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MFN 06-429
Supplement 2

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Subject: **Response to Portion of NRC Request for Additional Information Letter No. 59 Related to ESBWR Design Certification Application - ESBWR Probabilistic Risk Assessment - RAI Numbers 19.5-3 S02 thru 19.5-14 S02**

Enclosure 1 contains GE's response to the subject NRC RAI transmitted via the Reference 1 questions and from the NRC Seismic Fragility Audit of February 5, 2007.

If you have any questions or require additional information regarding the information provided here, please contact me.

Sincerely,

Kathy Sedney for

James C. Kinsey
Project Manager, ESBWR Licensing

D068

Reference:

1. MFN 06-329, Letter from U.S. Nuclear Regulatory Commission to David Hinds, *Request for Additional Information Letter No. 59 Related to ESBWR Design Certification Application*, September 13, 2006.

2. MFN 06-429, *Response to Portion of NRC Request for Additional Information Letter No. 59 Related to ESBWR Design Certification Application –ESBWR Probabilistic Risk Assessment and Chapter 19 – RAI Numbers 19.5-3 through 19.5-14.* October 29, 2006.

3. MFN 06-429, Supplement 1, *Supplemental Response to Portion of NRC Request for Additional Information Letter No. 59 Related to ESBWR Design Certification Application ESBWR - RAI Numbers 19.5-3 through 19.5-14.* January 16, 2007.

Enclosures:

1. MFN 06-429 Supplement 2, *Response to Portion of NRC Request for Additional Information Letter No. 59 Related to ESBWR Design Certification Application - ESBWR Probabilistic Risk Assessment - RAI Numbers 19.5-3S2 thru 19.5-14S2.*

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ENCLOSURE 1

MFN 06-429 SUPPLEMENT 2

Partial Response to RAI Letter No. 59

Related to ESBWR Design Certification Application (Previously Submitted[1] Under MFN 06-429 Supplement 1)

SUPPLEMENT 2

ESBWR Probabilistic Risk Assessment RAI Numbers 19.5-3 S2, 19.5.4 S2, 19.5-5 S2, 19.5-6 S2, 19.5-7 S2, 19.5-8 S2, 19.5-9 S2, 19.5-10 S2, 19.5-11 S2, 19.5-12 S2, 19.5-13 S2 and 19.5-14 S2

[1] Original response previously submitted under MFN 06-429 Supplement 1 without DCD updates. These responses are included to provide historical continuity during review.

NRC RAI 19.5-3, Supplement 1

DCD Tier 2, Section 19.2.2.4 provides a seismic margin analysis result of 0.6g for the High Confidence Low Probability of Failure (HCLPF). A seismic margins analysis to determine that the plant HCLPF for a certified design should be at least equal to 1.67 times the safe shutdown earthquake (SSE), based on criteria in SECY 93-087,"Policy, Technical and Licensing issues Pertaining to Evolutionary and Advances Light-Water Reactor (ALWRs) Designs," April 2, 1993. The seismic margins analysis addressing the criteria in SECY 93-087 should be located in this section of the DCD. The associated structural calculations and assumptions need to be presented in DCD Tier 2,Chapter 19, showing all relevant assessments of the critical elements necessary to maintain plant performance during and after the SSE. References applicable to HCLPF calculations should be presented in Chapter 19.

GE Response

Table 19.2-4 of DCD Tier 2 Chapter 19 Rev. 2 will be revised in the next update.

DCD Impact

Table 19.2-4 of DCD Tier 2 Chapter 19 will be revised in the next update as noted in the attached markup.

NRC RAI 19.5-3, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

As stated in RAI 19.2-66 S2, a single performance-base seismic design spectrum is considered for all sites. The plant level HCLPF of the ESBWR standard design is at least 1.67 times SSE.

DCD Impact

Sections 19.2.3.5, 19.2.4.1.5 and Table 19.2-4 of DCD Tier 2 Chapter 19 will be revised in the next update as noted in the attached markup.

NRC RAI 19.5-4, Supplement 1

All the certified design components important for the plant HCLPF analysis should be presented in a tabluar form in the DCD Tier 2,Chapter 19. Also, the table of HCLPF values in the ESBWR Probabilistic Risk Assessment (PRA) Report (NEDO-33201) should be incorporated into Tier 1 of the DCD as a part of an ITAAC item to ensure and verify that the as-built plant HCLPF is equal to or greater than the certified plant HCLPF value.

GE Response

Seismic categorizations of SSCs are part of the standard inputs within the Tier 1 Design Descriptions and ITAAC HCLPF margins information will not be included as ITAAC items in Tier 1 because existing ITAAC items for various SSCs ensure that the plant has adequate seismic margin beyond the design basis SSE due to the various conservatism introduced in the normal design process as explained in response to RAI 19.5-5.

DCD Impact

Table 19.2-4 of DCD Tier 2 Chapter 19 will be revised in the next update as noted in the attached markup.

NRC RAI 19.5-4, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

Verification of the as-built plant HCPLF is a COL Action Item as stated in DCD Tier 2 Section 19.5.1.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NRC RAI 19.5-5, Supplement 1

Provide the essential elements of a procurement specification and associated installation criteria that would ensure that Structures, Systems and Components (SSCs) are procured and installed to develop the necessary HCLPF capacities.

GE Response

HCLPF capacity depends on the seismic margins in the structural response, equipment response and equipment capacity.

Conservatisms in the design and current practices in design and qualification procedures typically provide for HCLPF capacities exceeding 1.67 times SSE ground motion. These include anchorage design per ACI-349, seismic qualification per IEEE-344 requirements and dynamic analysis per ASCE 43-05 (including the load factor of 1.4 on testing). In addition, designers typically optimize the design effort by enveloping the floor spectra at different floor locations for specification of required response spectra and by designing standard piping and cable tray supports that cover a number of system configurations and loading conditions. These considerations are used in assigning what are deemed "reasonably achievable" fragilities.

It is not practical to specify the HCLPF capacity of structures and equipment for the vendors to meet as target values. The responsibilities for design and qualification are distributed among the architect-engineer, reactor manufacturer and equipment vendors. The equipment vendor is only responsible for qualification testing; the anchorage design is by the architect-engineer and the structural analysis that develops the floor spectra is typically by another group within the AE. Therefore, inclusion of the HCLPF capacities as an element of the procurement specifications is not feasible.

Instead, the procurement specification and associated installation criteria will have the following:

- Required Response Spectra (RRS) for qualification by analysis and testing
- Requirements for anchorage design per ASME/ACI-349 code
- Conformance to IEEE requirements for qualification testing
- Conformance to ASCE 43-05 requirements on dynamic analysis and load combinations.

GE will apply load factors on SSE demands in the procurement specifications when it is deemed necessary in order to assure that the HCLPF capacity will equal or exceed 1.67 times the SSE.

DCD Impact

Table 19.2-4 of DCD Tier 2 Chapter 19 will be revised in the next update as noted in the attached markup.

NEDO-33201 Section 15.3 will be revised in the next update.

NRC RAI 19.5-5, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

DCD Tier 2 Section 19.5.1 COL Action Item ensures that procured items meet the HCLPF requirements.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NRC RAI 19.5-6, Supplement 1

In Section 15.3.3 of NEDO-33201, Rev. 1, it has been recognized that relative displacements limiting SSC operability frequently control their seismic capacity. The structural fragility assessment method in Reference 15-1, R.P. Kennedy, et al., "Assessment of Seismic Margin Calculation Methods", NUREG/CR-5270, Lawrence Livermore National Laboratory, March 1989, is somewhat dated, and is based on a PWR plant study. The ESBWR design is very different - it has a very tall reactor vessel and drywell functionality is very much dependent on proper functioning of all pressure suppression components. Simply because of the reactor vessel height, a small amount rotation at the pedestal would significantly scale up the displacement near the reactor vessel head and the top of the drywell. Please discuss individual elements of functionality limits for ensuring drywell and wetwell functionality and the integrity of components attached to the reactor vessel

GE Response

NUREG/CR-5270 is used as a guide for processes and procedures applicable to any type of plant. Any unique ESBWR features are addressed through the normal design process to ensure adequacy in withstanding the design basis earthquake, and their HCLPF capacities are in turn estimated from design basis information accordingly.

Please note that the methodology for seismic margin evaluation described in NUREG/CR-5270 has not changed significantly. However, ANS Standard ANS 58.21 refers to two documents EPRI Fragility Methodology report (EPRI TR -103959, Reference 19.5-6(1)) and EPRI seismic margin methodology report (EPRI NP-6041, Reference 19.5-6(2)). These were not referred to in the DCD since they were proprietary documents that have been made available to the public.

References:

19.5-6(1): Electric Power Research Institute, "*Methodology for Developing Seismic Fragilities*", prepared by R.P. Kennedy and J. W Reed, EPRI TR-103959, June 1994

19.5-6(2): Electric Power Research Institute, "A Methodology for Assessment of Nuclear Plant Seismic Margin", prepared by Jack R. Benjamin and Associated, Inc., et al, EPRI NP -6041, June 1991

DCD Impact

No DCD changes will be made in response to this RAI.

NEDO-33201 Section 15.3 will be revised in the next update.

NRC RAI 19.5-6, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

The two referenced EPRI reports mentioned in Supplement 1 to this RAI will be inlcluded in NEDO-33201 Section 15.3.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NEDO-33201 Section 15.3 will be revised in the next update as noted in the attached markup.

NRC RAI 19.5-7, Supplement 1

Provide a description of the failure modes used to determine the HCLPF values for category I structures, particularly the containment structure. Provide a description of the extrapolation process supplemented by judgment.

GE Response

See the response to RAI 19.2-67 for the containment structure and the response to RAI 19.2-66 for the shear wall structures.

The analyzed failure modes for structures are based on the recommendations given in EPRI TR-103959 (Reference 19.5-6(1)). The extrapolation of the SSE elastic response to the ultimate strength of the component (depending on the failure mode and available ductility) followed the guidance of the above document.

DCD Impact

No DCD changes will be made in response to this RAI.

NRC RAI 19.5-7, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

See RAI 19.2-67 S2 for the containment structure and RAI 19.2-66 S2 for the shear wall structures.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NRC RAI 19.5-8, Supplement 1

Provide a description of how HCLPF values are determined for equipment and components qualified by testing, especially for the North Anna early site permit (ESP) site-specific ground motion spectrum.

GE Response

The ESBWR design certification is a generic plant licensing activity, and by definition, the DCD is not a site-specific licensing document. Therefore, North Anna site-specific analyses are not within the scope of the ESBWR design certification.

The overall approach for determining HCLPF capacities of equipment and components qualified by seismic testing is described in EPRI TR-103959 (Reference 19.5-6(1)). Since the detailed design information on the equipment is not available at this time, generic HCLPF capacities are assigned following the ALWR URD. These generic HCLPF capacities assumed for equipment and components are considered achievable because of the margins or safety factors introduced at different stages of equipment design and qualification. Equipment qualified for application in GE ESBWR plants have additional seismic margins in high frequencies because the design considers high-frequency hydrodynamic loads in combination with seismic loads. The other sources of margin are from conservatism in the ESBWR seismic response analysis, e.g., use of single enveloping design spectra and conservative treatment of soil-structure interaction and the use of enveloping responses of all site conditions for design.

The equipment and components of the GE ESBWR plant will be qualified to the required floor response spectra arising from the single envelope ground motion input rich in both low and high frequencies and following the ASCE, ASME and IPEEE procedures. Their seismic HCLPF capacities are expected to meet the required value of 1.67 times 0.5g peak ground acceleration for rock sites or 1.67 times 0.3 peak ground acceleration for soil sites.

Given the single enveloping design spectra of the ESBWR and the performance-based design spectra for new units, it becomes obvious that the rock sites are the most challenging for meeting the required HCLPF capacity if the building frequency is higher than 9 Hz. At 9 Hz and above, the single enveloping design spectra is the same as the North Anna ESP design spectra and the structural response factor is only slightly greater than unity when other variables that affect the seismic response of the structures are considered. In such a case, the required response spectra (RRS) is appropriately factored throughout the frequency range to assure that the HCLPF margin of 1.67 is met.

DCD Impact

No DCD changes will be made in response to this RAI.

NEDO-33201 Section 15.3 will be revised in the next update.

NRC RAI 19.5-8, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

DCD Tier 2 Section 19.5.1 COL Action Item ensures that equipment and components qualified by testing meet the HCLPF requirements.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NRC RAI 19.5-9, Supplement 1

Justify the use of both ductility (inelastic energy absorption factor) as well as damping (structural response factor) effects to determine the overall factor of safety.

GE Response

The factor representing the median and variability in ductility (inelastic energy absorption) factor, F_u is calculated according to the effective frequency/effective damping method and/or effective Riddell-Newmark method recommended in EPRI TR-103959 (Reference 19.5-6(1)). Similarly, the damping factor (F_d) based on the damping when the structure is "at or just below yield point". EPRI TR-103959 states "--- for structures whose fragilities are determined using the simplified response spectra methods, the exact damping is unimportant since assumptions concerning the ductility ratio and the damping ratio are self-correcting. In other words, the assumption of increased damping results in a lower (more conservative) factor of safety for the inelastic energy dissipation factor and balances the increased factor of safety in the elastic demand due to higher damping."

In the seismic fragility calculation of ESBWR structures, a median damping value of 7% of critical is conservatively assumed for the inelastic energy absorption factor calculation. For rock sites, 5% damping is assumed even when the calculated effective damping value is greater than 5% because the North Anna ESP design spectrum, on which the high frequency components of the single envelope design input spectrum is based, is available only for the 5% damped case. This assumption results in an inelastic energy absorption factor with slight conservative bias.

DCD Impact

No DCD changes will be made in response to this RAI.

NEDO-33201 Section 15.3 will be revised in the next update.

NRC RAI 19.5-9, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

Damping and ductility factors have been recalculated for consistency with the single performance-base seismic design spectrum for all sites.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NEDO-33201 Section 15.3 will be revised in the next update as noted in the attached markup.

NRC RAI 19.5-10, Supplement 1

Section 15.3.1 of NEDO-33201, Rev. 1, states that generic fragilities were chosen based on a review of prior PRAs and fragility data and that they are considered achievable for the ESBWRs with an evolutionary improvement in the seismic capacities of the components designed to a 0.3g SSE minimum. Provide a list of the prior PRAs and the bases for using their fragility values. If multiple fragility values for similar components were available, please describe the bases for the chosen value. Please describe where and how these generic fragility data were used to establish 0.6g HCLPF value for the ESBWR. Elaborate on the meaning of the phrase "evolutionary improvement" and how this ensures that these fragilities are achievable.

GE Response

As stated in NEDO-33210 Rev 1, Section 15.3.5, generic fragilities and corresponding HCLPF capacities are the same as those considered in the ABWR SSAR and ALWR recommendations (EPRI ALWR Utility Requirements Document, Appendix A PRA Key Assumptions and Ground rules); furthermore, it was shown that these HCLPF capacities were achieved for the Lungmen NPP Project in Taiwan, which has a 0.40g SSE.

The HCLPF capacities assigned are expected to be "reasonably achievable" for the ESBWR just as they were for the high seismic accceleration in Taiwan. As stated in response to RAI 19.5-5, GE applies load factors on SSE demands in the procurement specifications when it is deemed necessary to assure that the HCLPF capacity equals or exceeds 1.67 times the SSE.

The use of the term "evolutionary improvements" means that equipment may become more seismically rugged as vendors improve upon the design of their products. GE reviews the specific equipment design against the designs that form the basis of the ALWR URD recommendations.

DCD Impact

No DCD changes will be made in response to this RAI.

NRC RAI 19.5-10, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

Generic HCLPFs will be revised to be at least 1.67 times SSE.

DCD Impact

Sections 19.2.3.5, 19.2.4.1.5 and Table 19.2-4 of DCD Tier 2 Chapter 19 will be revised in the next update as noted in the attached markup.

NEDO-33201 Section 15.3 will be revised in the next update as noted in the attached markup.

NRC RAI 19.5-11, Supplement 1

Section 15.3.1 of NEDO-33201, Rev. 1, states that the peak ground acceleration (PGA) of the design earthquakes is 0.3g for the SSE while the North Anna specific SSE has a PGA value of 0.49g. Please clarify which PGA value was used in your analyses to compute the capacity factors, particularly the strength factor (Fs). A certified design for the North Anna ESP response spectra would put the plant HCLPF value at 1.67x0.49g or about 0.82g, please explain how you meet the HCLPF.

GE Response

The HCLPF capacities currently shown in NEDO-33201 Rev. 1 are relative to 0.3g PGA of the RG 1.60 spectral shape. Two sets of HCLPF capacities will be developed: one for rock sites and another for soil sites. This approach provides consistentcy with the updated definition of SSE design ground motion which is a single envelope of the 0.3g RG 1.60 and North Anna specific SSE. As shown in Figure 19.5-13(1), the peak ground acceleration of the rock spectra is specified as 0.5g while it is 0.3g for soil sites.

DCD Impact

No DCD changes will be made in response to this RAI.

NEDO-33201 Rev 1, Section 15.3 will be revised in the next update.

NRC RAI 19.5-11, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

As stated in RAI 19.2-66 S2, a single performance-base seismic design spectrum anchored to 0.5g PGA is now considered for all sites.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NEDO-33201 Section 15.3 will be revised in the next update as noted in the attached markup.

NRC RAI 19.5-12, Supplement 1

Justify the use of Equation 15.3-11 in NEDO-33201, Rev. 1, to determine the ultimate shear strength for short reinforced concrete shear walls, typical of nuclear power plants. Provide the equation used to determine the ultimate shear strength for the containment wall.

GE Response

See the response to RAI 19.2-66 for shear strength of shear walls and the response to RAI 19.2-67 for the ultimate seismic strength assessment of containment. The method of calculating ultimate shear capacity follows the guidance in EPRI NP-6041 (Reference 19.5-6(2)) for containment structures.

DCD Impact

No DCD changes will be made in response to this RAI.

NRC RAI 19.5-12, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

See RAI 19.2-66 S2.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NEDO-33201 Section 15.3 will be revised in the next update as noted in the attached markup.

NRC RAI 19.5-13, Supplement 1

For the shape factor (Fsa), Section 15.3.3.1.2 of NEDO-33201, Rev. 1,states that for the purpose of seismic risk assessment, the median ground motion spectrum given in NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," is considered to be the realistic input ground motion definition. Considering the significant number of advancements in the field of seismic hazards since the development of this spectrum in the late 1970's, justify your consideration of the NUREG/CR-0098 spectrum as realistic input ground motion.

GE Response

The spectral shape factors are reevaluated for two site conditions: rock and soil. For rock sites, the North Anna ESP site SSE spectrum is compared to the ESBWR single envelope design spectrum to determine the shape factor. For soil sites, the bounding SSE spectrum of soil sites among the 28 sites (excluding Vogtle) included in the current EPRI study (Reference 19.5-13(1)) is compared to the ESBWR single envelope design spectrum to determine the shape factor. Figure 19.5-13(1) shows this spectra comparison.

DCD Impact

No DCD changes will be made in response to this RAI.

NEDO-33201 Section 15.3 will be revised in the next update.

References

Reference 19.5-13 (1) Electric Power Research Institute "Assessment of Performance-Based Approach for Determining the SSE Ground Motion for New Plant Sites, V 1: Performance-Based Seismic Design Spectra", Product ID # 1012044, Final Report, June 2005.

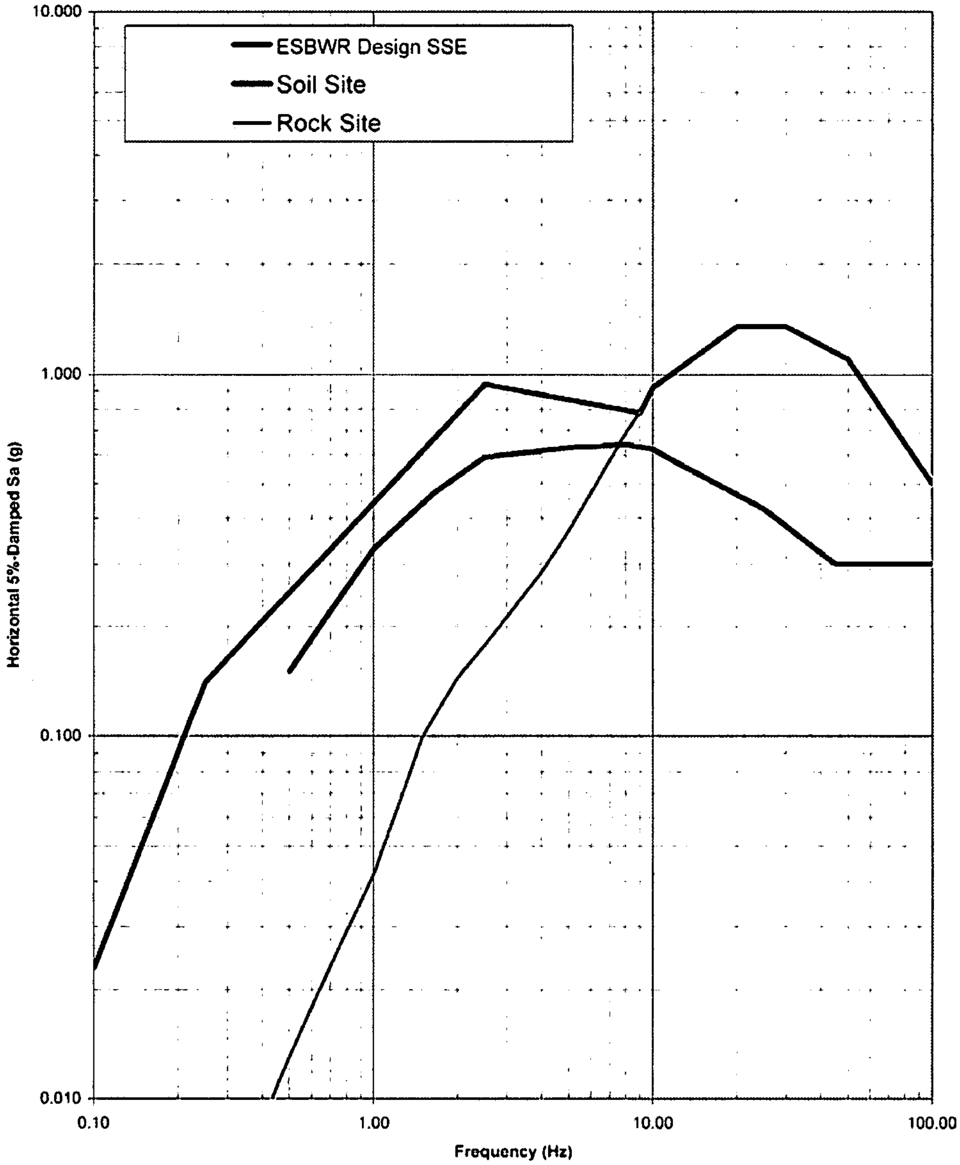

Figure 19.5-13 (1) Comparison of Design Ground Spectrum with Soil and Rock Spectra

NRC RAI 19.5-13, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

In order to be consistent with the updated definition of SSE design ground motion being a single envelope of 0.3g RG 1.60 and North Anna specific SSE, a single performance-based seismic design spectrum is used for the fragility calculation. Since the Reg. Guide 1.60 spectra which dominate the frequencies below 9 Hz are judged to be 84th percentile spectra or higher, a performance-based 5% damped design horizontal spectrum that bounds all the soil sites in the recent EPRI study, except Vogtle, is developed. The envelope of this bounding soil spectrum and the North Anna ESP performance-based design spectrum, hereafter called the performance-based seismic design spectrum, is used for the seismic fragility calculation of the ESBWR standard plant design. Figure 19.5-13(2) presents both the single envelope design spectrum and the performance-based seismic design spectrum, anchored to a 0.5g peak ground acceleration.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NEDO-33201 Section 15.3 will be revised in the next update as noted in the attached markup.

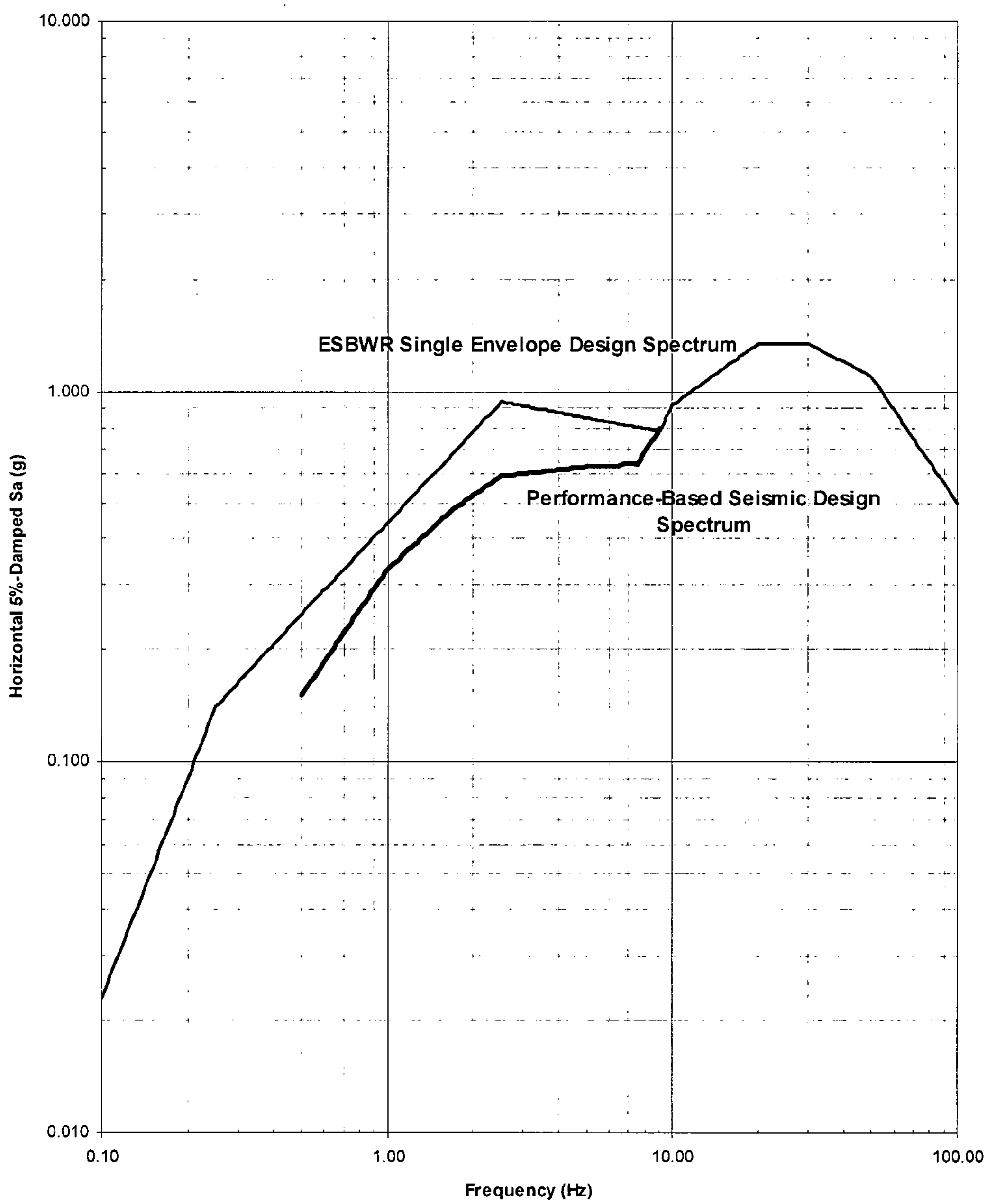

Figure 19.5-13 (2) Comparison of 5% Damped Single Envelope Design Ground Spectrum with Performance-Based Seismic Design Spectrum

NRC RAI 19.5-14, Supplement 1

Provide a comparison showing ratios of the bounding (all site conditions) seismic responses of the containment structure at important locations to the critical functionality limits. Using the highest ratio determine the HCLPF value.

GE Response

See response to RAI 19.2-67 for the containment structure.

The HCLPF values are revised based on the new spectra discussed in the response to RAI 19.5-13. The highest ratio (calculated value divided by the allowable value) for the critical functional failure mode) is used to determine the HCLPF capacity in accordance with EPRI TR 103959 methodology (Reference 19.5-6(1)).

DCD Impact

No DCD changes will be made in response to this RAI.

NEDO-33201 Section 15.3 will be revised in the next update.

NRC RAI 19.5-14, Supplement 2

This supplemental response is made for consistency with the issues addressed in RAIs 19.2-43 S2, 19.2-66 S2, 19.2-67 S2 and 19.2-68 S2 as discussed in the February 5, 2007 audit.

GE Response

See RAI 19.2-67 S2 for updated fragility summary for RCCV evaluated in accordance with updated performance-based seismic design spectra discussed in RAI 19.5.13 S2.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NEDO-33201 Section 15.3 will be revised in the next update as noted in the attached markup.

19.2.3.4.4 Significant Offsite Consequences of External Event Flood

Due to the bounding method that is used to calculate the high wind CDF and its very low value compared to that of internal events CDF, it is considered to be unnecessary to extrapolate offsite consequences.

19.2.3.4.5 Summary of Important Results and Insights of External Event High Wind

Table 19.2-3 summarizes the important initiating events, operator actions, common cause failures (CCF), SSCs, assumptions, and insights from importance, sensitivity, and uncertainty analyses.

19.2.3.5 Evaluation of External Event Seismic

19.2.3.5.1 Introduction to Evaluation of External Event Seismic

The seismic risk analysis is performed to assess the impacts of seismic events on the safe operation of the ESBWR plant. A PRA-based seismic margins analysis is performed for the ESBWR to calculate high confidence low probability of failure (HCLPF) accelerations for important accident sequences and accident classes. The ESBWR seismic margins HCLPF accident sequence analysis concludes that the ESBWR is inherently capable of safe shutdown in response to beyond design basis earthquakes and has a plant level HCLPF of 1.67 times the safe shutdown earthquake (SSE). Chapter 15 of NEDO-33201 provides details of the PRA-based seismic margin assessment.

Table 19.2-4 contains the systems evaluated in the ESBWR and contains minimum HCLPF ratio for these systems.

19.2.3.5.2 Significant Core Damage Sequences of External Event Seismic

A PRA-based Seismic Margins Analysis is used to derive seismic vulnerability insights. Therefore, there are no CDF calculations performed. The PRA-based Seismic Margins Analysis concludes that the most significant HCLPF sequences are seismic-induced loss of preferred power and seismic-induced ATWS due to seismic-induced failure of the fuel channels and seismic-induced failure of the SLC tank.

Based on previous industry seismic analyses, seismic risk is dominated by seismic-induced SSC failures, and not by random SSC failures or human actions. Human actions are typically not necessary until the long-term.

19.2.3.5.3 Significant Large Release Sequences of External Event Seismic

A PRA-based Seismic Margins Approach is used to derive seismic vulnerability insights. Therefore, there are no LRF calculations performed.

19.2.3.5.4 Significant Offsite Consequences of External Event Seismic

A PRA-based Seismic Margins Approach is used to derive seismic vulnerability insights. Therefore, there are no off-site consequences calculations performed. Due to the bounding method that is used to calculate the seismic margin, it is considered to be unnecessary to extrapolate offsite consequences.

19.2.4.1.2 Fire During Shutdown

Important fire initiating events in the shutdown internal fires PRA are fires in the Reactor Building Division I and Division II Zones during Mode 5. The analysis conservatively assumes that a fire in these zones could cause the inadvertent opening of an SRV. Failure of the corresponding safety system division is assumed, along with failure of one train of RWCU/SDC and CRD, depending on the particular zone that contains the fire.

The important operator actions in the shutdown internal fires PRA are failure to recognize the need for RPV depressurization, and failure to start a condensate pump.

The important individual SSC failure for fire scenarios is Reactor Building fire barrier failure.

19.2.4.1.3 Flooding During Shutdown

The important flood initiating event in the shutdown internal flooding PRA is a CRD break in the Reactor Building during Mode 6. However, the total contribution flood during shutdown sequences is negligible.

19.2.4.1.4 High Winds During Shutdown

Similar to the full power risk profile, the shutdown risk for high winds are limited to Loss of Preferred Power events with a potential loss of the Condensate Storage Tank.

Operator actions are non-significant contributors to the shutdown high wind risk profile. The important common cause failure (CCF) in the shutdown tornado PRA is CCF of all batteries. Random failures of systems, structures or components are not significant contributors to the internal events shutdown CDF.

19.2.4.1.5 Seismic Events During Shutdown

The PRA-based Seismic Margins Analysis for shutdown concludes that the most significant HCLPF sequences are seismic-induced loss of preferred power.

Based on previous industry seismic analyses, seismic risk is dominated by seismic-induced SSC failures, and not by random SSC failures or human actions. Human actions are typically not necessary until the long-term.

19.2.4.1.6 Shutdown PRA Assumptions

Important design assumptions in the shutdown analysis are as follows:

Compared to Residual Heat Removal System in BWRs, the RWCU/SDCS in the ESBWR does not have the potential for diverting RPV inventory to the suppression pool through the SP suction, return, or spray lines.

The arrangement for preventing vessel draining through back-seating of the control rod drive mechanism (CRDM) is the same as the one used in the BWRs and in the ABWR. Therefore, the ESBWR design does not introduce a new challenge to vessel inventory relative to CRDMs.

It is assumed that both RWCU/SDCS trains are running, because the time periods in which only one is running occurs when the reactor well is flooded. Consequently, failure of one of the trains is not considered an initiating event.

Table 19.2-4
ESBWR Systems and Structures in Seismic Margins Analysis With Plan Level HCLPF Margin ≥ 1.67[1]

<u>PLANT STRUCTURES</u> - Reactor Building - Containment - RPV Pedestal - Control Building - Reactor Pressure Vessel Support <u>DC POWER</u> - Batteries - Cable trays - Motor control centers <u>REACTIVITY CONTROL SYSTEM</u> - Fuel assembly - CRD Guide tubes - Shroud support - CRD Housing - Hydraulic control unit <u>SRV</u> - SRV <u>STANDBY LIQUID CONTROL</u> - Accumulator Tank - Check valve - Squib valve - Piping - Valve (motor operated)

Table 19.2-4
ESBWR Systems and Structures in Seismic Margins Analysis With Plan Level HCLPF Margin ≥ 1.67[1]

<u>ISOLATION CONDENSER</u>
- Piping
- Heat exchanger
- Valve (motor operated)
- Valve (nitrogen operated)
<u>DPV</u>
- DPV
<u>GRAVITY-DRIVEN COOLING</u>
- Check valve
- Squib valve
- Piping
<u>VACUUM BREAKERS</u>
- Vacuum breaker valve
<u>PASSIVE CONTAINMENT COOLING</u>
- Heat Exchanger
- Piping
<u>IC/PCC POOL INTERCONNECTION</u>
- Valve (motor operated)
<u>FIRE PROTECTION WATER SYSTEM</u>
- Pump (diesel driven)

Note:

1. A minimum HCLPF value of 1.67*SSE will be met for the equipment shown.

15 SEISMIC MARGINS ANALYSIS

This section documents the PRA-based seismic margin analysis of the ESBWR.

15.1 INTRODUCTION

The seismic risk analysis is performed to assess the impacts of seismic events on the safe operation of the ESBWR plant.

A PRA-based seismic margins analysis (SMA) is performed for the ESBWR using the systems models and the fragility analysis method of Reference 15-1 to calculate high confidence low probability of failure (HCLPF) accelerations for important accident sequences and accident classes.

The analysis shows that the ESBWR plant is capable of withstanding an earthquake of at least 1.67 times the safe shutdown earthquake (SSE) with a high confidence of low probability of failure.

The scope of the analysis includes both at-power and shutdown seismic-induced accident scenarios.

15.2 METHODOLOGY

The seismic risk assessment uses a seismic margins analysis (SMA) method based on that of References 15-1 and 15-2 to calculate high confidence low probability of failure (HCLPF) seismic capacities for important accident sequences and accident classes.

The PRA-based seismic margins approach used in this analysis evaluates the capability of the plant to withstand an earthquake of 1.67 times the safe shutdown earthquake (1.67*SSE).

The analysis involves the following two major steps:

(1) Seismic fragilities

(2) Accident sequence HCLPF analysis

The seismic fragilities of the ESBWR systems, structures, and components are based on generic industry information and ESBWR specific seismic capacity calculations for certain structures.

The MIN-MAX method is used in the determination of functional and accident sequence fragilities. Per the MIN-MAX method, the overall fragility of a group of inputs combined using OR logic (i.e., seismic event tree nodal fault tree) is determined by the lowest (minimum) HCLPF input. Conversely, per the MIN-MAX method, the overall fragility of a group of inputs combined using AND logic (i.e., seismic event tree sequence) is determined by the highest (maximum) HCLPF input.

Both at-power and shutdown seismic-induced accident scenarios are analyzed.

15.3 SEISMIC FRAGILITIES

15.3.1 Overview

This subsection presents seismic capacities for selected structures and components that have been identified as potentially important to the seismic risk analysis of the ESBWR standard plant. The seismic capabilities in terms of seismic fragilities are first estimated, from which the high confidence of low probability of failure (HCLPF) capacities are then derived. The HCLPF capacities serve as input to the system analysis following the seismic margins approach.

The peak ground acceleration of the design earthquakes is 0.5g for the Safe Shutdown Earthquake (SSE). Extensive seismic soil-structure interaction analyses of the reactor/fuel building complex and control building were performed for a wide range of generic site conditions under a 0.5g single envelope design spectra. This single envelope design spectra is a composite of Reg. Guide 1.60 spectra anchored to 0.3g and the North Anna ESP design spectra anchored to 0.5g. The analysis results in terms of site-envelope SSE loads are presented in Appendix 3A of the ESBWR DCD Tier 2 Rev. 3 (Reference 15-3). The standard plant designed to these site-envelope seismic loads may result in significant design margins when it is situated at a specific site, particularly a soft soil site. Thus, the seismic capacities estimated from the site-envelope design requirements may be very conservative for certain sites and confirmation of margins could be done for as-built conditions.

For the seismic category I structures for which seismic design information is available, the seismic fragilities are evaluated using the Separation-of-Variable method in Reference 15-1. This approach identifies various conservatisms and associated uncertainties introduced in the seismic design process (both capacity and demand sides) and provides a probabilistic estimate of the earthquake level required to fail a structure or component in a postulated failure mode by extrapolating from the design information supplemented by limited nonlinear analysis to account for building response beyond yielding.

For safety-related components such as pumps, valves, and electrical equipment whose design details are not currently available, a generic HCLPF capacity of 1.67*SSE is assigned. This generic HCLPF is considered to be "reasonably achievable" for the ESBWRs designed to the single envelope design spectra for a wide range of sites.

15.3.2 Fragility Formulation

Seismic fragility of a structure or component is defined herein to be the cumulative conditional probability of its failure as a function of the mean peak ground acceleration (i.e., the average of the peak of the two horizontal components).

The probability model adopted for fragility description is the lognormal distribution. Using the lognormal distribution assumption, an entire family of fragility curves can be fully described in terms of the median ground acceleration and two random variables as:

$$A = A_m \varepsilon_\gamma \varepsilon_\mu \tag{15.3-1}$$

Where:

A_m = median peak ground acceleration corresponding to 50% failure probability.

ε_γ = a lognormally distributed random variable accounting for inherent randomness about the median. It is characterized by unit median and logarithmic standard deviation β_γ.

ε_μ = a lognormally distributed random variable accounting for uncertainty in the median value. It is characterized by unit median and logarithmic standard deviation β_μ.

With known values of A_m, β_γ, and β_μ, the failure probability P_f at acceleration less than or equal to a given acceleration a can be computed using the following equation for any non-exceedance probability (NEP) level Q.

$$P_f(A \leq a|Q) = \phi\left[\frac{1}{\beta_\gamma} In\left(\frac{a}{A_m}\right) + \frac{\beta_\mu}{\beta_\gamma}\phi^{-1}(Q)\right] \quad (15.3\text{-}2)$$

Where Φ is the standard Gaussian cumulative distribution function. Figure 15-1 shows a typical family of fragility curves for various NEP levels. The center solid curve represents the median fragility curve at 50% NEP level. The logarithmic standard deviation of the randomness component β_γ determines the curve slope. The logarithmic standard deviation of the uncertainty component β_μ is a measure of the spread from the median curve. The 95th percentile and 5th percentile curves in Figure 15-1 are the upper and lower bounds of the failure probability for a given acceleration, corresponding to 95% and 5% NEP levels, respectively.

When only the point estimate is of interest, which is the case for this analysis, the total variability about the median value is taken to be the square root of the sum of the squares (SRSS) of the randomness and uncertainty components.

$$\beta_c = \sqrt{\beta_\gamma^2 + \beta_\mu^2} \quad (15.3\text{-}3)$$

The fragility curve corresponding to the median value A_m with associated composite logarithmic standard deviation can be computed by the following equation:

$$P_f(A \leq a) = \phi\left[\frac{1}{\beta_c} \ln\left(\frac{a}{A_m}\right)\right] \quad (15.3\text{-}4)$$

This composite fragility curve is also called the mean fragility curve and is shown as the dashed curve in Figure 15-1 for illustration. It represents the best estimate fragility description.

In estimating the median ground acceleration capacity and the associated variability, an intermediate variable defined as safety factor F is utilized. The safety factor is related to the median ground acceleration capacity by the following relationship.

$$A_m = FA_d \quad (15.3\text{-}5)$$

Where A_d is the ground acceleration of the reference design earthquake to which the structure or component is designed. A key step in the seismic fragility estimate thus involves the evaluation of the factor of safety associated with the design for each important potential failure mode. The design margins inherent in the component capacity and the dynamic response to the specific acceleration are the two basic considerations. Each of the capacity and response margins involves several variables, and each variable has a median factor of safety and variability associated with it. The overall factor of safety F is the product of the factor of safety for each variable F_i.

$$F = \prod_i F_i \tag{15.3-6}$$

The overall composite logarithmic standard deviation is SRSS of the composite logarithmic standard deviations in the individual factors of safety.

$$\beta_c = \sqrt{\sum_i \beta_{ci}^2} \tag{15.3-7}$$

Knowing the median peak ground acceleration (A_m) and associated logarithmic standard deviation (β_c); the HCLPF capacity is obtained using the equation below.

$$\text{HCLPF} = A_m \exp(-2.326\beta_c) \tag{15.3-7a}$$

15.3.3 Structural Fragility

The plant structures are divided into two categories according to their function and the degree of integrity required to protect the public during a seismic event. These categories are seismic category I and non-category I. Seismic category I includes those structures whose failure might cause or increase the severity of an accident, which would endanger the public health and safety. The reactor building and control building structures are in this category. The non-category I structures are those structures which are important to reactor operation, but are not essential for preventing an accident which would endanger the public health and safety, and are not essential for the mitigation of the consequences of these accidents. One example is the turbine building structure.

For the purpose of this study, structures are considered to fail functionally when inelastic deformations of the structure under seismic load increase to the extent that the operability of the safety-related components attached to the structure cannot be assured. The drift limits chosen for structures are estimated as corresponding to the onset of significant structural damage. For many potential modes of failure, this is believed to represent a conservative bound on the level of inelastic structural deformation that might interfere with the function of the system housed within the structure.

The potential of seismic-induced soil failure such as liquefaction, differential settlement, or slope instability is highly site dependent and cannot be assessed for generic site conditions. It is assumed in this analysis that there is no soil failure potential in the range of ground motions considered.

Building-to-building impact due to differential building displacements under strong earthquakes is deemed not credible since a sufficient distance to avoid impact separates adjacent buildings. Differential building displacements of sufficient magnitude could, however, potentially result in damage to interconnecting piping, depending on system configuration and sliding resistance of building foundation. Detailed evaluation of seismic capacities of interconnecting systems against differential building displacement cannot be made due to lack of design details and specific site conditions. It is assumed that the mode of failure due to differential building displacement has a capacity no less than the required margin of 1.67*SSE.

15.3.3.1 Reactor Building Complex Structures

Detailed fragility evaluations were made for the following structures in the reactor building (RB) and fuel building (FB) complex. The RB and FB share the same basement and are fully integrated. The term "reactor building" when mentioned hereafter also includes the structures of the fuel building. As for the containment structure, it is enclosed by and integrated into the RB.

- Building shear walls
- Containment wall (upper drywell and wetwell)
- RPV pedestal (same as lower drywell wall)
- RPV support brackets

Those structures were evaluated according to the approach outlined previously and using various safety factors as presented below.

The factor of safety for a structure against a specific failure mode is the product of the capacity factor F_c and structural response factor F_{rs};

$$F = F_c F_{rs} \tag{15.3-8}$$

The individual factors, the capacity factor and the response factor, are discussed in the following subsections.

15.3.3.1.1 Capacity Factor (F_c)

The capacity factor represents the capability of a structure to withstand seismic excitation in excess of the design earthquake. This factor is composed of two parts:

$$F_c = F_s F_u \tag{15.3-9}$$

Where:

F_s = the ultimate structural strength margin above the design SSE load, and

F_u = the inelastic energy absorption factor accounting for additional capacity of the structure to undergo inelastic deformations beyond yield.

The capacity estimated by this approach is the elastic capacity equivalent to the actual nonlinear behavior under strong motion earthquakes.

<u>Strength Factor (Fs)</u>

The strength factor associated with seismic load can be calculated using the following equation.

$$F_s = \frac{P_u - P_n}{P_s} \tag{15.3-10}$$

Where:

P_u = the actual ultimate strength,

P_n = the normal operating loads, and

P_s = the design SSE load.

The earthquake-resistant structural elements of the reactor building are reinforced concrete shear walls that are integrated with the reinforced concrete cylindrical containment through concrete floor slabs. The specified compressive strength of concrete is 34.5 MPa for the building and 27.5 MPa for the mat. The specified yield strength of reinforcing steel of ASTM A615, Grade 60 is 414 MPa. These are design values; the actual material strengths are higher.

Concrete compressive strength used for design is normally specified as a value at a specific time after mixing (28 or 90 days). This value is verified by laboratory testing of mix samples. The strength must meet specified values, allowing a finite number of failures per number of trials. There are two major factors that affect the actual strength:

a. To meet the design specifications, the contractor attempts to create a mix that has an "average" strength somewhat above the design strength, and

b. As concrete ages, it increases in strength.

Taking those two elements into consideration, the actual compressive strength of aged concrete is commonly 1.3 times the design strength (Reference 15-8). The total logarithmic standard deviation about the median strength is about 0.13.

According to the same reference, the ratio of the median yield strength to the specified strength of reinforcing steel is taken to be 1.2 with logarithmic standard deviation of 0.12.

The median yield strength of steel plates is typically 1.25 times the code specified strength with logarithmic standard deviation of 0.14 (References 15-8 and 15-9).

The reactor building shear wall is chosen as an example for the discussion of the strength factor evaluation. For reinforced concrete shear walls the ultimate shear strength can be computed using the following equation (Reference 15-1).

$$\begin{aligned} v_u &= v_c + v_s \\ &= 8.3\sqrt{f_c} - 3.4\sqrt{f'_c}\left(\frac{h}{w} - \frac{1}{2}\right) + \frac{N}{4wt} + \rho_{se} f_y \end{aligned} \qquad (15.3\text{-}11)$$

Where:

v_c	=	shear strength provided by concrete
v_s	=	shear strength provided by reinforcing steel
f'_c	=	concrete compressive strength
h	=	wall height
w	=	wall length
N	=	bearing load
f_y	=	yield strength of reinforcing steel
t	=	wall thickness
ρ_{se}	=	$A\rho_v + B\rho_h$

ρ_h = horizontal steel reinforcement ratio

ρ_v = vertical steel reinforcement ratio

A & B = constants depending on h/w:

	A	B
h/w< 0.5	1	0
0.5 ≤ h/w< 1.5	1.5-h/w	h/w-0.5
1.5 < h/w	0	1

In computing ultimate shear strength with this equation, the median material strengths of the concrete and reinforcing steel defined above are used and the wall bearing load is conservatively neglected.

The strength factor F_s is then calculated using Equation 15.3-10 for each of the levels of the reactor building shear walls. The normal operating loads do not result in lateral force and horizontal loads induced by SRV actuations are found to be negligible compared to the SSE-induced horizontal loads. Therefore, the strength factor is the ratio of the median shear strength to the design SSE shear. The lowest strength factor is found to be 1.82. This is calculated for the generic medium soil stiffness site condition that has the highest calculated seismic response. The associated logarithmic standard deviation is calculated to be 0.01 using the second moment approximation (Reference 15-10) accounting for both concrete and reinforcing steel material strength variability. There is also an uncertainty associated with Equation 15.3-11 since it is an approximate model fit to data. The modeling uncertainty is 0.20 expressed in terms of logarithmic standard deviation (Reference 15-1). The total composite logarithmic standard deviation in the median strength factor is 0.20, which is the SRSS value of 0.01 for the material strength uncertainty and 0.20 for the equation uncertainty. Flexural failure of the wall is found to have higher strength factor as such shear failure is the governing mode of failure.

<u>Inelastic Energy Absorption Factor (F_u)</u>

The inelastic energy absorption factor (F_u) accounts for the fact that an earthquake represents a limited energy source and many structures are capable of absorbing substantial amounts of energy beyond yield without loss of function. The parameter commonly used to measure the energy absorption capacity in the inelastic range is the system ductility, μ_{sys}. It is defined as the ratio of the summation of product of each story weight and median displacement at ultimate capacity to the summation of product of each story weight and story displacement at yielding of the critical story as shown below (Reference 15-1):

$$\mu_{sys} = \frac{\sum W_i \cdot \delta_{Ti}}{\sum W_i \cdot \delta_{ei}} \qquad (15.3\text{-}12)$$

Where:

W_i = weight of each story

δ_{Ti} = median maximum deflection of each story at ultimate capacity

δ_{ei} = median elastic deflection of each story scaled to reach yield in the critical story

A story drift of 0.5% is used to estimate the deflection profile at failure of the governing shear wall. Once the median system ductility is calculated, the median inelastic energy absorption factor is calculated using two different procedures, i.e., the Effective Frequency/Effective Damping Method and the Effective Riddell-Newmark Method (Reference 15-1) and the average value is the median inelastic energy absorption factor of the structure.

A median damping value of 7% of critical is conservatively assumed in the inelastic energy absorption factor calculation. This is to avoid double-counting of energy dissipation due to hysteresis damping and inelastic response of the building.

The inelastic energy absorption factor of the Reactor Building shear wall is calculated to be 1.8.. The associated randomness and uncertainty logarithmic standard deviations are 0.05 and 0.09, respectively determined from using the lower bound story drift of 0.36% (Table 3-5 of Reference 15-1).

15.3.3.1.2 Structural Response Factor (F_{rs})

The structural response factor (F_{rs}) consists of a number of factors or parameters introduced in the calculation of structural response in the seismic dynamic analysis. Response calculations performed in the design analysis utilized conservative deterministic parameters. The actual response may differ significantly from the calculated response for a given peak ground acceleration level since many of these parameters are random. The structural response factor is evaluated as the product of the following factors that are considered to have the most influence on the structural response.

$$F_{rs} = F_{gm}F_dF_{ssi}F_mF_{mc}F_{ecc} \qquad (15.3\text{-}13)$$

Where:

F_{gm} = ground motion factor accounting for the margin of the single envelope design ground response spectra with respect to the performance based seismic design spectra (Reference 15-11) and conservative or unconservative bias in the treatment of horizontal direction peak response and vertical component response.

F_d = damping factor accounting for the variability in response due to difference in expected damping at failure and damping used in the analysis,

F_{ssi} = soil-structure interaction factor accounting for the variability associated with SSI effects on structural response,

F_m = structural modeling factor accounting for the variability in response due to modeling assumptions,

F_{mc} = modal response combination factor accounting for the variability in response due to the method used in combining modal responses,

F_{ecc} = earthquake component combination factor accounting for the variability in response due to the method used in combining the earthquake components.

Ground Motion Factor (F_{gm})

Three factors are considered under the ground motion factor, i.e., spectral shape factor (F_{sa}), horizontal direction peak response (F_{HD}), and vertical component response (F_V) as presented in this section.

The ground response spectrum considered in the seismic design is the envelope of the 0.3g Regulatory Guide (RG) 1.60 site-independent ground spectra and the 0.5g North Anna ESP site-specific performance-based design ground spectra. The resulting single envelope design spectra are anchored to 0.5g peak ground acceleration as shown in Figure 15-2. The ground response spectrum used for the seismic margin assessment is also shown in Figure 15-2. In the frequency range lower than 9 Hz, a performance-based seismic design horizontal spectrum that bounds all the soil sites except Vogtle in the recent EPRI study (Reference 15-15) is developed. This is due to the fact that the Reg. Guide 1.60 spectra which dominate in the frequency range below 9 Hz are more like 84th percentile spectra or higher. The envelope of this spectrum and the North Anna ESP performance-based design spectrum, hereafter called as performance-based seismic design spectrum, is used for the seismic fragility calculation.

In accordance with the soil-structure interaction analysis performed and described in DCD Tier 2 Appendix 3A, generic medium soil stiffness site yields the highest seismic responses.. Therefore, the spectral shape factor is derived by comparing the single envelope design spectra with the performance-based seismic design spectra (see Figure 15-2) at the dominant frequency of the soil-structure system of medium site. The differences between these two spectra are the margins in the ground motion input. At the dominant frequency of 2.6 Hz of the reactor building in medium soil stiffness site, the 5% damped spectral accelerations of the two spectra are 0.93g and 0.59g, respectively. Thus the spectral shape factor is:

$$F_{sa} = 0.93g / 0.59g = 1.58 \qquad (15.3\text{-}14)$$

Similarly a spectral shape factor of 1.37 is calculated for soft soil stiffness site. In consideration that soil stiffness is likely to degrade at the acceleration level which building failure is expected, an average value of 1.47 is used for the spectral shape factor.

The logarithmic standard deviation of randomness in the spectral shape factor is the peak to valley variability of the performance-based seismic design spectra, which is 0.2 according to Reference 15-1. Since the Uniform Hazard Spectra (UHS) is derived from the probabilistic seismic hazard assessment, no uncertainty is assigned to the spectral shape factor to avoid double counting the uncertainty in the seismic hazard analysis.

Horizontal Direction Peak Response (F_{HD})

The ground motion parameter (e.g., peak ground acceleration) is the average of the two horizontal directions. Thus, the ground motion in one direction may be higher than that in the perpendicular direction. For a box-type structure such as the Reactor Building, seismic demand of a major shear wall is affected primarily by one directional horizontal response. The effect of earthquake in the perpendicular direction is insignificant. Since an average parameter is used, the real response could be either higher or lower, hence no bias either way. Thus,

$$F_{HD} = 1.0 \qquad (15.3\text{-}15)$$

The associated randomness and uncertainty are 0.13 and 0, respectively per Reference 15-1.

Vertical Component Response (F_V)

The vertical component of the ESBWR single envelope design spectra follows the Reg. Guide 1.60 vertical spectrum from 0.1 Hz up to 10 Hz and follows the North Anna performance-based design spectra above 10 Hz. This is conservative in comparison to the case where vertical component ground motion is assumed to be 2/3 of the horizontal component. Though relatively large randomness and uncertainty variability are associated with the vertical component (Table 3-2 of Reference 15-1), because of the small effect the vertical component has on the governing failure mode of the building (i.e., shear wall failure), they are significantly diminished in the final fragility parameters. Therefore,

$$F_V = 1 \qquad (15.3\text{-}16)$$

The associated randomness logarithmic standard deviation is 0.10. Therefore, the overall ground motion factor of safety is 1.47 (= 1.47*1.0*1.0) and the overall randomness is 0.26 by combining the randomness of spectral shape, horizontal direction peak response, and vertical component response per Equation 15.3-7.

Damping Factor (F_d)

For reinforced concrete structures the damping ratio considered in the SSE analysis is 7%. The realistic values when the stress is at or near yield range from 7 to 10% (Reference 15-14). The upper bound value is considered to be the median and the lower bound corresponds to the 84th percentile level.

Soil springs and dashpots are used in the soil-structure interaction modeling of the reactor building on generic sites. In such a soil-structure interaction system, the damping value of the building structure has less significant effect on the overall response of the building since soil modes are dominant. Thus, a factor of safety of unity is assigned to the damping factor.

$$F_d = 1 \qquad 15.3\text{-}17)$$

The associated logarithmic standard deviation can be estimated using the ratio of the amplification factor at 84th percentile damping (AF_{bd}) to the amplification factor at median damping (AF_{md}) at the same

$$\beta c = \ln (AF_{bd} / AF_{md}) \qquad (15.3\text{-}18)$$

Since conservatism in the structure hysteretic damping of the design seismic response analysis is neglected above, no value is assigned to the uncertainty logarithmic standard deviation.

Soil-Structure Interaction Factor (F_{ssi})

The factor of safety of soil-structure interaction between the reactor building and the supporting media includes the following considerations

- Ground motion incoherence (F_{GMI})
- Vertical spatial variation of ground motion (F_{VSV})
- SSI analysis (F_{SSI})

The dominant frequency of the SSI system of reactor building founded on uniform half space of medium soil stiffness site is 2.6 Hz. At this frequency the ground motion incoherence effects is insignificant, therefore, $F_{GMI} = 1.0$ and there is no variability associated with it.

The vertical spatial variation factor is to account for conservative bias in the SSI analysis that arises from choice of location of the control motion. The ground motion at the surface level in the free field decreases with depth of embedment. The ESBWR single envelope design ground spectra are defined as the outcrop motion at the foundation level of the reactor building for all site conditions. This conservative bias may be quantified if the surface ground motion is deconvoluted from the finished grade to the foundation level. Due to lack of this information, the embedment effect is estimated by using the design floor response spectra at the reactor building basemat for the medium site condition and the layered site condition. For the layered site cases, the surface motion calculated from SHAKE is used as input for the SASSI calculation. The factor of safety due to embedment effect is determined to be 1.22. No reduction due to embedment is estimated at three standard deviations from the median case. Based on this the associated uncertainty variability is calculated to be 0.07. The randomness variability is estimated to be 0.08 per Reference 15-1.

Furthermore, the median strength factor calculated in Section 15.3.3.1.1 is based on seismic demand of the medium site conditions. It is expected that soil degradation will occur at the ground acceleration level where the reactor building failure is calculated. It is observed from Appendix 3A of DCD that seismic responses of the soft soil site are lower than that of medium soil site. Therefore, the third factor of safety under the soil-structure interaction factor is to account for this effect and a factor of safety of 1.31 is calculated using the average responses of medium soil and soft soil site conditions.

The final SSI factor of safety is 1.6 (= 1.22*1.0*1.31) and the associated randomness and uncertainty variability are 0.08 and 0.28, respectively.

Modeling Factor (F_m)

The reactor building structural model considered in the seismic design analysis is a multi-degree-of-freedom system constructed according to common modeling techniques and the Standard Review Plan (SRP) requirements in terms of number of degrees of freedom and subsystem decoupling. The model is thus considered to be best estimate and the resulting dynamic characteristics to be median-centered. The modeling factor is thus unity. Uncertainty in the modeling has effects on the mode shapes and modal frequencies of the structure. For soil sites, frequency uncertainty of the soil-structure system is primarily due to uncertainty in soil properties and soil degradation at higher strain levels. Such uncertainty is estimated in the soil-structure interaction factor of safety. Thus, no uncertainty in response due to frequency uncertainty is included. A logarithmic standard deviation of 0.15 is estimated to account for uncertainty in the mode shape per Reference 15-1.

Modal Response Combination Factor (F_{mc})

The method used in the seismic response analysis is the time history method solved by direct integrations. The phasing between individual modal responses is known and the total response is the algebraic sum of all modes of interest. The maximum response is thus precise and the modal response combination factor (F_{mc}) is unity. The associated uncertainties are less than the uncertainties associated with the response spectrum method, in which the maximum modal responses are combined by the SRSS method. Therefore, a nominal value of 0.05 is assigned to the logarithmic standard deviation of randomness.

Earthquake Component Combination Factor (F_{ecc})

The effects of multi-directional earthquake excitation on structural response depend on the geometry, dynamic response characteristics, and relative magnitudes of the two horizontal and the vertical earthquake components. The design method to combine the contributions from different earthquake components is SRSS or 100-40-40. Either method is considered to result in a median-centered response. The earthquake component combination factor is 1.0.

The reactor building walls are designed to resist in-plane loads. The walls mainly respond to the horizontal motion parallel to the walls. The vertical loads on the walls due to the vertical excitation are typically less significant in contributing to the total stresses and there is an equal probability of acting upward or downward. The earthquake component combination effect on the wall design is thus not significant and a small logarithmic standard deviation of 0.05 is estimated.

15.3.3.1.3 Fragility Results for Reactor Building Complex

The result of the fragility analysis for the identified reactor building failure mode is summarized in Table 15-2. The overall safety factor is the product of the individual factors. The total logarithmic standard deviation is the SRSS value of the individual logarithmic standard deviations. The seismic fragility, in terms of median ground acceleration, is the product of the overall factor and the SSE design ground acceleration of 0.5 g. The HCLPF calculated in accordance with Equation 15.3-7a is presented at the bottom of the table.

15.3.3.2 RCCV and RPV Pedestal

Other major structures inside the reactor building are the reinforced concrete containment vessel (RCCV) and the Reactor Pressure Vessel (RPV) pedestal. The pedestal is part of the RCCV pressure boundary. Both the RCCV and the pedestal are reinforced concrete cylindrical structures interconnected to the reactor building via walls and slabs as such they respond to the seismic input motion as an integral unit.

The governing failure mode of the RCCV is shear failure of the cylindrical wall below the RCCV. The critical location is determined by calculating the ratio of capacity to demand at different locations of the containment wall. This cylindrical wall is not part of the RCCV pressure boundary, but it is on the seismic load path of the RCCV. The median shear capacity of the cylindrical wall is based on the equations in Appendix N of Reference 15-2 that are developed from a considerable amount of testing conducted in Japan on scale models of reinforced and prestressed concrete containment structures. The equation is as shown below:

$$v_u = 0.8\sqrt{f_{c\_m}} + \rho\sigma_y \leq 21.1\sqrt{f_{c\_m}} \qquad (15.3\text{-}19)$$

Where f_{c_m} is the median compressive strength of concrete of the containment wall (psi)
σ_y is the median yield strength of containment wall reinforcing steel (psi)
ρ is the effective reinforcing steel ratio of the containment wall

The median shear capacity of the cylindrical wall is

$$V_u = \frac{v_u \cdot \pi \cdot D \cdot t_w}{\alpha} \tag{15.3-20}$$

Where D is mean diameter of the cylindrical wall

t_w is thickness of the containment wall

α is a factor to convert the cross section area into effective shear area

$$\begin{aligned} \alpha &= 2.0 \text{ if } \frac{M}{V \cdot D_0} \le 0.5 \\ &= 0.667 \cdot \left(\frac{M}{V \cdot D_0} \right) + 1.67 \text{ if } 0.5 \le \frac{M}{V \cdot D_0} \le 1.25 \\ &= 2.5 \text{ otherwise} \end{aligned} \tag{15.3-21}$$

Where M and V are overturning moment and story shear at the section where median capacity is calculated.

The flexural strength of the cylindrical wall is found to have higher factor of safety than that of shear. The other factors of safety are calculated similar to that of reactor building. The median seismic capacity of the RCCV is 5.41g peak ground acceleration (pga) with an associated combined logarithmic standard deviation of 0.48. The HCLPF capacity is 1.75g pga. The summary table of the RCCV fragility is presented in Table 15-3.

The RPV pedestal is a thick-walled cylinder based on its geometry. The governing failure mode is tangential shear near the base. Flexural failure does not govern. The formula used for calculating the median shear strength of the pedestal is developed based on test data as discussed in Reference 15-16. The median seismic capacity of the RPV pedestal is 5.1g peak ground acceleration (pga) with an associated combined logarithmic standard deviation of 0.5. The HCLPF capacity is 1.59g pga. The summary table of RPV pedestal fragility is presented in Table 15-4.

15.3.3.3 RPV Support Brackets

The eight RPV support brackets are located at the junction of the RPV pedestal and the vent wall structure. The brackets are made of structural steel and they provide structural support to the RPV as well as the Reactor Shield Wall (RSW).

The structural integrity analysis of the RPV support bracket is documented in DCD Tier 2 Appendix 3G. The calculated stresses of normal, severe, extreme, abnormal, and abnormal extreme conditions of the RPV support brackets are presented in the appendix. The most severe case of stresses in the RPV brackets is identified in Table 3G.1-41 of Appendix 3G to be the vertical plate size 150 mm in compression. Anchorage of the brackets to the pedestal wall is found to have higher strength factor than the vertical plate of the bracket. It is noted that the vertical plates of the brackets are dimensioned such that plate buckling will not occur prior to yielding of the plate material. Therefore, median yield strength of the vertical plate material (i.e., A516 Grade 70) is used to calculate a median strength factor of 7.39. The inelastic energy absorption factor is unity since failure of the bracket is considered to be localized.

The maximum enveloping seismic forces acting on the support brackets are from seismic response of the fixed base model with in-fill concrete stiffness of vent wall (VW) and diaphragm floor (DF). The fundamental frequency of the RPV of the fixed base model is estimated at 12 Hz. At this frequency, there is no margin between the single envelope design spectra and the performance-based seismic design spectra (see Figure 15-2). Thus, the spectral shape factor of safety is unity. A ground motion incoherence factor of safety of 1.15 is calculated using the approach in Reference 15-1. The median seismic capacity of the RPV support brackets is 4.24g pga with an associated combined logarithmic standard deviation of 0.33. The HCLPF capacity is 2.0g pga. The summary table of RPV support bracket fragility is presented in Table 15-5.

15.3.3.4 Control Building Structure

The control building is a rectangular shape reinforced concrete box type structure. Its seismic fragility is evaluated using the same procedure described above for the reactor building. The controlling mode of failure is found to be shear failure of walls. Table 15-6 presents the margin in each of the capacity and response factors. The resulting median seismic capacity is 3.72 g pga with a logarithmic standard deviation of 0.51. The HCLPF capacity of the control building is 1.17g.

15.3.4 Component Fragility

The overall approach for determining HCLPF capacities of equipment and components qualified by seismic testing and analysis is described in EPRI TR-103959 (Reference 15-1). Since the detailed design information on the equipment is not available at this time, generic HCLPF capacities of 1.67*SSE are assigned. These generic HCLPF capacities assumed for equipment and components are considered achievable because of the margins or safety factors introduced at different stages of equipment design and qualification. Equipment qualified for application in GE ESBWR plants has additional seismic margins in high frequencies due to design consideration of high-frequency hydrodynamic loads in combination with seismic loads. The other sources of margin are from conservatism in the ESBWR seismic response analysis, e.g., use of single enveloping design spectra and conservative treatment of soil-structure interaction and the use of enveloping responses of all site conditions for design.

The equipment and components of the GE ESBWR plant will be qualified to the required floor response spectra arising from the single envelope ground motion input rich in both low and high frequencies and following the ASCE, ASME and IPEEE procedures, their seismic HCLPF capacities should be able to meet the required value of 1.67 times 0.5g peak ground acceleration.

Given the single enveloping design spectra of ESBWR and the performance-based seismic design spectra, it becomes obvious that the rock sites will be most challenging to meet the required HCLPF capacity if the building frequency is higher than 9 Hz. At 9 Hz and above, the single enveloping design spectra is same as the performance-based seismic design spectra such that the structural response factor will only be slightly greater than unity when other variables that would affect seismic response of the structures are considered. In such a case, the required response spectra (RRS) will be appropriately factored throughout the frequency range to assure that the HCLPF margin of 1.67 will be met.

15.3.5 Fragility Summary

The structural seismic fragilities and corresponding HCLPF values of the Reactor Building, the RCCV, the RPV pedestal, the RPV support brackets, and the Control Building are summarized in Table 15-1. All have HCLPF seismic capacities greater than 1.67 times the SSE.

15.8 REFERENCES

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15-15 Electric Power Research Institute "Assessment of Performance-Based Approach for Determining the SSE Ground Motion for New Plant Sites, V 1: Performance-Based Seismic Design Spectra", Product ID # 1012044, Final Report, June 2005.

15-16 Bader, M and H. Krawinkler, "Shear Transfer in Thick-Walled Reinforced Concrete Cylinders", 6th SMiRT, Paris, France, August 1981.

Table 15-1

Seismic Capacity Summary [3]

Structure/Component	Failure Mode	Capacity[1] A_m (g)	Fragility Combined[2] Uncertainty	HCLPF (g)
Reactor Building	Shear failure of wall	3.57	0.47	1.2
Containment	Shear	5.41	0.48	1.75
RPV Pedestal	Shear	5.1	0.5	1.59
RPV support brackets	Yielding of bracket	4.24	0.33	2.0
Control building	Shear	3.72	0.5	1.17

Notes to Table 15-1:

(1) Capacities are in terms of median peak ground acceleration.

(2) Combined uncertainties are composite logarithmic standard deviations of uncertainty and randomness

(3) HCLPF capacity for components that are significant contributors to overall plant level seismic margin is assumed to be 0.84g minimum which is 1.67 times SSE.

Table 15-2

Seismic Fragility for Reactor Building Shear Walls

Failure Mode:		Shear Failure of Wall Along Column Line R1			
Factor of Safety			**Median Value**	β_R	β_U
F_C	F_S	Strength	1.82	0.00	0.20
	F	Inelastic Energy Absorption	1.66	0.04	0.04
F_{RS}	F_{SA}	Spectral Shape			
		Response Spectrum Shape	1.47	0.20	0.00
		Horizontal Direction Peak Response	1.00	0.13	0.00
		Vertical Component Response	1.00	0.10	0.00
	F_D	Damping	1.00	0.00	0.00
	F_M	Modeling	1.00	0.00	0.15
	F_{MC}	Modal Response Combination	1.00	0.05	0.00
	F_{ECC}	Earthquake Component Combination	1.00	0.05	0.00
	F_{SSI}	Soil Structure Interaction			
		Ground Motion Incoherence	1.00	0.00	0.00
		Vertical Spatial Variation	1.22	0.08	0.07
		SSI Analysis	1.37	0.00	0.27
Overall Factor of Safety			**7.13**	**0.28**	**0.38**
A_d = Peak Ground Acceleration of the Single Evelope Design Spectra = 0.5g					
A_m = Median Peak Ground Acceleration = F*A_d = 3.57g					
β_C = Combined Logarithmic Standard Deviation = 0.47					
HCLPF = 1.2g					

Table 15-3

Seismic Fragility for Containment Wall

Component:		Cylindrical Wall Below Reinforced Concrete Containment Vessel (RCCV)			
Failure Mode:		Shear			
Factor of Safety			Median Value	β_R	β_U
F_C	F_S	Strength	3.16	0.00	0.21
	F	Inelastic Energy Absorption	1.49	0.06	0.11
F_{RS}	F_{SA}	Spectral Shape			
		Response Spectrum Shape	1.47	0.20	0.00
		Horizontal Direction Peak Response	1.00	0.13	0.00
		Vertical Component Response	1.00	0.10	0.00
	F_D	Damping	1.00	0.00	0.00
	F_M	Modeling	1.00	0.00	0.15
	F_{MC}	Modal Response Combination	1.00	0.05	0.00
	F_{ECC}	Earthquake Component Combination	1.00	0.12	0.00
	F_{SSI}	Soil Structure Interaction			
		Ground Motion Incoherence	1.00	0.00	0.00
		Vertical Spatial Variation	1.22	0.08	0.07
		SSI Analysis	1.28	0.00	0.24
Overall Factor of Safety			**10.82**	**0.30**	**0.38**
A_d = Peak Ground Acceleration of Single Envelope Design Spectra = 0.5g					
A_m = Median Peak Ground Acceleration = F*A_d = 5.41g					
β_C = Combined Logarithmic Standard Deviation = 0.48					
HCLPF = 1.75g					

Table 15-4
Seismic Fragility for RPV Pedestal

Component:		Reactor Pressure Vessel Pedestal			
Failure Mode:		Shear			
Factor of Safety			Median Value	β_R	β_U
F_C	F_S	Strength	3.29	0.00	0.25
	F	Inelastic Energy Absorption	1.34	0.06	0.10
F_{RS}	F_{SA}	Spectral Shape			
		Response Spectrum Shape	1.48	0.20	0.00
		Horizontal Direction Peak Response	1.00	0.13	0.00
		Vertical Component Response	1.00	0.10	0.00
	F_D	Damping	1.00	0.00	0.00
	F_M	Modeling	1.00	0.00	0.15
	F_{MC}	Modal Response Combination	1.00	0.05	0.00
	F_{ECC}	Earthquake Component Combination	1.00	0.12	0.00
	F_{SSI}	Soil Structure Interaction			
		Ground Motion Incoherence	1.00	0.00	0.00
		Vertical Spatial Variation	1.22	0.08	0.07
		SSI Analysis	1.28	0.00	0.24
Overall Factor of Safety			**10.19**	**0.30**	**0.40**
A_d = Peak Ground Acceleration of Single Envelope Design Spectra = 0.5g					
A_m = Median Peak Ground Acceleration = F*A_d = 5.1g					
β_C = Combined Logarithmic Standard Deviation = 0.5					
HCLPF = 1.59g					

Table 15-5

Seismic Fragility for RPV Support Brackets

Component:		RPV Support Brackets			
Failure Mode:		Yielding of Vertical Plate of the Bracket			
Factor of Safety			Median Value	β_R	β_U
F_C	F_S	Strength	7.39	0.00	0.12
	F	Inelastic Energy Absorption	1.00	0.00	0.00
F_{RS}	F_{SA}	Spectral Shape			
		Response Spectrum Shape	1.00	0.20	0.00
		Horizontal Direction Peak Response	1.00	0.13	0.00
		Vertical Component Response	1.00	0.10	0.00
	F_D	Damping	1.00	0.00	0.00
	F_M	Modeling	1.00	0.00	0.13
	F_{MC}	Modal Response Combination	1.00	0.05	0.00
	F_{ECC}	Earthquake Component Combination	1.00	0.05	0.00
	F_{SSI}	Soil Structure Interaction			
		Ground Motion Incoherence	1.15	0.00	0.07
		Vertical Spatial Variation	1.00	0.00	0.00
		SSI Analysis	1.00	0.00	0.00
Overall Factor of Safety			**8.47**	**0.27**	**0.19**
A_d = Peak Ground Acceleration of Single Envelope Design Spectra = 0.5g					
A_m = Median Peak Ground Acceleration = F*A_d = 4.24g					
β_C = Combined Logarithmic Standard Deviation = 0.33					
HCLPF = 2.0g					

Table 15-6

Seismic Fragility for Control Building

Component:		Control Building			
Failure Mode:		Shear Failure of Wall Along Column Line CA			
Factor of Safety			Median Value	β_R	β_U
F_C	F_S	Strength	2.36	0.00	0.20
	F	Inelastic Energy Absorption	1.63	0.06	0.10
F_{RS}	F_{SA}	Spectral Shape			
		Response Spectrum Shape	1.42	0.20	0.00
		Horizontal Direction Peak Response	1.00	0.13	0.00
		Vertical Component Response	1.00	0.10	0.00
	F_D	Damping	1.00	0.00	0.00
	F_M	Modeling	1.00	0.00	0.15
	F_{MC}	Modal Response Combination	1.00	0.05	0.00
	F_{ECC}	Earthquake Component Combination	1.00	0.05	0.00
	F_{SSI}	Soil Structure Interaction			
		Ground Motion Incoherence	1.00	0.00	0.00
		Vertical Spatial Variation	1.00	0.00	0.00
		SSI Analysis	1.36	0.00	0.31
Overall Factor of Safety			**7.44**	**0.29**	**0.42**
A_d = Peak Ground Acceleration of Single Design Spectra = 0.5g					
A_m = Median Peak Ground Acceleration = F^*A_d = 3.72g					
β_C = Combined Logarithmic Standard Deviation = 0.5					
HCLPF = 1.17g					

Table 15-7

ESBWR Systems and Components/Structures Fragilities

System/Component as a function of Event Tree Node	A_m(g)	β_c	HCLPF(g)
PLANT ESS STRUCTURES (SI)			
- Reactor Building (FRBLDG)	3.57	0.47	1.2
- Containment (FCONT)	5.41	0.48	1.75
- RPV Pedestal (FPEDST)	5.1	0.5	1.59
- Control Building (FCTRBLDG)	3.72	0.5	1.17
- Reactor Pressure Vessel Support (FRPV)	4.24	0.33	2.0
DC POWER (DC)			
- Batteries (FBTR)			0.84
- Cable trays (FCTRAY)			0.84
- Motor control centers (FMCC)			0.84
REACTIVITY CONTROL SYSTEM (SCRAM)			
- Fuel assembly (FFASSY)			0.84
- CRD Guide tubes (FCRDGTB)			0.84
- Shroud support (FSHRSPT)			0.84
- CRD Housing (FCRDHS)			0.84
- Hydraulic control unit (FHYLTUT)			0.84
SRV (SRV)			
- SRV (FSRV)			0.84

Table 15-7

ESBWR Systems and Components/Structures Fragilities

System/Component as a function of Event Tree Node	A_m(g)	β_c	HCLPF(g)
STANDBY LIQUID CONTROL (SLCS)			
- Accumulator Tank (FACCT)			0.84
- Check valve (FCHV)			0.84
- Squib valve (FSQUV)			0.84
- Piping (FPIP)			0.84
- Valve (motor operated) (FMOV)			0.84
ISOLATION CONDENSER (IC)			
- Piping (FPIP)			0.84
- Heat exchanger (FICHEX)			0.84
- Valve (motor operated) (FMOV)			0.84
- Valve (nitrogen operated) (FNOV)			0.84
DPV (DPV)			
- DPV (FDPV)			0.84
GRAVITY-DRIVEN COOLING (GDCS)			
- Check valve (FCHV)			0.84
- Squib valve (FSQUV)			0.84
- Piping (FPIP)			0.84
VACUUM BREAKERS (VB)			
- Vacuum breaker valve (FVB)			0.84

Table 15-7

ESBWR Systems and Components/Structures Fragilities

System/Component as a function of Event Tree Node	$A_m(g)$	β_c	HCLPF(g)
PASSIVE CONTAINMENT COOLING (PCCS)			
- Heat Exchanger (FPCCSHEX)			0.84
- Piping (FPIP)			0.84
IC/PCC POOL INTERCONNECTION (PI)			
- Valve (motor operated) (FIC/PCCI)			0.84
FIRE PROTECTION WATER SYSTEM (FPW)			
- Pump (diesel driven) (FPUMPDD)			0.84

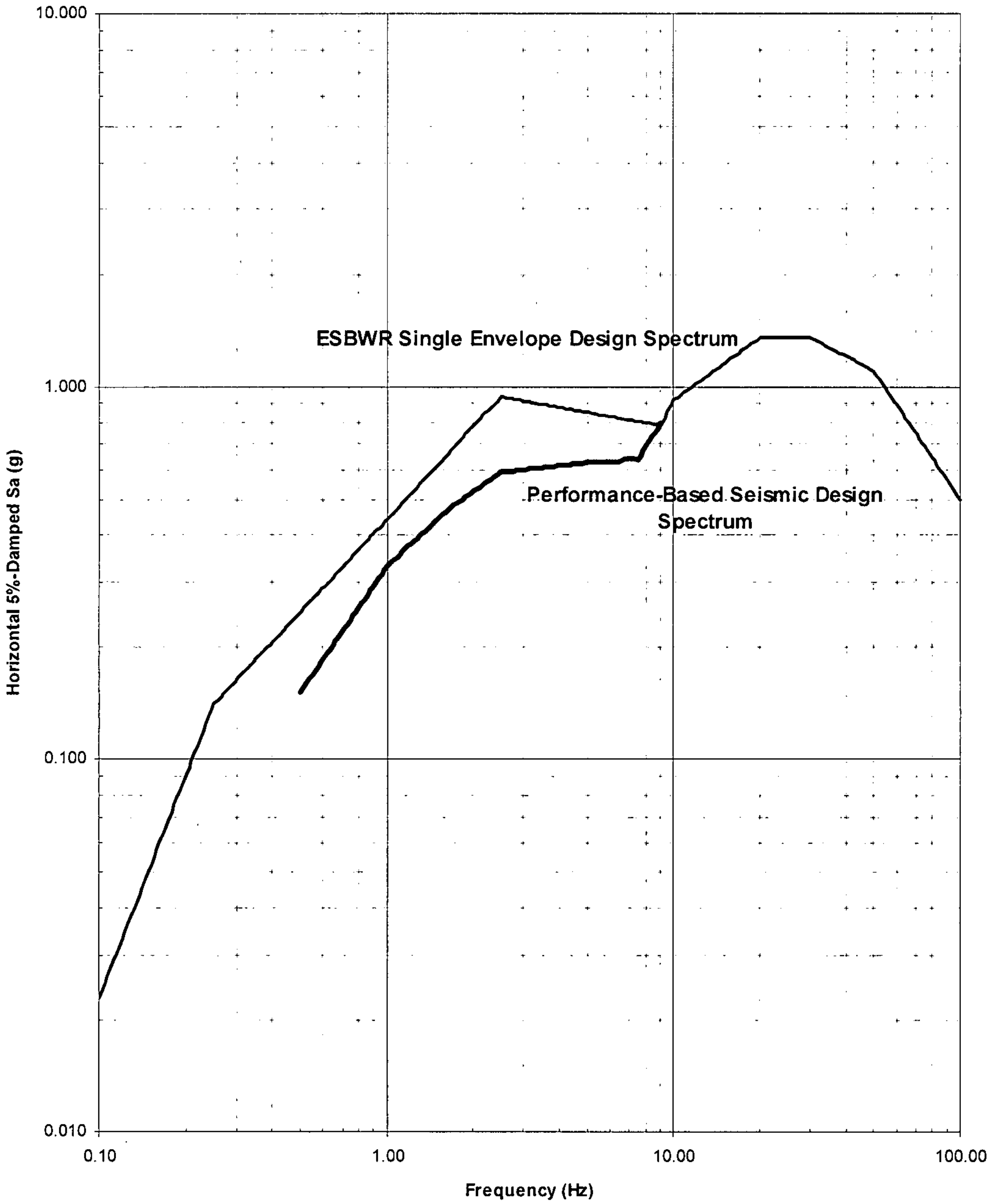

Figure 15-2. Horizontal Single Envelope Design and Performance-Based Seismic Design Spectra of ESBWR