19H Seismic Capacity Analysis

19H.1 Introduction

This subsection presents seismic capacities for selected structures and components that have been identified as potentially important to the seismic risk analysis of the ABWR standard plant. The seismic capabilities in terms of seismic fragilities are first estimated, from which the high confidence 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 0.3g for the Safe Shutdown Earthquake (SSE). Extensive seismic soil-structure interaction analyses of the reactor building and control building complex were performed for a wide range of generic site conditions under a 0.3g SSE. The analysis results in terms of site-envelope SSE loads are presented in Appendix 3A. 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.

For the seismic category I structures and components for which seismic design information is available, the seismic fragilities are evaluated using the factor of safety approach, which is called the Zion method in NUREG/CR-2300, PRA Procedures Guide (Reference 19H-1). This approach identifies various conservatisms and associated uncertainties introduced in the seismic design process and provides a probabilistic estimate of the earthquake level required to fail a structure or component in a postulated failure mode by linear extrapolation of the design information supplemented by judgement.

For certain safety-related components such as pumps, valves, and electrical equipment whose design details are not currently available, the generic seismic fragilities recommended in the EPRI ALWR Requirements Document, Appendix A PRA Key Assumptions and Groundrules (Reference 19H-2) or other data sources are used as appropriate. Those generic fragilities were chosen based on a review of prior PRAs and fragility data. They are considered achievable for the ABWRs with an evolutionary improvement in the seismic capacities of the components designed to a 0.3g SSE.

19H.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:

$$A = A_m \varepsilon_\gamma \varepsilon_\mu \qquad (19H\text{-}1)$$

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 nonexceedance probability (NEP) level Q.

$$P_f(A \le a|Q) = \phi\left[\frac{1}{\beta_\gamma}\ln\left(\frac{a}{A_m}\right) + \frac{\beta_\mu}{\beta_\gamma}\phi^{-1}(Q)\right] \qquad (19H\text{-}2)$$

where $\phi(.)$ is the standard Gaussian cumulative distribution function. Figure 19H-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 19H-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.

$$\beta_c = \sqrt{\beta_\gamma^2 + \beta_\mu^2} \qquad (19H\text{-}3)$$

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

$$P_f(A \le a) = \phi\left[\frac{1}{\beta_c}\ln\left(\frac{a}{A_m}\right)\right] \qquad (19H\text{-}4)$$

This composite fragility curve is also called the mean fragility curve and is shown as the dashed curve in Figure 19H-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.

$$A_m = FA_d \tag{19H-5}$$

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.

$$F = \prod_i F_i \tag{19H-6}$$

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

$$\beta_c = \sqrt{\sum_i \beta_{ci}^2} \tag{19H-7}$$

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

$$HCLPF = A_m \exp(-2.326\beta_c) \tag{19H-7a}$$

19H.3 Structural Fragility

19H.3.1 General

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 ductility 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 which 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 incredible since adjacent buildings are separated by more than 182 cm (6 feet). 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 generic piping fragility.

19H.3.2 Reactor Building Complex Structures

Detailed fragility evaluations were made for the following structures in the reactor building complex:

- Reactor building shear walls
- Containment
- Reactor pressure vessel pedestal

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};

$$F = F_c F_{rs} \tag{19H-8}$$

The individual factors in the capacity and response factors are presented in the following subsections.

19H.3.2.1 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:

$$F_c = F_s F_u \tag{19H-9}$$

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.

(1) Strength Factor (F_s)

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

$$F_s = \frac{P_u - P_n}{P_s} \tag{19H-10}$$

where:

P_u = the actual ultimate strength,

P_n = the normal operating and operation transient (i.e., SRV) loads, and

P_s = the design SSE load.

The earthquake-resistant structural elements of the reactor building are reinforced concrete shear walls which are integrated with the reinforced concrete cylindrical containment through concrete floor slabs. The reactor pressure vessel pedestal is of a composite steel-concrete construction consisting of two concentric steel shells filled with concrete in the annulus. In addition, stiffeners are welded to the steel shells. The specified compressive strength of concrete is 27.5 MPa. The specified yield strength of reinforcing steel of ASTM A615, Grade 60 is 414 MPa. The structural steel material for the pedestal shells and stiffeners is A572, Gr. 50, for which the specified yield strength is 345 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 which 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 19H-3). 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 (Reference 19H-3).

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 19H-5).

$$\begin{aligned} v_u &= v_c + v_s \\ &= 8.3\sqrt{f_c} - 3.4\sqrt{f'_c}\left(\frac{h}{w} - \frac{1}{2}\right) + \frac{N}{4wt} + \rho_{se} f_y \end{aligned} \tag{19H-11}$$

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
$0.5 < h/w < 1.0$	$2(1 - h/w)$	$2h/w - 1$
$1.0 < h/w$	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 19H-10 for each of the levels of the reactor building shear walls. The operating loads do not result in lateral shear 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 least strength factor is found to be 3.32. The associated logarithmic standard deviation is calculated to be 0.09 using the second moment approximation (Reference 19H-5) accounting for both concrete and reinforcing steel material strength variabilities. There is also an uncertainty associated with Equation 19H-11 since it is an approximate model fit to data. The modeling uncertainty is 0.15 expressed in terms of logarithmic standard deviation (Reference 19H-5). The total composite logarithmic standard deviation in the median strength factor is 0.17, which is the

SRSS value of 0.09 for the material strength uncertainty and 0.15 for the equation uncertainty.

(2) Inelastic Energy Absorption Factor (F_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 ductility ratio, μ. It is defined as the ratio of the maximum displacement to the displacement at yield. Newmark, Reference 19H-6, has shown that in the amplified acceleration range (approximately 2 to 8 Hz) the inelastic energy absorption factor F_u can be estimated by

$$F_u = \varepsilon\sqrt{2\mu - 1} \qquad \text{(19H-12)}$$

where ε is an error variable to account for the uncertainty associated with the use of this equation. This error variable is assumed to be lognormally distributed with a median of unity and a logarithmic standard deviation ranging from 0.02 to 0.1 (Reference 19H-7). For rigid structures (fundamental frequency above 20 Hz), the following equation given by Reference 19H-7 may be used.

$$F_u = \varepsilon\mu^{0.13} \qquad \text{(19H-13)}$$

Again, ε is an error variable of unit median and logarithmic standard deviation ranging from 0.02 to 0.1. For intermediate frequencies, the F_u factor can be interpolated from Eqs. 19H-12 and 19H-13.

According to Reference 19H-3, the system ductility ratio for reinforced concrete shear walls failing in shear is 2.5. The integrated building/containment system responds in multiple modes with predominant modes up to 10 Hz. The corresponding inelastic energy absorption factor is thus about 2.0 according to Equation 19H-12. The associated logarithmic standard deviation is 0.25 (Reference 19H-3). Flexural failures tend to be more ductile than shear failures. A ductility ratio of 4.0 is estimated and the corresponding F_u is 2.65 with logarithmic standard deviation of 0.25.

Steel structures are typically more ductile than concrete structures. When local buckling is prevented, the allowable ductility ratio is 5 (Reference 19H-8) for which the corresponding F_u is 3. The F_u factor is taken as unity when the failure mode is of a brittle type such as buckling or failure of high strength anchor bolts.

19H.3.2.2 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.

$$F_{rs} = F_{sa} F_d F_{ssi} F_m F_{mc} F_{ecc} \qquad (19H\text{-}14)$$

where:

F_{sa} = spectral shape factor accounting for the margin of the design ground response spectra with respect to the median centered spectra,

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.

(1) Spectral Shape Factor (F_{sa})

The ground response spectrum considered in the seismic design is the site-independent spectrum from Regulatory Guide (RG) 1.60, normalized to the design ground acceleration. To facilitate dynamic analysis using the time history method, artificial acceleration time histories of three directional components were generated so that the resulting spectra envelop the design spectra for the damping ratios of interest.

For the purpose of seismic risk assessment, the median ground spectrum given in NUREG/CR-0098 (Reference 19H-9) is considered to be the realistic input ground motion definition. The differences between the design spectra and median spectra are the margins in the ground motion input.

The spectral shape factor (F_{sa}) is defined to be the ratio of the amplification factor of the design spectrum to that of the median spectrum at the same frequency and damping level.

$$F_{sa} = AF_d / AF_m \tag{19H-15}$$

In constructing the median spectrum, the competent soil condition is conservatively assumed since it results in higher maximum ground velocity and displacement amplitudes than the rock condition for a same maximum ground acceleration. The design spectrum and median spectrum are compared at the 5% damping level for the maximum ground acceleration of 1g. The average spectral shape factors in representative frequency ranges are approximately

Frequency Range (Hz)	Average F_{sa}
2 to 10	1.34
10 to 20	1.20
20 to 33	1.07
above 33	1.00

The logarithmic standard deviation in the spectral shape factor is the variability in the median spectra which is 0.2 according to Reference 19H-2. No variability exists for frequencies above 33 Hz.

(2) Damping Factor (F_d)

The SSE loads were calculated using the SSE damping ratios specified in RG 1.61. The RG 1.61 damping values are considered to be quite conservative, particularly at response levels near failure. More realistic damping values are specified in Reference 19H-9.

For reinforced concrete structures the damping ratio considered in the SSE analysis is 7%. The realistic values at or near yield range from 7 to 10% (Reference 19H-9). The upper bound value is considered to be median and the lower bound corresponds to the 84th percentile level.

The RG 1.60 design ground spectra are used to evaluate the margin in response due to difference in actual damping at failure and design damping. The damping factor

F_d can be calculated to be the ratio of the amplification factor at design damping (AF_{dd}) to the amplification factor at median damping (AF_{md}) at the same frequency.

$$F_d = AF_{dd}/AF_{md} \tag{19H-16}$$

The associated logarithmic standard deviation can be calculated to be the natural log of the ratio of the amplification factor at 84th percentile damping (AF_{bd}) to the amplification factor at median damping (A_{md}) at the same frequency.

$$\beta_c = \ln(AF_{bd}/AF_{md}) \tag{19H-17}$$

For reinforced concrete structures the average damping factors and associated logarithmic standard deviations in representative frequency ranges are approximately

Frequency Range (Hz)	**Average F_d**	**Average β_c**
2 to 10	1.19	0.18
10 to 20	1.12	0.11
20 to 33	1.02	0.02
above 33	1.00	0.0

(3) Soil-Structure Interaction Factor (F_{ssi})

Seismic soil-structure interaction (SSI) analyses for the SSE were performed for the reactor building complex situated in a wide range of generic site conditions as described in Appendix 3A. The design seismic loads were established to be the site-envelope loads calculated by the SSI analyses. The site-envelope loads may have margins for a given site. The margin may be substantial if the specific site is a soft soil site. Since the ABWR standard plant is designed for generic site conditions, no credit is taken for site margins. Thus, the F_{ssi} factor is taken as 1.0. The associated logarithmic standard deviation is estimated to be 0.1.

(4) Modeling Factor (F_m)

The reactor building complex 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 the best estimate and the resulting dynamic characteristics are median centered. The modeling factor is thus unity. A relatively large logarithmic standard deviation of 0.15 is estimated to account for the complexity of the integrated reactor building and the containment design.

(5) Modal Combination Factor (F_{mc})

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

(6) 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 is SRSS, according to RG 1.92, which is considered to result in median-centered response. The earthquake component combination factor is 1.0.

The reactor building walls are designed to resist in-plane loads. The torsional effects were found to be small and 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.

Other major structures inside the reactor building such as the containment and the pedestal are cylindrical structures. The responses to the three orthogonal excitation components are essentially uncoupled. The logarithmic standard deviation is estimated to be 0.05.

19H.3.2.3 Reactor Building Complex Summary

The overall factor is the product of all individual factors. The total logarithmic standard deviation is the SRSS value of 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 0.3 g.

19H.3.3 Other Seismic Category I Structures

Seismic category I structures other than the reactor building structures in the ABWR standard plant include the control building structures.

The control building fragility is evaluated using the same procedure described above for the reactor building. The controlling mode of failure is shear of shear walls.

19H.4 Component Fragility

19H.4.1 General

Seismic fragilities of safety-related components were assessed for the following two categories of components:

(1) ABWR specific components whose fragility evaluation is made according to existing design information.

(2) Generic components whose fragilities are based on the data recommended in Reference 19H-2 or other data sources as appropriate.

19H.4.2 ABWR Specific Components

Detailed seismic fragility evaluations are performed for the following ABWR specific components:

- Reactor pressure vessel (RPV)
- Shroud support
- Control rod drive (CRD) guide tubes
- CRD housings
- Fuel assemblies

The design seismic loads for these components were calculated directly using a coupled building structures and RPV/internals model. Consequently, no subsystem dynamic analyses using input motions at support points were required. Therefore, the fragility evaluation procedures used for the reactor building structures as presented previously are also applicable to these specific components.

Reactor Pressure Vessel (RPV)

The failure of the RPV due to an earthquake results in a sequence similar to a large break loss-of-coolant accident, with the exception that there may be no means to provide makeup (i.e., injection or cooling) to the core. The ABWR RPV is supported by a conical skirt which is anchored to the pedestal with 120-68 mm minimum diameter high-strength anchor bolts. At an upper elevation, the RPV is laterally restrained by stabilizers which are connected to the reactor shield wall.

Failure of the RPV support system would result in excessive RPV deflection which could induce failure of the connecting pipes. The ultimate capacity of the support system is provided by both the skirt and the stabilizers. In this analysis, the resistance capacity of the support system is conservatively limited to the yielding capacity of the stabilizers or the skirt, whichever is smaller.

The critical failure mode is found to be stabilizer yielding.

RPV Internal Components

The internal components examined for seismic fragilities include the shroud support, CRD guide tubes, CRD housings, and fuel assemblies. Failure of those components could potentially result in inability to insert the control rods to shut down the reactor.

As noted, the fuel assemblies are found to have the lowest seismic capacity among the RPV internal components. The failure mode is excessive deflection of the fuel channel. The maximum deflection that the channel can undergo without collapse is limited by the amount that would inhibit the control rod from insertion to achieve reactor scram. The scram limited deflection is larger than the channel deflection at yield. To assess the seismic capacity of the channel, the moment-deflection resistance function is conservatively assumed to be of perfect elasto-plastic. The strength margin is taken to be the ratio of the yielding moment to the SSE induced moment. The additional capacity due to inelastic deformation is accounted for with a ductility ratio equal to the scram-limited deflection divided by the yielding deflection.

19H.4.3 Generic Components

Detailed fragility evaluations for safety-related components other than those specific components presented above cannot be made at this stage of certification due to lack of design details.

The ABWR generic components of interest for this seismic risk analysis are the following:

- Cable trays
- Large flat-bottom storage tanks
- Air-operated valves
- Heat exchangers
- Off-site Power (transformers and ceramic insulators)
- Batteries and battery racks

- Electric equipment (chatter failure mode)
- Switchgear/Motor control centers
- Transformers (480V)
- Diesel generators and support systems
- Turbine-driven pumps
- Motor-driven pumps
- Diesel-driven pumps
- Small tanks (e.g., standby liquid control tank)
- Motor-operated valves
- Safety relief, manual, and check valves
- Hydraulic control units
- Heating, ventilation, and air conditioning ducting
- Air handling units/room air conditioners
- Piping
- Service water pump house

Their seismic fragilities and corresponding HCLPF values are selected from a review of ALWR recommendation (Reference 19H-2) and other PRA studies (References 19H-10 and 19H-11).

19H.5 COL License Information

19H.5.1 Seismic Capacity

The COL applicant shall determine the HCLPF values for the plant-specific/as-designed components corresponding to those generic components defined in Subsection 19H.4.3. The values should be compared to their assumed HCLPF values. It should be noted that only the capacities of important contributors (Section 19.8) need to be determined and compared. These important contributions are hereafter referred to as SMA SSCs for systems, structures, and components needed for consideration in the seismic margins assessment.

An explicit evaluation of HCLPF values of only the important contributors (Section 19.8) need to be performed. However, prior to the HCLPF evaluation it is essential to verify that the quality of construction of structures and installation of equipment and systems are in conformance with the certified design commitments and that the as-built structures systems and components meet all the applicable ITACC requirements. These important components are hereafter referred to as SMA SSCs for systems, structures, and components needed for consideration in seismic margins assessment.

The HCLPF calculations can be made using fragility analysis or the conservative deterministic failure margin (CDFM) approach. The location effects should be taken into account in determining the limiting capacity of the same component on different locations.

For structures, equipment and systems other than the important items metioned above, it is only necessary to verify that the site-dependent conditions are within the site envelope parameters in accordance with the procedure described in Subsection 2.3.1.2 or that site-specific SSE responses are bounded by those considered in the standard design, provided that the as-biult structures, systems and components are verified to be designed, constructed, installed and tested in accordance with Tier 2 and Tier 1 commitments. Otherwise, site-specific HCPLF capacities for these structures and components need to be established.

It is not necessary that in each case the HCLPF equal or exceed the value assumed in the margins analysis of the standardized design, especially since the NRC has judged that HCLPF=0.5 is acceptable. However, depending on the degree of difference and the significance of the component in accident sequences, an evaluation of the site-specific plant level HCLPF capacity may be needed. The level of acceptable seismic margin for the plant should be established in a manner consistent with that used in existing nuclear power plants.

The site should also be investigated for the potential of seismic-induced soil failure (liquefaction, differential settlement, or slope stability) at 1.67 times the site-specific SSE.

In order to increase confidence that the as-designed seismic capacities of the SMA SSCs are realized in the final constructed plant, a seismic walkdown shall be performed by the COL applicant according to the process as follows:

- Step 1—Preparation for Plant Walkdown
- Step 2—Plant Seismic Logic Model Walkdown
- Step 3—Assessment of As-Built SMA SSC HCLPF Values
- Step 4—Seismic Plant Walkdown
- Step 5—Plant Damage State and Plant Level HCLPF Calculations

These steps are discussed in detail in the remainder of this subsection.

Step 1—Preparation for Plant Walkdown

The SMA presented in Appendix 19I contains seismic logic models for the plant. These models include the seismic-induced failures that were considered necessary to be evaluated as part of the SMA. These failures, and the associated HCLPF values of the SMA SSCs shall be reviewed. In preparing for the plant walkdown, all appropriate information regarding these failures should be gathered. These include, but are not necessarily limited to:

- Piping and instrumentation drawings,

- Electrical one-line diagrams,
- Plant arrangement drawings,
- Detailed design drawings,
- Procurement specifications,
- Construction drawings (especially those concentrating on seismic detailing and load paths),
- Quality assurance records,
- Seismic analysis used for defining floor response spectra,
- Floor spectra used as required response spectra by vendors,
- Engineering analyses of seismic performance (especially for representative seismic anchorages), and
- Equipment qualification data/material test data.

Step 2—Plant Seismic Logic Model Walkdown

The walkdown will concentrate on the identification of potential systems interactions that could impact the performance of the front-line and support SSCs included in the models. The original SMA model considered in Appendix 19I included the most significant systems interactions (e.g., collapse of major buildings). However, it is necessary to assure that no other interactions exist in the as-built plant that were not included in the SMA model. The walkdown should include a thorough examination of the SSCs included in the SMA, including piping runs, cable trays, etc. During the walkdown process, the team should identify the presence of any SSCs whose failure could impact the performance of the SMA SSCs. Based on a review of the seismic event trees, it was considered appropriate to add the following systems to SMA SSCs for this step; RCIC one HPCF train, one LPFL train, and SLC.These could include such things as:

- Non-load bearing walls adjacent to SMA SSCs
- Non-safety components above or adjacent to SMA SSCs
- Hard surfaces within deflection range of SMA SSCs
- Flooding/deluge sources in the vicinity of SMA SSCs.

All such potential interactions should be identified, along with the failure mode that could impact the performance of the SMA SSCs. These are new failure modes based on as-built plant conditions. This must be done for 100 percent of the SSCs included in the event and fault tree models. These new failure modes should be added as basic events on the SMA fault/event trees as appropriate and be added to the list of SMA SSCs. In addition, the design information specified in Step 1 should be assembled for these new failures. Note that all future reference to

SMA SSCs is intended to refer to the expanded list, including the newly added system interactions.

Step 3—Assessment of As-Built SMA SSC HCLPF Values

For each SMA SSC, a compilation of the design characteristics that control the HCLPF value should be prepared. These design characteristics can be one of two things: either they directly contribute to the dominant failure mode(s) or to failure modes that are close to being dominant. The dominant failure modes(s) is defined as the failure modes(s), from the list of all potential failure modes that will cause the SSC to be unable to perform its safety function, whose HCLPF value is the lowest (or equal to the lowest). Thus, the reduction of the HCLPF value of this failure mode would result in a corresponding reduction in the HCLPF of the SSC. This being the case, the design characteristics that would be compiled would include all of the specific design conditions that directly contribute to the dominant SSC failure mode(s). Another way to express this is that any change in any one of these design conditions that results in a reduction in seismic capacity will directly cause a reduction in the SSC HCLPF value. In addition, they would also include all such conditions that directly contribute to SSC failure mode(s), if any, that could become the dominant failure mode if it were to have a “somewhat” lower HCLPF value. For the purpose of this review, “somewhat” is defined as about a 10 percent to 20 percent HCLPF reduction. Thus, these failure modes are those whose calculated HCLPF value is only on the order of 10 percent to 20 percent higher than the dominant failure mode.

The characteristics that would be identified could include such things as:

- Size, type and number of anchor bolts,
- Size, type and orientation of support members,
- Distance between rigid pipe supports (allowance for differential motion),
- Distance between components.

The specification of these characteristics should be quite definitive (i.e., numerical where possible).

Step 4—Seismic Plant Walkdown

Final determination of the as-built plant design characteristics affecting HCLPF values is required. This should take the form of a final plant walkdown of the SMA SSCs, and RCIC, one HPCF train, one LPFL train and SLC as noted in step 2. As a product of Step 3, a compilation of key design characteristics (those that control or could control the HCLPF value of the SMA SSCs) was prepared. The plant walkdown is intended to determine the extent to which these design characteristics exist in the plant. Each SSC should be inspected and the as-build condition compared with the key design characteristics.

It is not required to perform a detailed walkdown inspection of 100 percent of the SMA SSCs. A 100 percent “walk by” is sufficient. The “walk by” is intended to assure that there is a reasonable basis for the assumption that the HCLPF of broad classes of SSC are essentially the same (i.e., that the SSCs are of similar design and manufacture and are similarly anchored). For each group of SSCs for which this condition of similarity can reasonably be established by the "walk by", it will then be necessary to select one representative SSC from each group to be subjected to a more rigorous inspection. This inspection will be conducted in such a manner as to determine if the representative SSC is in agreement with the assumed design characteristics compiled in Step 3.

It is understood that it will not always be possible to visually determine the existence of all the key characteristics, since some of them may be embedded within walls or in other inaccessible places. In such cases, it will be acceptable to use the construction QA records as adequate demonstration that the as-build SSC has the design characteristics required. In all cases, the result of the seismic plant walkdown should be fully documented.

Step 5—Plant Damage State and Plant Level HCLPF Calculations

The final step in the process is to determine HCLPF values for each event sequence, each plant damage state and for the overall plant. This should be done using both the min-max and convolution approaches and reported in the same form as in the SMA in Appendix 19I.

19H.6 References

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19H-2 "ALWR Utility Requirements Document, Volume II, Chapter 1, Appendix A PRA Key Assumptions and Groundrules", EPRI, October 1988.

19H-3 "Report on Quantification of Uncertainties, Report of Seismic Analysis Main Committee", ASCE, March 15 1983.

19H-4 Not Used

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19H-6 Newmark, N. M., "Inelastic Design of Nuclear Reactor Structures and Its Implication on Design of Critical Equipment", SMIRT Paper K4/1, 1977 SMIRT Conference, San Francisco, 1978.

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19H-9 "Development of Criteria for Seismic Review of Selected Nuclear Power Plants", NUREG/CR-0098, May 1978.

19H-10 Harrison, S. W., Esfandiari, S., Pandya, D., and Ahmed, R., "Seismic Fragility Curves for Evaluation of Generic Electrical Conduit Supports, to be presented in the ASME PVP Annual Meetings", Honolulu, Hawaii, July 22-24, 1989.

19H-11 Campbell, R. D., Ravindra, M. K., and Bhatia, A., "Compilation of Fragility Information from Available Probabilistic Risk Assessments", LLNL, September 1985.

Table 19H-1 Not Used

Table 19H-2 Not Used

Table 19H-3 Not Used

Table 19H-4 Not Used

Table 19H-5 Not Used

Table 19H-6 Not Used

Table 19H-7 Not Used

Table 19H-8 Not Used

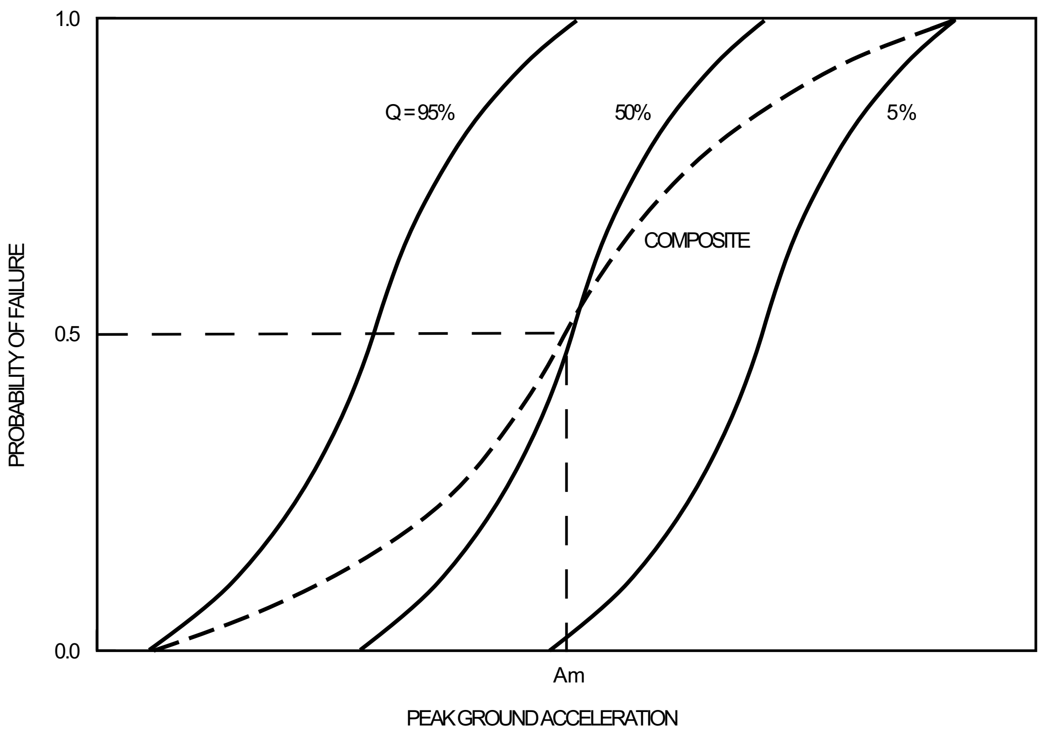

Figure 19H-1 Typical Fragility Curves