



SEISMIC PROBABILISTIC RISK ASSESSMENT OF NUCLEAR POWER PLANTS USING DETAILED STRUCTURAL MODELS

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ABSTRACT

Seismic Probabilistic Risk Assessment (SPRA) is a method employed to evaluate the frequency of adverse consequences in nuclear power plants (NPPs) resulting from seismic events. Seismic fragility assessment is a crucial component of this methodology, representing the failure probability of systems, structures, and components (SSCs) conditional to seismic intensity. In majority of SPRA studies, seismic fragilities derived from experience or historical shake table tests are used rather than detailed structural models of actual SSCs. The general practical approach is to utilize simplified methods (e.g., separation of variables (SOV) and hybrid methods) to estimate seismic fragility of SSCs. However, in the literature, the level of conservatism in those approaches has not been investigated comprehensively. This paper examines the impact of choice of fragility approach on the seismic fragility for a selected equipment, namely hydro-accumulator. 3D finite element (FE) models of an NPP structure were developed using SAP2000 software (2023) and its components were modeled using Abaqus/CAE software (2020). The seismic fragility of the component was evaluated using four different approaches: 1) incremental dynamic analysis with FE method (FEM), 2) SOV method, 3) hybrid method, and 4) experience-based approach. Based on the research conducted, the followings are the major conclusions: 1) it was found that SOV and hybrid methods provide more conservative results than IDA with FEM, and 2) experience-based fragility approach calculates the lowest failure probability among the four approaches.

INTRODUCTION

Seismic Probabilistic Risk Assessment (SPRA) has been employed in the nuclear engineering field since the 1980s to assess the severe accidents that may be induced by earthquakes (e.g., Kennedy et al., 1980; Segarra et al., 2023). Xie et al. (2018) divided the SPRA procedure into five key steps, namely: i) seismic hazard analysis, ii) seismic demand evaluation, iii) seismic fragility assessment, iv) plant-system analysis, and v) seismic risk quantification. The purpose of seismic hazard analysis is to characterize the seismic intensity of a potential earthquake at a specific site in terms of occurrence frequency. The output includes seismic hazard curve and uniform hazard response spectrum (UHRS), which are needed to generate design basis earthquake (DBE) ground motions (GMs). Seismic demand evaluation aims to quantify seismic input (i.e., in-structure response spectra or time histories) at the floor level that critical systems, structures, and components (SSCs) are subjected. The seismic fragility assessment is essential for calculating the conditional failure probability of SSCs at given seismic intensity measures (IMs), such as peak ground acceleration (PGA) or peak ground velocity (PGV). Seismic-induced accident scenarios are commonly identified using event tree/fault tree methodology with plant-system analysis. Then, the likelihood of undesired consequences, such as core damage frequency (CDF), is estimated through seismic risk quantification.

Kennedy et al. (1980) pioneered the seismic fragility approach which was employed to represent the seismic-induced failure probability of SSCs utilized in Oyster Creek NPP. This approach significantly relies on subjective engineering judgment introduced with safety factor method. Another conservative approach involves using experience-based (generic) seismic fragilities. It has been similarly developed based on expert judgment and past seismic experience. Currently, they are employed for rapid preliminary seismic

assessments in the industry when plant-specific fragilities are unavailable. They are also commonly used in academic studies owing to their ease of implementation. The current seismic guide presents two numerical approaches: 1) separation of variables (SOV) method, and 2) hybrid method (EPRI, 2018). These methods are commonly used in practice with simplified structural models, introducing significant variabilities into their fragilities to accommodate the approximation of the methods. However, the lack of realism in estimating the median capacity of components and the assumed dominance of the fundamental period versus the complete vibrational response raise concerns about the credibility of the analysis, especially for seismic fragility assessment of structurally complex equipment such as water tanks. Moreschi et al. (2012) highlighted the importance of advanced structural analysis and modeling techniques (e.g., finite element method (FEM), dynamic time history analysis) for evaluating in-structure response spectra (ISRS). Incremental dynamic analysis (IDA) is one of the most widely used approaches employed to assess seismic fragility of SSCs with these advanced structural methodologies. In this study, IDA and FEM are used together to represent the state-of-the-art fragility methodology.

In this paper, a water tank, namely Stage 2 hydro-accumulator (HA-2), was selected as a benchmark equipment, and its seismic fragility was calculated using four approaches, namely: 1) IDA with FEM, 2) SOV method, 3) hybrid method, and 4) experience-based approach. The impact of choice of fragility approach on the seismic fragility of HA-2 was examined. For this purpose, a hypothetical NPP was created based on VVER-1000 design from available open-source references. Site-specific ground motions were developed for the seismic characteristics of a site in Savannah, Georgia (U.S.) in accordance with ASCE 43-19 (2019). A 3D FE model was developed for the NPP structure using SAP2000 software (2023), and time history analyses were performed using the developed GMs. Then, ISRS and in-structure time histories (ISTHs) were computed specifically at the elevation level where HA-2 is located. For seismic fragility assessment with the-state-of-art methodologies, a 3D FE model was developed for HA-2 using Abaqus/CAE software (2020), and fluid-structure interaction was considered using arbitrary Lagrangian-Eulerian (ALE) approach. Through IDA, seismic fragility was computed. Also, the seismic fragility of the same equipment was calculated using SOV and hybrid methods following the steps outlined in the EPRI (2018). Additionally, the experience-based seismic fragility of HA-2 was generated from the recommended fragility parameters provided for accumulators in NRC (2008).

GROUND MOTIONS AND STRUCTURAL MODEL DEVELOPMENT

This section provides a summary of the procedure followed to develop site-specific design response spectra (DRS) and compatible DBE ground acceleration histories for a hypothetical site in Savannah, Georgia, U.S. Additionally, it outlines the evaluation of seismic demand demonstrated for the representative NPP. The horizontal design response spectrum that corresponds to target annual frequency of exceeding level of 10^{-5} yr⁻¹ was computed in accordance with ASCE 43-19. For the seismic characteristics of the site, UHRS and seismic hazard curve provided in EPRI (2013) were used. From NGA-West2 database (PEER, 2017) 20 seed GMs were selected using PEER's GM search tool and then modified using spectral matching technique through RspMatch09 software (Atik and Abrahamson, 2010) (Figure 1a).

The representative NPP system was developed based on the VVER-1000 design from the details provided in ARIS (2011) and Krutzik (1995). The NPP structure is made of reinforced concrete and contains four substructures elevated above its base structure (BS). These substructures include reactor building (RB), inner containment (IC), outer containment (OC), and surrounding building (SB). A 3D FE model was constructed in SAP2000 (2023) using shell elements (Figure 1b). The influence of soil behavior on structural model was neglected, and fixed boundary condition was used at the bottom of the FE model. Damping ratio, unit weight, and elasticity modulus were sampled from their distributions using LHS method, and ISRS and ISTHs were calculated through time history analyses. Table 1 presents the median values and logarithmic standard deviations (also referred to as beta or β) of the structural parameters along with their respective references.

Table 1: Material properties of reinforced concrete and corresponding uncertainty parameters

Variables	Median	Beta (β)	Distribution	Reference
Elasticity modulus (ksi)	4,475	0.12	Lognormal	EPRI (2018)
Unit weight (pcf)	150	0.05	Lognormal	NEUP (2017)
Damping ratio	4%	0.29	Lognormal	EPRI (2018)

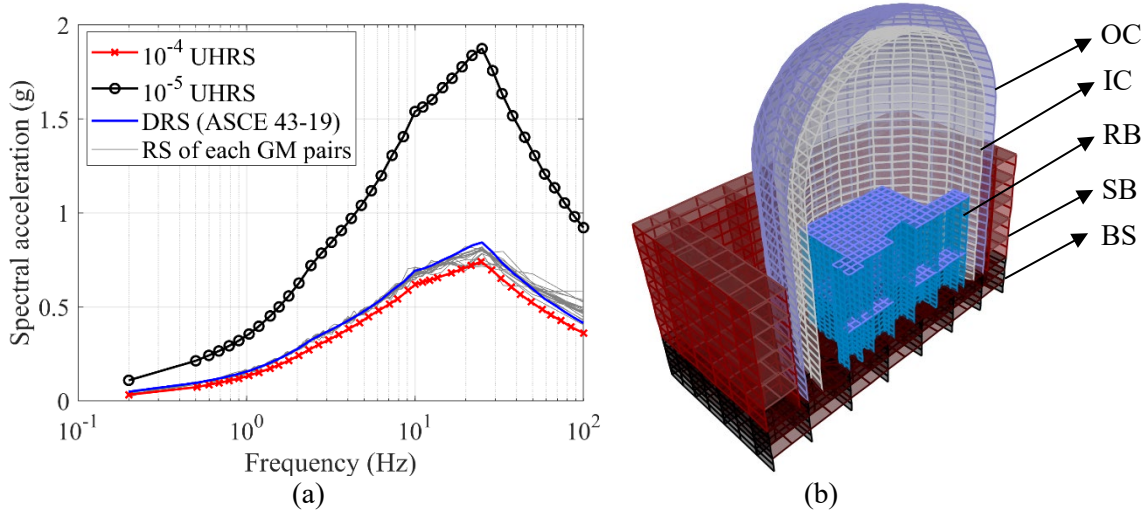


Figure 1: (a) Response spectra (%5 damped), and (b) cross section of FE model of NPP structure in SAP2000 (2023)

SEISMIC FRAGILITY ASSESSMENT

In SPRA, the general approach accounts for aleatory (randomness) uncertainty, β_R , and epistemic (modeling and data) uncertainty, β_U , separately, and incorporates them into a double lognormal model (Equation 1).

$$P_f = \Phi \left[\frac{\ln\left(\frac{a}{A_m}\right) + \beta_U \Phi^{-1}(Q)}{\beta_R} \right] \quad (1)$$

where $\Phi(\cdot)$ is the cumulative standard normal distribution function, a is ground acceleration variable, A_m is median seismic capacity, and Q is the probability of confidence level (e.g., 5%, 50%, 95%). For mean fragility curve, composite logarithmic standard deviation, β_C , is calculated using the square root of the sum of the squares (SRSS) method and mean conditional failure probability can be computed by replacing β_R with β_C and setting β_U equal to zero in Equation 1. In the following subsections, the four distinct fragility approaches, utilized for HA-2, are summarized. These approaches include: 1) incremental dynamic analysis with finite element method, 2) SOV method, 3) hybrid method, and 4) experience-based (generic) approach.

Incremental Dynamic Analysis with Finite Element Method

IDA stands out as a prevalent approach for assessing the effect of crucial IMs (e.g., PGA) that cause SSCs to reach the specified failure limit, such as story drift ratio, elastic strength limit etc. The Fourier amplitudes of a set of ground motion histories are adjusted to the desired minimum intensity level (e.g., typically 0.1

PGA). Subsequently, time history analyses are performed by gradually scaling the Fourier amplitudes of these histories up to the interested maximum intensity level (e.g., 2.5 PGA in this study). The goal is to identify the onset IMs that lead to structural failure. Then, a fragility curve can be developed from the identified onset IMs using the maximum likelihood estimation (MLE) method. For more in-depth information, readers are directed to Baker (2015).

FEM is a powerful tool to simulate the deformational behavior of structures under specified load conditions. In this study, the integration of IDA and FEM is considered as the state-of-the-art seismic fragility approach (labeled as IDA&FEM in the figures). A detailed 3D FE model was developed in Abaqus/CAE software (2020) to estimate the structural response of HA-2 considering its complex geometry and the challenges associated with capturing sloshing effects (Figure 2a). Arbitrary Lagrangian-Eulerian (ALE) method was utilized to realistically model fluid-structure interaction. Structural parameters of the equipment were randomly selected from their associated distributions (Table 2) employing LHS. The FE model was subjected to ISTHs calculated at the floor level where HA-2 is mounted in RB. Implementing IDA approach, the reaction forces at the fixed end were obtained, covering an IM range from 0.1 to 2.5 PGA.

It is assumed that a total of 48 embedded anchor bolts with a diameter of 2 in. (5 cm) each are used at the bottom of HA-2, based on open-source information (Gidropress, 2017). After a comprehensive examination, anchor bolt failure was determined as the governing failure mode. Median bolt tension, P_{um} , and bolt shear, V_{um} , strengths were calculated using Equations 2 and 3.

$$P_{um} = 0.90\sigma_{um}A_{eff} \quad (2)$$

$$V_{um} = 0.62\sigma_{um}A_{eff} \quad (3)$$

where σ_{um} is median ultimate strength, which is 64 ksi (440 MPa), and A_{eff} is effective bolt area, which is 2.5 in.² (16.1 cm²). As per EPRI (2018), the logarithmic standard deviations of 0.13 and 0.10 for the tension and shear strength of anchor bolts, respectively, were considered using LHS. The overturning moment strength of HA-2 were determined following the approach presented for a water tank in EPRI (2018), and the onset PGAs were obtained. Employing MLE fitting technique, the mean fragility curve was generated from the empirical data (Figure 2b). Further details can be found in Baker (2015).

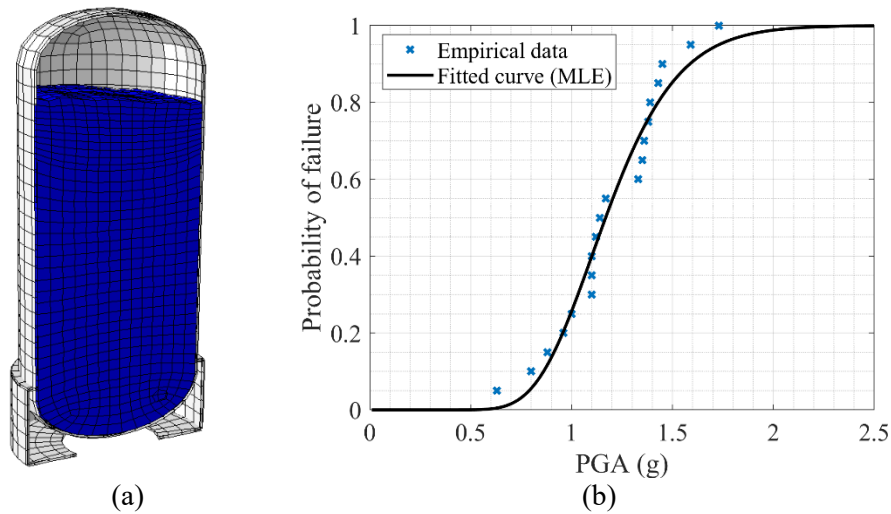


Figure 2: (a) FE model of HA-2 (Abaqus/CAE, 2020), and (b) its mean fragility curve obtained from IDA and FEM

Table 2: Material properties of steel and corresponding uncertainty parameters

Variables	Median	Beta	Distribution	Reference
Elasticity modulus (ksi)	29,000	0.033	Lognormal	Haselton (2006) and Melchers (1999)
Unit weight (pcf)	490	0.05	Lognormal	Osório and Vrouwenvelder (2011)
Damping ratio	4.0%	0.30	Lognormal	EPRI (2018)

Separation of Variables Method

With this approach, fragility curve is evaluated in two steps. First, median ground acceleration capacity is estimated from median structural properties (e.g., elasticity modulus, damping ratio, unit weight, strength) and median seismic demand (ISRS corresponding to the median DBE). Next, logarithmic standard deviations for each identified source of uncertainty are quantified separately. Subsequently, the combined aleatory and epistemic logarithmic standard deviations in median ground acceleration capacity are estimated using SRSS method, and fragility curve is generated by plugging median capacity, aleatory and epistemic logarithmic standard deviations into Equation 1.

For HA-2, the seismic demand was assessed by considering both impulsive and sloshing modes, following the methodology outlined in ASCE 4-16 (2017) and the demonstration example provided in Appendix S of the seismic guide (EPRI, 2018) for a vertical liquid tank. The effective impulsive and sloshing fluid weights were calculated. For the impulsive weight, the fundamental mode of HA-2 was calculated using Abaqus/CAE (2020), while the sloshing mode was estimated following the approach recommended in ASCE 4-16. Spectral accelerations were obtained from the median ISRS where HA-2 is mounted at (top floor of RB), and the median overturning moment was calculated at the bottom of HA-2. Following the same procedure described for IDA method, the median tension and shear strengths were calculated using Equations 2 and 3, and the median overturning moment capacity of the group of bolts was calculated. Subsequently, the scale factor was determined by incrementally increasing the median overturning moment demand until the bolt capacity was reached. Then, the median ground acceleration capacity of the equipment was calculated by multiplying F_S of 2.52 by the PGA corresponding to the DBE, which is 0.41 g.

Regarding uncertainty quantification, four primary variability sources were identified for HA-2, which are: 1) ground motion, 2) structure response, 3) equipment response, and 4) anchor bolt capacity. The variables associated with each main source and their corresponding logarithmic standard deviations were determined in accordance with EPRI (2018), as presented in Table 3. It should be noted that β represents the uncertainty in each variable (presented in the third column of Table 3), and further evaluation is required to quantify the logarithmic standard deviation in median ground acceleration capacity for each variable individually. To quantify this, the median parameter, such as median damping ratio, is increased or decreased by a factor corresponding to one standard deviation, and then the associated scale factor, $F_{S\sigma}$, is re-computed. Subsequently, the logarithmic standard deviation in scale factor, meaning in median seismic capacity as well, is calculated for the associated variable using Equation 4.

$$\beta = \ln(F_S/F_{S\sigma}) \quad (4)$$

For instance, two random variables were identified to account for the randomness of ground motion: 1) peak response in the horizontal direction, and 2) vertical component response. The recommended logarithmic standard deviations for these two variables in EPRI (2018) are 0.18 and 0.25, respectively. The spectral acceleration in the horizontal direction for the median DBE, S_{ahm} , and the median scale factor were calculated as 2.23 g and 2.52, respectively. The one-standard-deviation-increased spectral acceleration in the same direction, $S_{ah\sigma}$, was determined as 2.68 g using Equation 5.

$$S_{ah\sigma} = S_{ahm}e^{\beta} \quad (5)$$

The scale factor corresponding to $S_{ah\sigma}$ was calculated as 2.13, and the logarithmic standard deviation in the scale factor was determined as 0.17 using Equation 4. For $S_{ah\sigma}$, the logarithmic standard deviation in the variable and median seismic capacity are identical since the seismic demand is dominated by the horizontal direction. However, for random variable of vertical component response, the logarithmic standard deviation in the median capacity was calculated as 0.02, while the variability in this variable is 0.25. This is associated with the limited influence of the vertical component on the seismic demand. Following the same procedure, variabilities in tension and shear strengths of anchor bolts were evaluated, and the corresponding variabilities in the median seismic capacity were calculated (See Table 3).

The variabilities relating to structural parameters of the NPP structure in median seismic capacity were calculated by reevaluating seismic demand for plus- and/or minus-one-standard-deviation parameters, separately. For example, response analysis was performed for a damping ratio of 2.8%, corresponds to one-minus-standard-deviation damping (median damping ratio is 4.0%). Then, a new spectral acceleration was calculated for the equipment, and the associated logarithmic standard deviation in the seismic capacity was calculated using Equation 4. To quantify the variability resulting from the uncertainty in structural frequency, three cases were created for the structure: 1) lower bound (LB), 2) best estimate (BE), and 3) upper bound (UB). The LB and UB were established for the plus- and minus-one-standard-deviation by modifying the elasticity modulus, respectively, whereas BE represents the median frequency of the structure. Dynamic time-history analyses were performed for the additional two cases, and the associated ISRS were computed. The seismic guide (EPRI, 2018) recommends quantifying the variability of structure frequency for equipment fragility using Equation 6.

$$\beta = \ln(S_{a84\%}/S_{am}) \quad (6)$$

where $S_{a84\%}$ is 84% non-exceedance probability (NEP) spectral acceleration, and S_{am} is median spectral acceleration. According to the seismic guide (EPRI, 2018), 84% NEP and median ISRS can be computed by enveloping and averaging the ISRS obtained from the three cases, respectively. Following this approach, the corresponding logarithmic standard deviation was calculated for this variable. Regarding the randomness of damping ratio, ISRS were regenerated for minus-one-standard-deviation damping ratio. Then, the scale factor was recalculated, and the associated logarithmic standard deviation was determined using Equation 4. Similarly, for the uncertainty of fundamental frequency of equipment, spectral acceleration was obtained for plus- and minus-one-standard-deviation frequencies and the associated variability in the seismic capacity was determined based on the largest logarithmic standard deviation of these two cases (Table 3).

On the other hand, variabilities of some uncertainty sources (i.e., fidelity of the responses of structure and equipment, the response phasing, and direction combination) need to be determined based on engineering judgment. A logarithmic standard deviation of 0.05 was considered for these four variables in accordance with EPRI (2018). For time-history phasing, it is necessary to incorporate a logarithmic standard deviation of 0.15 to account for both randomness (β_R) and uncertainty (β_U) separately when a single ground motion is employed in assessing seismic demand. If five ground motions are utilized, the seismic guide permits to compute the logarithmic standard deviation for time-history phasing randomness using Equation 7, and no consideration for uncertainty is required (as presented in the ninth row of Table 3).

$$\beta = \ln(ZPA_{MAX}/ZPA_m)/Z \quad (7)$$

where ZPA_{MAX} and ZPA_m are the maximum and median zero period accelerations (ZPAs) of the five time ground acceleration histories, respectively, and Z is number of logarithmic standard deviations

judged to separate the median and maximum ZPAs, taken as 1.282 for median to 90% NEP in accordance with EPRI (2018). In this study, two cases were established for each approach: 1) considering 1 GM, and 2) considering five GMs. The logarithmic standard deviations in seismic capacity were evaluated for each uncertainty source, presented in the fourth to seventh columns of Table 3. Using SRSS method, the combined β_R and β_U values were calculated for HA-2, and the fragility curves were generated as illustrated in Figure 3a. To reduce the dependency on response spectra from a single and selected five GMs, 20 simulations were generated, and the mean fragility curves were obtained for these two cases. In Figure 3a, the curve labeled '1 GM' represents the mean of 20 GMs, while the '5 GMs' signifies the mean of 20 combinations (each created by randomly selecting five GMs from the pool of 20 GMs).

Table 3: Uncertainty parameters calculated from SOV fragility approach

Uncertainty Sources	Reference sections (EPRI, 2018)	β	Conservative (1 GM)		Recommended (5 GMs)	
			β_R	β_U	β_R	β_U
Ground motion						
Horizontal direction peak response	5.4.1.2	0.18	0.17		0.17	
Vertical component response	5.4.1.3	0.25	0.02		0.02	
Structure response						
Frequency	5.4.3.1	0.15		0.32		0.32
Damping	5.4.2	0.30	0.21		0.21	
Fidelity	5.4.3.2	0.05		0.05		0.05
Time history phasing	5.4.4.1	0.15	0.15	0.15	0.09	0
Equipment response						
Frequency	5.5.2.3	0.15		0.04		0.04
Damping	5.5.2.2	0.30	0.09		0.09	
Fidelity	5.5.2.3	0.05		0.05		0.05
Response phasing	5.5.2.4	0.05		0.05		0.05
Direction Combination	5.4.6	0.05	0.05		0.05	
Anchor bolt capacity						
Tension strength	4.7.1	0.13	-	0.12	-	0.12
Shear strength	4.7.1	0.10				
Combined			0.33	0.39	0.30	0.35

Hybrid Method

This approach uses high confidence of low probability of failure (HCLPF) capacity relationship and generic uncertainty parameters to estimate fragility curve. As per EPRI (2018), it is assumed that the ground acceleration capacity corresponding to a 1% conditional probability of failure on the mean fragility curve ($A_{1\%}$) is equivalent to the ground acceleration capacity associated with a 5% conditional probability of failure on the 95% subjective confidence fragility curve (HCLPF). To derive $A_{1\%}$, conservative deterministic failure margin (CDFM) method is utilized, incorporating recommended conservative values to generate seismic fragility curve.

The CDFM method incorporates specific levels of conservatism in material properties and analysis methods to achieve targeted confidence levels in terms of exceedance and non-exceedance probabilities. For non-ductile failure modes, seismic capacity is estimated using material properties corresponding to a

95% exceedance probability, and static strength equations developed for an 84% exceedance probability are employed to achieve a 99% exceedance probability capacity. Meanwhile, structural responses, conducted for mean earthquake and structural parameters, need to be scaled by a factor to estimate an 84% NEP in accordance with EPRI (2018). In this particular case, where anchor bolt failure governs for HA-2s, the discussion is limited to the calculation of bolt capacity using the CDFM approach. To estimate $A_{1\%}$, the tension and shear capacities of each anchor bolt (P_{CDFM} and V_{CDFM} , respectively) were determined through Equations 8 and 9, utilizing the code-specified ultimate material strength, σ_u , corresponding to the 95th percentile.

$$P_{CDFM} = \phi_t \sigma_u A_{eff} \quad (8)$$

$$V_{CDFM} = \phi_s 0.60 \sigma_u A_{eff} \quad (9)$$

where ϕ_t and ϕ_s are strength reduction factors for tension and shear strengths which equal to 0.75 and 0.65 for the targeted conservatism levels, respectively. For a bolt with σ_u of 60 ksi and A_{eff} of 2.50 in.², P_{CDFM} and V_{CDFM} were calculated as 112.50 kips and 58.50 kips, respectively. Also, 84% NEP demand parameters were calculated by scaling the bolt tension and shear demands, which are estimated for median seismic hazard, by 1.12, as recommended in Table I-3 in EPRI (2018). Furthermore, 5% NEP permissible tank uplift height was estimated as 1% of anchor embedment length which is equal to 0.29 in. Using these values, the overturning moment capacity was computed and $A_{1\%}$ was calculated as 0.77 g. The generic logarithmic standard deviations for active components mounted at high elevation in structures are recommended in EPRI (2018) as $\beta_R = 0.24$ and $\beta_U = 0.38$. The median ground acceleration capacity using CDFM method, A_{mCDFM} , was calculated using Equation 10 as 2.14 g, and hybrid fragility curve was generated for the equipment as illustrated in Figure 3b.

$$A_{mCDFM} = A_{1\%} e^{(1.65(\beta_R + \beta_U))} \quad (10)$$

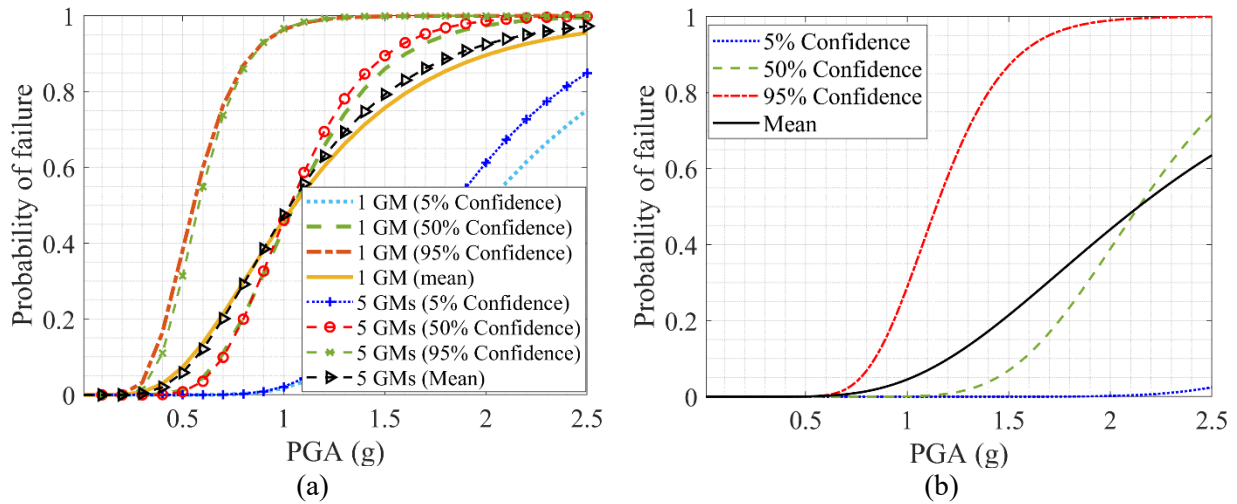


Figure 3: Fragility curves for HA-2 obtained from: (a) SOV approach, and (b) hybrid approach

Experience-Based Approach

Experience-based, also referred to as generic, seismic fragilities are provided for quick initial seismic evaluations in the absence of plant-specific fragilities. These fragilities have been developed based on seismic experience, engineering judgment from past seismic events, shake table tests, and plant seismic examination reports (NRC, 1985; NRC, 2002). The latest risk assessment handbook (NRC, 2008) provides essential fragility parameters (A_m , β_R , and β_U) for various components used in an NPP. For this study,

the seismic fragility curves at different confidence levels were generated for HA-2 using the parameters recommended for accumulators in NRC (2008) ($A_m = 2.5$ g $\beta_R = 0.30$ and $\beta_U = 0.35$), as shown in Figure 4a. The mean seismic fragility of HA-2 obtained from four different approaches are compared in Figure 4b.

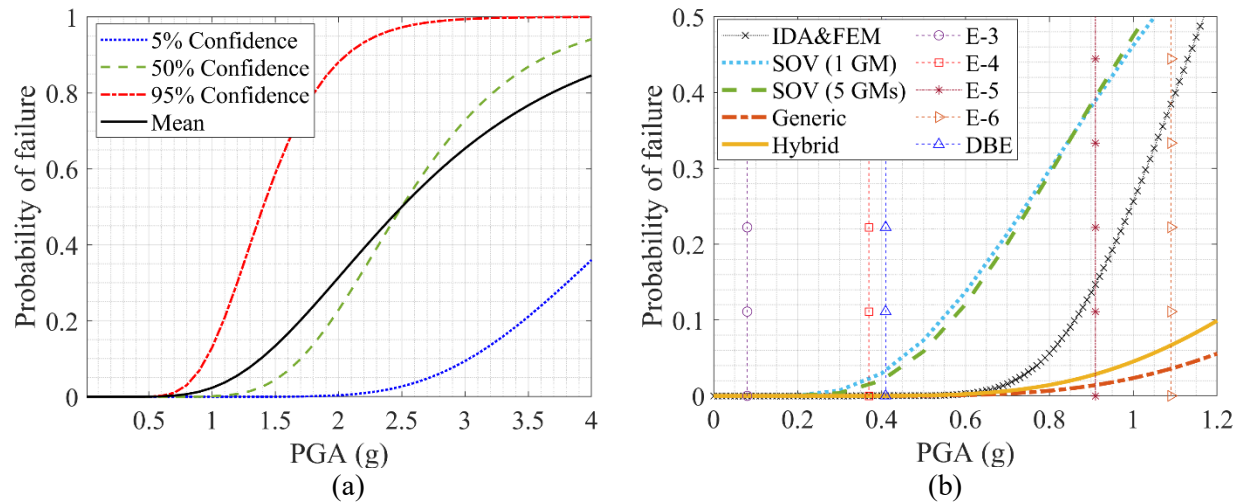


Figure 4: (a) Generic fragility and (b) comparison of different fragility approaches employed for HA-2

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

This paper aims to assess how different fragility approaches affect the calculation of likelihood of seismic-initiated failure for HA-2. For this purpose, mean seismic fragility of HA-2 was calculated using four different methods, and their impact was compared (Figure 4). The primary findings of this study are as follows:

- Employing 5 GMs reduces the composite logarithmic standard deviation from 0.50 (calculated for the case with only 1 GM) to 0.47 for the HA-2 analyzed in this research.
- Hybrid method results in a lower probability of failure compared to the advanced methods (IDA with FEM) and SOV method across the entire seismic range. This can be associated with the deterministic approaches, such as the CDFM method and biased variabilities, used in estimating the parameters of the fragility curve.
- The fragility curve developed using SOV method provides a higher probability of failure than the one obtained through the state-of-the-art methods (IDA with FEM) at any given PGA. This can be attributed to the simplified approach's assumption that the vibrational response is predominantly influenced by the fundamental mode of the equipment.
- The experience-based (generic) fragility approach yields the lowest probability of failure for HA-2 in comparison to other methods covered in this study. Relying on these approaches in the early stages of NPP design may be misleading.
- The state-of-the-art methods (IDA with FEM) are capable of more accurately calculating median seismic capacity by accounting for all vibrational modes and eliminating the uncertainty sources associated with the approximation of simplified approaches used in SOV-based fragility in this study.

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