goal was determined through a comprehensive assessment of reliabilities implied by structural design practices at the time (1970's) and a judgment on where to place the reliability target for structural members and connections. Those reliability targets were on the order of 10^{-3} in 50 years for steel or reinforced concrete members and 10^{-4} in 50 years for steel connections. The knowledge of building performance designed by traditional engineering practices made this calibration process possible.

[16] The basis for the limit state probability 10^{-5} that is used as the Performance Goal is less well-defined than it was for ordinary building structures, since the code calibration process followed in the former case could not be applied.

Since SDB-5D (with FOSID limit states) is coupled to a Target Performance Goal of 10^{-5}/year, it is probable that the CDF/LRF for a plant designed by ASCE 43-05 would be substantially less than this value because any postulated behavior of SSC designed by that standard involves inelastic deformations at the state of core damage. Thus, it is probable that the use of ASCE 43-05, with SDB-5D, will lead to SSCs that also meet the safety goal in SRM SECY 90-016. Any concerns in this regard can be addressed by coupling the SRM/SECY 93-087 requirement to the design requirements, as noted below.

6.2 Recommendations

With the resurgence in interest in NPP construction and the relative maturity of probabilistic methods for modeling uncertainty, and the availability of new performance-based guidance, there is an excellent opportunity to re-examine the fundamental bases for NPP design and derive new regulatory guidance to achieve the goals of public safety and environmental protection. This review has examined the philosophies underlying the requirements in ASCE Standard 43-05 and the reliability-based formulations on which its specific provisions are based. On the basis of this review, some observations and specific recommendations have been described within the body of this report. The major findings and recommendations are listed below.

- Subject to the limitations identified in this report, there exists a technical engineering basis for NRC to endorse portions of ASCE Standard 43-05 for application to nuclear power plant design and construction.

- The Seismic Design Basis for nuclear power plant design and construction should be stipulated as SDB-5D, with a Target Performance Goal (limit state probability) of 10^{-5}/yr.

- The Target Performance Goal shall be met by ensuring that there is less than 1% mean probability of unacceptable performance for the SSE ground motion.[17]

- The SRM/SECY 93-087 requirement - that the HCLPF shall be greater than or equal to 1.67 times the SSE in a margin assessment of seismic events - should be uncoupled from the seismic design criteria in ASCE Standard 43-05. However, SRM/SECY 93-087 may be applied, as circumstances warrant, as an independent verification that safety goals related to core damage are met.

- The specific dates or editions of codes and standards referenced in Section 1.2 of ASCE Standard 43-05 should be identified to allow confirmation that these codes and standards have been endorsed by the NRC for the seismic design of NPPs. Otherwise, each use of this Standard will need to be reviewed on a case-by-case basis to ensure the acceptability of the codes and standards utilized.

- The NRC should update its regulatory guidance documents (e.g., 10CFRs, SRPs, and RGs) to reflect the current generation of codes and standards and to develop one

[17] The second criterion in Section 1.3 of ASCE Standard 43-05 (Less than 10% probability of unacceptable performance for a ground motion equal to 150% of the SSE) is unlikely to govern NPP design, but it will have no impact on regulatory guidance since both criteria must be satisfied.

source of regulatory guidance that identifies the endorsement and any special regulatory positions that may apply to codes and standards. This would result in an efficient and productive utilization of up-to-date knowledge, would avoid having to repeatedly review each licensing application for its use of recent codes and standards, and would ensure consistency in the use of referenced codes and standards across the inventory of NPPs.

- Section 4.2.2 of the Standard states that in lieu of the ACI code provisions, the low-rise shear wall capacity of reinforced concrete walls may be determined using the procedures and Eq. 4-3 presented in Section 4.2.3 of the Standard. While it is reasonable to permit an increase above traditional ACI limits in determining the capacity of low-rise concrete shear walls, further investigation is warranted before this approach is accepted for design purposes.

- When performing qualification by analysis of equipment and distribution systems, the capacity for equipment and distribution systems should be determined using the current accepted methods defined by NRC SRP, RGs, and technical positions, rather than the provisions of Section 8.2.3 of the Standard.

- The NRC should complete the revision of RG 1.100 to present the regulatory positions on the use of IEEE Standard 344-2004, including the use of test experience data and earthquake experience data.

- The NRC should develop a technical position on the use of ASCE Standard 4, because this standard is integrated with ASCE 43-05 to a considerable degree.

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SRM SECY 93-087. Staff Requirement Memorandum, Subject: SECY-93-087 - Policy, Technical, and Licensing Issues Pertaining to Evolutionary and Advanced Light-Water Reactor (ALWR) Designs, U.S. Nuclear Regulatory Commission, July 21, 1993.

APPENDIX A

COMPARISON OF ASCE/SEI STANDARD 43-05 AND ACI STANDARD 349 FOR DESIGN OF LOW-RISE REINFORCED CONCRETE SHEAR WALLS AT NPPS

Introduction

In order to examine the differences in design procedures between ASCE Standard 43-05 and the current regulatory guidance contained in the SRP and Regulatory Guides, a reinforced concrete shear wall was selected for this study. The reinforced concrete shear wall was selected because it is a major structural member used exensively in all NPPs to provide seismic load resisting capability. In addition, the low-rise shear wall was selected because ASCE Standard 43-05 permits as an alternative, the use of a different formulation than that presented in ACI 349. The ACI Standard 349-97, subject to certain regulatory positions, is currently endorsed by the NRC Regulatory Guide 1.142, Rev. 2, 2001.

For this study, a comparison was made between the ASCE Standard 43-05 alternative method and the ACI 349-01 (current edition of ACI 349) approach. First, the design equations contained in ACI 349-01 were presented followed by the alternative design equations given in ASCE 43-05. Then a representative low-rise shear wall, having a wall configuration typically found at NPPs, was selected and the shear capacity for this wall was determined using ACI 349-01 and ASCE 43-05 design methods.

In the following sections, US Units (in, lbf, psi) are utilized in the calculation in order to facilitate the use of the ACI 349 equations. These values are presented in US Units followed by SI Metric units in parenthesis.

A1.0 ACI 349-01 Design Method

A1.1 Description of Methodology

A1.1.1 Based on ACI 349-01, Chapter 11 - Shear and Torsion

For Concrete

Per ACI 349-01 (as well as ACI 318-02), Section 11.10 - Special Provisions for Walls, the nominal shear strength V_c provided by the concrete is computed by the smaller value obtained from Eqs. (A1-1) and (A1-2).

$$V_c = 3.3\sqrt{f_c'}hd + \frac{N_u d}{4l_w} \qquad \text{(A1-1)}$$

$$V_c = \left(0.6\sqrt{f_c'} + \frac{l_w\left(1.25\sqrt{f_c'} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right) hd \qquad \text{(A1-2)}$$

where,

f'_c = compressive strength of concrete

h = wall thickness

l_w = length of wall

d = 0.8 x l_w (per ACI 349-01, Section 11.10.4). A larger value, corresponding to the distance from the extreme compression fiber to the centroid of force of all reinforcement in tension when determined by a strain compatibility analysis is permitted.

N_u = factored axial load normal to cross section

M_u = factored moment at section

Vu = factored shear force at section

When the expression

$$\frac{M_u}{V_u} - \frac{l_w}{2} \qquad \text{(A1-3)}$$

is negative, then Eq. (A1-2) shall not be used.

Eq. (A1-1) determines the inclined cracking strength corresponding to a principal tensile stress of approximately $4\sqrt{f_c'}$. Eq. (A1-2) calculates the shear strength corresponding to a flexural tensile stress of $6\sqrt{f_c'}$ at a section $\frac{l_w}{2}$ above the section being investigated. Also, in accordance with Section 11.10.7 of ACI 349-01, the critical section to be used in design for shear is a distance equal to the smaller of one-half of the wall length or one-half of the wall height.

<u>For Steel Reinforcement</u>

Per ACI 349-01 (and ACI 318-02), Section 11.10.9.1, the nominal shear strength V_s provided by the steel reinforcement shall be computed by

$$V_s = \frac{A_v f_y d}{s_2} \qquad \text{(A1-4)}$$

where,

A_v = area of horizontal shear reinforcement within distance s2

s_2 = spacing of horizontal reinforcement

f_y = yield strength of reinforcement

The above equation is based on the horizontal shear reinforcement. A separate check on the vertical shear reinforcement is required. Per ACI 349-01, Section 11.10.9.4, the ratio ρ_n, which represents the vertical shear reinforcement area divided by the gross concrete area of horizontal section, shall not be less than

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_h - 0.0025) \qquad \text{(A1-5)}$$

nor 0.0025, but does not need to be greater than the required horizontal shear reinforcement ratio.

Total Shear Strength

The total nominal shear strength is given by

$$V_n = V_c + V_s \qquad \text{(A1-6a)}$$

The total shear strength for design is given by

$$\phi V_n \qquad \text{(A1-6b)}$$

where the strength reduction factor (ϕ) for shear is taken as 0.6 per Section 9.3.4 of ACI 318-02 (more recent criteria than ACI 349-01).

In addition to the above provisions, ACI 349-01, Section 11.10.3, indicates that the nominal shear strength must satisfy the following expression

$$V_n < 10\sqrt{f_c'}hd \qquad \text{(A1-7)}$$

A1.1.2 Based on ACI 349-01, Chapter 21 - Special Provisions for Seismic Design

ACI 349-01, Section 21.6 - Structural Walls, Diaphragms, and Trusses, provides requirements that apply to structural walls that are part of a seismic lateral load resisting system. Section 21.6.1 indicates that for shear walls with height to length ratios less than 2.0, the provisions of 21.6.5 - Shear Strength, can be waived. However, for purposes of comparison with Chapter 11 of ACI 349, the nominal shear strength provided by Section 21.6.5 was calculated. According to

Section 21.6.5, the nominal shear strength V_n can be determined from Eq. (A1-8) for $\frac{h_w}{l_w}$ ratios less than 2.0.

$$V_n = A_{cv}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right) \tag{A1-8}$$

where,

α_c varies linearly from 3.0 for $\frac{h_w}{l_w}$ = 1.5 to 2.0 for $\frac{h_w}{l_w}$ = 2.0

$A_{cv} =$ the net cross-sectional area of the horizontal wall segment.

$\rho_n =$ the ratio of shear reinforcement on a plane perpendicular to the plane of A_{cv}.

Also, per ACI 349-01, Section 21.6.5.7,

$$V_n < 10A_{cp}\sqrt{f_c'} \tag{A1-9}$$

where,

$A_{cv} =$ the cross-sectional area of a horizontal wall segment.

A1.2 Application of Methodology to Representative Wall

A1.2.1 Based on ACI 349-01, Chapter 11 - Shear and Torsion

The selected representative low-rise concrete shear wall is shown in Fig. 1. This wall was selected on the basis of characteristics of shear walls found at NPPs as described in the ASCE Publication, "Stiffness of Low Rise Reinforced Concrete Shear Walls," (ASCE, 1994). The shear wall selected as being representative, has a height to width ratio equal to one, a thickness of 2 ft (61.0 cm), and a reinforcement ratio equal to 0.003 in each direction.

As shown in Fig. 1, the wall is 20 ft (6.1 m) high by 20 ft (6.1 m) wide and is 2 ft (61.0 cm) thick. The reinforcement consists of #6 bars spaced at 12 in. (30.5 cm) at each face in each direction which results in a horizontal and vertical reinforcement ratio of 0.00306. An axial load in the building is assumed to produce a uniform compressive stress in the wall equal to 300 psi (2.07 Mpa). The concrete strength is taken as 4 ksi (27.6 MPa) and grade 60 reinforcement is used, which are typical values found at NPPs.

$f_c' =$ 4,000 psi (27.6 MPa)

$f_y =$ 60,000 psi (414 MPa)

$h =$ 24.0 in. (60.96 cm)

$l_w =$ 240.0 in. (609.6 cm)

$h_w = 240.0$ in. (609.6 cm)

$d = 0.8 l_w = 192$ in. (487.7 cm)

$p = 300.0$ psi (2.07 MPa)

$N_u = p h l_w =$ 1,728,000 lb (7687 kN)

$s_2 = 12.0$ in. (30.5 cm)

$\rho = 0.00306$

$\rho_h = \rho = 0.00306$

$A_v = \rho_h h s_2 = 0.881$ in.^{2} (568 mm ^{2})

<u>Shear Strength From Concrete</u>

Substituting the design parameters defined above into Eq. (A1-1) results in the shear strength contribution from concrete equal to $V_c =$ 1,307,000 lb (5814 kN).

Using the second term in Eq. (A1-1), the contribution to concrete shear capacity from compression is equal to

$$\frac{N_u d}{4 l_w} = 345{,}600 \text{ lb} \qquad (1537 \text{ kN})$$

To determine whether Eq. (A1-2) should have been used to determine the shear strength from concrete, the expression in Eq. (A1-3) needs to be calculated first. To do this, the critical section for shear based on Section 11.10.7 of ACI 349-01 is determined. The critical section is the smaller of $l_w / 2$ or $h_w / 2$.

$$\frac{l_w}{2} = 120 \text{ in.} \quad (304.8 \text{ cm}) \qquad\qquad \frac{h_w}{2} = 120 \text{ in.} \quad (304.8 \text{ cm})$$

Therefore, the critical section to use for calculating shear strength is located at a height of $h_c = 120$ in (304.8 cm). above the base.

The moment at the critical section is given by the expression

$$M_u = V_u \left(h_w - h_c \right) = 120 \ V_u$$

Substituting M_u into Eq. (A1-3) results in the following

$$120 - \frac{l_w}{2}$$

Since $l_w / 2 = 120$ in. (304.8 cm), this term equals zero and therefore, Eq. (A1-1) (not Eq. (A1-2)) governs the calculation of the concrete shear strength.

Shear Strength From Steel

Substituting the design parameters defined earlier into Eq. (A1-4) results in the shear strength contribution from steel equal to $V_s =$ 846,000 lb (3763 kN).

Using Eq. (A1-5) the required vertical shear reinforcement ratio is calculated to be $\rho_n = 0.00292$.

The vertical shear reinforcement ratio provided is equal to 0.00306 which is greater than the required value calculated above.

Total shear strength

The total nominal shear strength is calculated from Eq. (A1-6a) as the sum of the contribution from concrete and steel.

$$V_n = 1{,}307{,}000 + 846{,}000 = 2{,}153{,}000 \text{ lb} \qquad (9577 \text{ kN})$$

The total shear strength for design is calculated from Eq. (A1-6b) and is equal to

$$= 0.6 \times 2{,}153{,}000 = 1{,}292{,}000 \text{ lb} \qquad (5747 \text{ kN})$$

An additional check is required based on the limitation on the nominal shear strength defined in Section 11.10.7 of ACI 349-01. The limitation given by Eq. (A1-7) is equal to 2,914,000 lb (12,960 kN), and therefore, the nominal shear strength of 2,153,000 lb (9577 kN) satisfies this limitation.

A1.2.2 Based on ACI 349-01, Chapter 21 - Special Provisions for Seismic Design

Using the definition of parameters given in Section A1.1.2,

$$A_{cv} = hl_w = 5760 \text{ in.}^2 \quad (3.716 \text{ m}^2)$$
$$\alpha_c = 3.0 \text{ for } \frac{h_w}{l_w} = 1.0$$
$$\rho_n = 0.00306$$

The total nominal shear strength can then be calculated using Eq. (A1-8) which results in $V_n = 2{,}150{,}000$ lb (9564 kN).

The total shear strength for design is calculated using $\phi = 0.6$ (as was done previously) which results in

= 0.6 x 2,150,000 = 1,290,000 lb (5738 kN)

An additional check is required based on the limitation on the nominal shear strength defined in Section 21.6.5.7 of ACI 349-01. The limitation given by Eq. (A1-9) is equal to 3,643,000 lb (16,200 kN), and therefore, the nominal shear strength of 2,150,000 lb (9564 kN) satisfies this limitation.

A1.2.3 Summary of Shear Wall Strength Based on Chapters 11 and 21 of ACI 349

Based on the previous calculations, the shear strength for design using ACI 349-01 is 1,292 kips (5747 kN) using Chapter 11 versus 1,290 kips (5738 kN) using Chapter 21. These results are very close to one another suggesting that for this wall configuration, there isn't much difference between using Chapters 11 or 21 to design the wall. This close comparison occurred primarily for the following reasons:

	Chapter 11	Chapter 21
1. Coefficient applied to $\sqrt{f_c'}$ term:	3.3	3.0
2. Effective shear area:	d = 0.8 length of wall	d = 1.0 length of wall
3. Contribution from compressive loads	$\frac{N_u d}{4 l_w}$	None
4. Separate check on V_n limit:	Didn't control	Didn't control

For this particular wall configuration, using the factors tabulated above for the Chapter 11 approach, the product of 3.3 x 0.8 (which equals 2.64) is slightly below the Chapter 21 product of 3.0 x 1.0 (which equals 3.0). Then, when the contribution from compressive loads, prescribed in the Chapter 11 approach, are considered the 2.64 factor approaches very close to the 3.0 factor in the Chapter 21 approach.

Even though the results of Chapter 11 and Chapter 21 give comparable results, there will be other configurations where the results between the two methods give different shear strengths. For example, if the compressive stress used in this sample problem was smaller, then Chapter 21 will provide a higher shear capacity; while conversely, if the compressive stress was larger than assumed in this sample problem, then Chapter 11 would provide higher shear capacity.

As noted earlier, according to ACI 349-01, Section 21.6.1 - Scope, for "shear walls with $\frac{h_w}{l_w}$ of less than 2.0, provisions of 21.6.5 can be waived."

A2.0 ASCE/SEI Standard 43-05

A2.1 Description of Methodology

From ASCE Standard 43-05, Section 4.2.3 "Capacity of Low-Rise Concrete Shear Walls," the equation for calculating the shear strength of a low-rise ($\frac{h_w}{l_w} \leq$ 2.0) reinforced concrete shear wall is given by Eq. (A2-1) shown below. This equation is based on the Barda, et al. method described in the Commentary Section C4.2.3 of ASCE 43-05. The ASCE Standard 43-05 methodology is very similar to the Barda, et al. method with some exceptions. The differences primarily relate to the calculation of the A and B constants, limitation on ρ_v and ρ_u, and reduced distance "d" to be used to calculate total shear strength from the unit shear stress v_u.

$$v_u = \phi\left(8.3\sqrt{f_c'} - 3.4\sqrt{f_c'}\left(\frac{h_w}{l_w} - 0.5\right) + \frac{N_A}{4 l_w t_n} + \rho_{se} f_y\right) \qquad \text{(A2-1)}$$

where,

ϕ = Capacity reduction factor = 0.8
v_u = ultimate shear strength, psi
f_c' = concrete compressive strength, psi
h_w = wall height, in.
l_w = wall length, in.
N_A = axial load, lbs
t_n = wall thickness, in
ρ_{se} = A ρ_v + B ρ_u (A2-2)
f_y = steel yield strength, psi
ρ_v = vertical steel reinforcement ratio
ρ_u = horizontal steel reinforcement ratio
A, B = constants given as follows:

$\frac{h_w}{l_w} \leq 0.5$ A = 1 B = 0 (A2-3a)

$$0.5 \le \frac{h_w}{l_w} \le 1.5 \qquad A = -\frac{h_w}{l_w} + 1.5 \qquad B = \frac{h_w}{l_w} - 0.5 \qquad \text{(A2-3b)}$$

$$\frac{h_w}{l_w} \ge 1.5 \qquad A = 0 \qquad B = 1 \qquad \text{(A2-3c)}$$

According to ASCE/SEI Standard 43-05, Eq. (A2-1) is applicable to shear wall aspect ratios less than or equal to 2.0 and for ρ_v and ρ_u less than or equal to 0.01. If ρ_v or ρ_u is greater than 0.01, then ρ_{se} shall be limited to 0.01. Also, v_u shall satisfy the following expression:

$$v_u < 20\phi\sqrt{f_c'} \qquad \text{(A2-4)}$$

The total shear capacity is given by:

$$V_u = v_u d t_n \qquad \text{(A2-5)}$$

where d is the distance from the extreme compression fiber to the center of force of all reinforcement in tension which may be determined from a strain compatibility analysis. In lieu of an analysis:

$$d = 0.6 l_w \qquad \text{(A2-6)}$$

A2.2 Application of Methodology to Representative Wall

Capacity Calculation

The application of the ASCE Standard 43-05 approach to the representative reinforced concrete shear wall shown in Fig. 1 is presented below. The design parameters are the same as those listed in Section A1.2.1 of this report.

f_c' = 4,000 psi (27.6 MPa)

f_y = 60,000 psi (414 MPa)

h_w = 240 in. (609.6 cm)

l_w = 240 in. (609.6 cm)

t_n = 24.0 in. (60.96 cm)

N_A = 1,728,000 lb (7687 kN)

ρ_v = 0.00306

ρ_u = 0.00306

$\frac{h_w}{l_w}$ = 1.0

For $\frac{h_w}{l_w}$ = 1.0, the coefficients A and B can be calculated using the expressions defined in Eq. (A2-3b). This leads to A = 0.5 and B = 0.5. Using Eq. (A2-2), the effective steel ratio ρ_{se} is calculated to be 0.00306. Based on ASCE Standard 43-05 requirements, the reinforcement ratio is less than 0.01, and so it is acceptable to use ρ_{se} = 0.00306.

Concrete Contribution

Substituting the parameters listed above into Eq. (A2-1), without the capacity reduction factor (ϕ) and the last term for the contribution of steel, results in a unit shear strength from concrete equal to 492 psi (3.39 MPa).

Steel Contribution

Substituting the parameters into the last term in Eq. (A2-1), without the capacity reduction factor (ϕ), results in a unit shear strength from the steel reinforcement equal to 184 psi (1.27 MPa).

Total Unit Shear Strength

Adding the contribution from concrete and steel, the total nominal unit shear strength is equal to 676 psi (4.66 MPa). This is equivalent to 10.7$\sqrt{f_c'}$ which is less than the limit of 20$\sqrt{f_c'}$ given in Eq. (A2-4).

The total unit shear strength for design per Eq. (A2-1) requires multiplying the unit shear strength by ϕ = 0.8 which results in a strength of 541 psi (3.73 MPa).

Total Shear Capacity

Without performing a strain compatibility analysis, ASCE 43-05 specifies that d can be determined using Eq. (A2-6). This leads to d = 144 in. (365.8 cm), which can be substituted into Eq. (A2-5) to calculate a total shear strength across the wall equal to 1,870,000 lb (8318 kN).

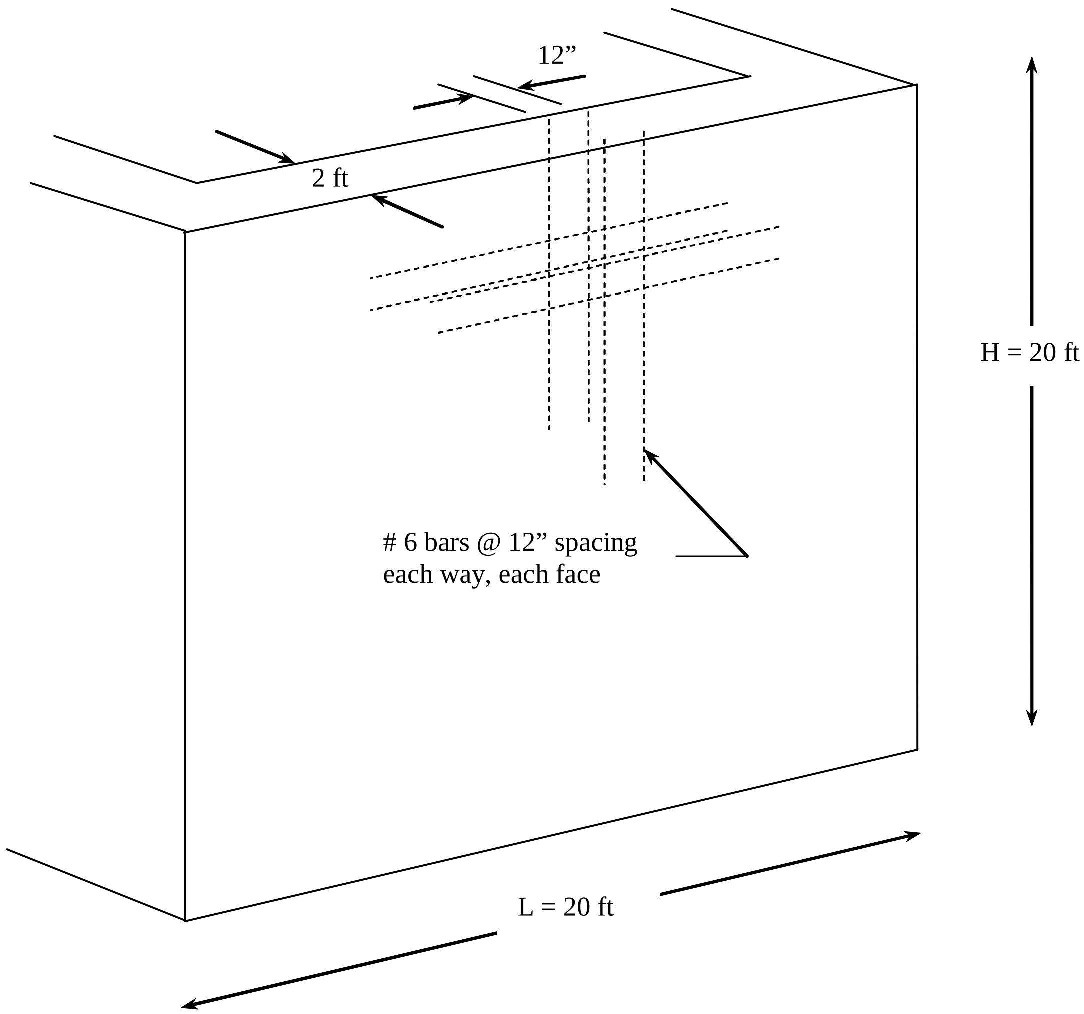
A3.0 Summary and Conclusion

The calculated nominal shear load capacity using the ASCE 43-05 method is 1,870 kips (8318 kN). This shear load capacity is much higher than the 1,292 kips (5,747 kN) and 1,290 kips

(5,738 kN) calculated using the equations presented in Chapter 11 and Chapter 21, respectively, from ACI 349-01. It is generally recognized that the ACI Code equations are conservative for low aspect ratio shear walls and the results presented above seem to support this conclusion. However, the equation used in ASCE/SEI Standard 43-05 is somewhat different than the equation based on the Barda, et al. method used in the past in other references. The differences primarily relate to the calculation of the A and B constants, limitation on ρ_v and ρ_u, and reduced distance "d" which represents the distance from the extreme compression fiber to the center of force of all reinforcement in tension. Also, the ACI Code Committees are currently reviewing the application of improved formulations based on test data for low-rise shear walls for potential revision of the Code equations.

It is recognized that equations, based on the Barda, et al. approach (very similar to the equation in ASCE/SEI Standard 43-05) have been used in the past for seismic fragility type calculations. When these methods were used for fragility assessments (not design) they have been generally reviewed on a case-by-case basis.

Based on the above discussion, unless additional review of the methodology is performed, it is recommended that the NRC should not generically endorse the equation for calculating the shear capacity of low-rise shear walls in ASCE/SEI Standard 43-05 for purposes of design, until a consensus on this approach is developed in the concrete design community and ACI 349 is revised to reflect this change.



1 in. = 2.54 cm; 1 ft = 30.48 cm

Fig. A-1 Representative Low-Rise Reinforced Concrete Shear Wall Problem