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Seismic Performance Assessment for Safety-Related Nuclear Structures

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1 ABSTRACT

Seismic performance assessments of a sample conventional and base-isolated nuclear power plant (NPP) are performed using a new procedure that modifies the current risk assessment procedures for NPPs using tools developed in part for performance-based earthquake engineering of buildings. The new procedure uses a) response-based fragility curves to represent the capacity of structural and nonstructural components of NPPs, b) nonlinear response-history analysis to characterize the demands on those components, and c) Monte Carlo simulations to determine the damage state of the components. The use of response- rather than ground-motion-based fragility curves enables the curves to be independent of seismic hazard and closely related to component capacity. The use of Monte Carlo procedure enables the correlation in the responses of components to be directly included in the risk assessment. The new performance assessment procedure is used to evaluate the mean annual probability of unacceptable performance of the sample NPP reactor buildings. The seismic performance assessment confirms the utility of seismic isolation at reducing seismic risk in NPPs.

2 INTRODUCTION

In 1991, the United States Nuclear Regulatory Commission (NRC) issued Supplement 4 to Generic Letter No. 88-20 (USNRC 1991) requiring nuclear power plant (NPP) utilities to perform an Individual Plant Examination of External Events (IPEEE) and also issued NUREG-1407 (Chen et al. 1991) to help guide the IPEEE. For an Individual Plant Examination (IPE) of seismic events, NUREG-1407 identified Seismic Margin Assessment (SMA) and Seismic Probabilistic Risk Assessment (SPRA) as acceptable methodologies for the examination of earthquake risk.

SMA seeks to identify critical components and systems in a NPP and determine the High-Confidence-Low-Probability-of-Failure (HCLPF) capacity of each critical NPP component and plant damage state, all in terms of ground-motion intensity. The HCLPF capacity of a NPP or its component represents the value associated with a 95% confidence of a 5% probability of failure. SMA procedures can be found in Reed et al. (1991). SMA cannot be used to compute the seismic risk (annual frequency of unacceptable performance) of a NPP. The annual frequency of unacceptable performance, such as core melt and release of radiation, can be computed using SPRA, which involves integration of plant fragility data and seismic hazard curves over a wide range of ground-shaking intensity and requires a full consideration of uncertainty in seismic hazard, structural response and properties and capacities of NPP components. The focus of this paper is SPRA.

NUREG/CR-2300 (USNRC 1983) provides general guidance for performing a SPRA. The guideline describes two SPRA methods: 1) Zion method and 2) the Seismic Safety Margin (SSM) method. The Zion method was developed and applied first in the Oyster Creek probabilistic risk assessment and later improved and applied in 1981 to estimate seismic risk for the Zion Plant (Pickard, Lowe, and Garrick, Inc., et al. 1981). The SSM method was developed in an NRC-funded project at the Lawrence Livermore National Laboratory (Smith et al. 1981) but only a simplified version of the method has been applied to NPP projects to the knowledge of the authors (USNRC 1990).

The computation of risk in the Zion and SSM methods are both based on the total probability theorem, which was also used by Cornell to develop probabilistic seismic hazard analysis (Cornell 1968). An important difference between the Zion and SSM methods is the parameter used to characterize fragility curves. In the Zion method, the component fragility curves are defined in terms of ground-motion parameters (generally peak ground acceleration), although the failure of a component has a much improved correlation to structural response parameters, such as floor spectral acceleration and story drift. In the SSM method, the fragility curves are defined in terms of response parameters and the structural responses are computed using response-history analysis, where the ground motions are scaled based on peak ground accelerations.

Procedures for seismic performance assessment of buildings have been developed in the ATC-58 project and proposed in the 50% draft *Guidelines for Seismic Performance Assessment of Buildings* (ATC 2009) (termed the *draft ATC-58 Guidelines* hereafter). These procedures use a) fragility curves defined using structural response parameters, b) response-history analysis with state-of-the-art ground motion scaling procedures and c) Monte Carlo simulations to determine the damage state of structural components. The methodology in the *draft ATC-58 Guidelines* cannot be used directly to compute the seismic risk associated with a NPP because it cannot accommodate event trees and fault trees that are required to determine accident sequences that lead to unacceptable performance. However, the methodology provides the opportunity to develop an alternative procedure to compute seismic risk of NPPs.

This paper summarizes a new procedure (Huang et al. 2008, 2009a, 2009b) for seismic performance assessment of NPPs based on the existing NPP SPRA methods and the methodology presented in the *draft ATC-58 Guidelines*. The approaches used in the new procedure to scale ground motions and to determine the probability of unacceptable performance at a given ground-motion intensity level are fundamentally different from those used to date for assessment of nuclear structures. This paper introduces the new procedure by presenting a seismic performance assessment of a sample NPP reactor building of conventional and base-isolated construction. The impact of the implementation of base isolation on the seismic performance of the sample NPP (Huang et al. 2008) is identified.

3 SAMPLE NPP REACTOR BUILDINGS

Figure 1 shows a cutaway view of a NPP reactor building of conventional construction. For the purpose of the risk assessment described herein, the NPP is sited in the Eastern United States. A lumped-mass stick model of this reactor building was developed in the computer code SAP2000 Nonlinear (CSI 2002) for the purpose of response-history analysis. The model, shown in Figure 2a, is composed of two sticks: one representing the containment structure and the other representing the internal structure. The two sticks are structurally independent and are connected only at the base. The mechanical properties of the frame elements that compose each stick were back-calculated from a 3D model of the reactor building. Bilinear shear hinges with 3% post-yield stiffness were assigned to all frame elements in the internal-structure stick only since the containment vessel was designed for large internal pressures resulting from a postulated accident and load combinations including seismic effects did not control its design. The total height of the containment structure is 59.5 meters and its first mode period is approximately 0.2 second. The thickness of the post-tensioned concrete cylindrical wall of the containment structure is about 1 meter. The height of the internal structure is 39 meters. The first mode period of the internal structure in both horizontal directions is approximately 0.14 second. The total weight (W) of the NPP reactor building is approximately 75,000 tons.

Three numerical models of base-isolated reactor buildings were developed in SAP2000 Nonlinear to study the impact of seismic isolation on demands on secondary systems in the sample NPP reactor building. The three base-isolated models include representations of Friction Pendulum™ (FP) bearings, lead-rubber (LR) bearings and low damping rubber (LDR) bearings, respectively. Only the results for the conventional NPP and the base-isolated NPP using LR bearings are presented in this paper. The LR bearings were modeled using bilinear plasticity elements. Figure 3 shows the key variables defining the bilinear hysteresis loop. The period associated with the second-slope stiffness was assigned a value of 2 seconds.

The seismic performance assessments performed in this study focus on the secondary systems in the sample NPP reactor building because the costs associated with analysis, design, construction, testing and regulatory approval of secondary systems can dominate the cost of NPPs (Huang et al. 2007). Figure 1 illustrates the distribution of several important secondary systems in the sample reactor building, such as the reactor, steam generator and emergency coolant injection (ECI) tank. These secondary systems are attached to the internal structure and supported at elevations of 7 (Node 201), 18 (Nodes 1006 and 1009) and 39 m (Nodes 215 and

216). Figure 2b identifies the node numbers assigned to the internal structure of the sample NPP. In this study, the floor response spectral demands at these three elevations are computed using response-history analysis for seismic performance assessments.

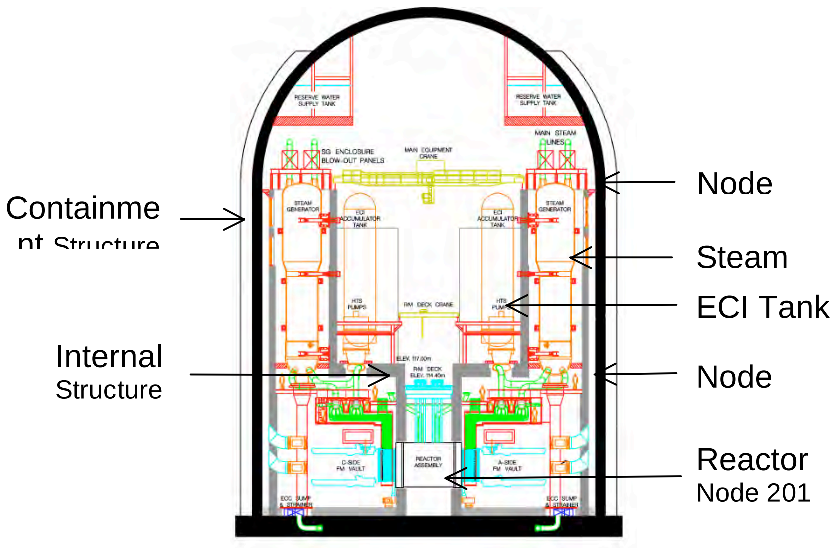
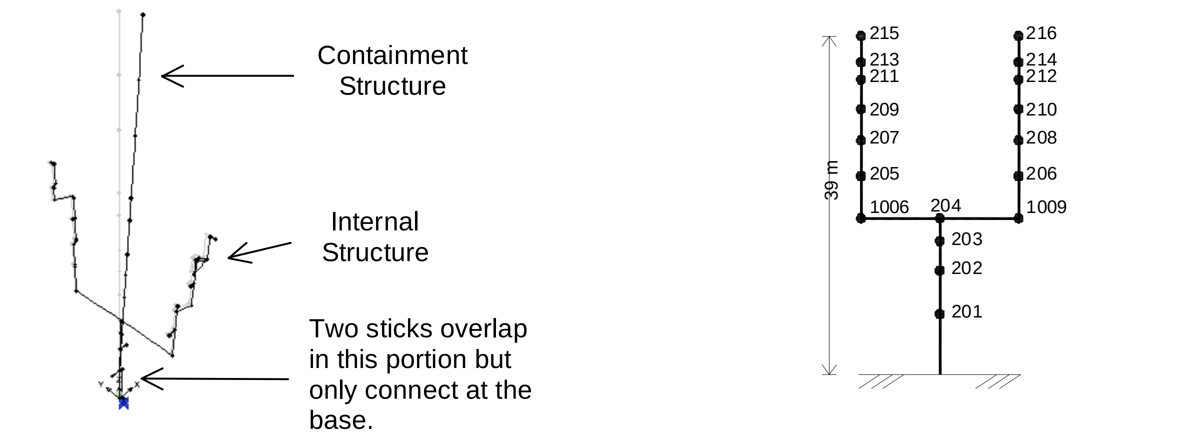


Figure 1. Cutaway view of the sample NPP reactor building



a. First mode shape of the stick model in SAP2000

b. Stick model for the internal structure

Figure 2. Stick model for the sample NPP reactor building

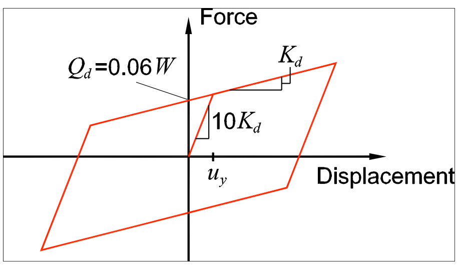


Figure 3. Assumed properties of the LR bearings

4 SEISMIC PERFORMANCE ASSESSMENT

4.1 Overview of the new procedure

The new procedure includes five steps, as presented in Figure 4. Step 1 involves the characterization of the seismic hazard using seismic hazard curves. Step 2 of the procedure requires the user to develop fragility curves for the structural and nonstructural components of the NPP, as well as the possible accident sequences for unacceptable performance, such as core melt and radiation release. Step 3 involves response-history analysis of the NPP subjected to the seismic hazard of Step 1 to estimate the accelerations, forces, displacements and deformations that serve as demands on the NPP's components and contents. Damage of the structural and nonstructural components is assessed in Step 4 using the demands computed in Step 3 and fragility curves developed in Step 2. Step 5 involves the computation of seismic risk using the results of Step 4 and the accident sequences developed in Step 2. The rest of Section 4 presents each step of the performance/risk assessment of the sample NPP. Only the procedures and results of the assessment are reported herein.

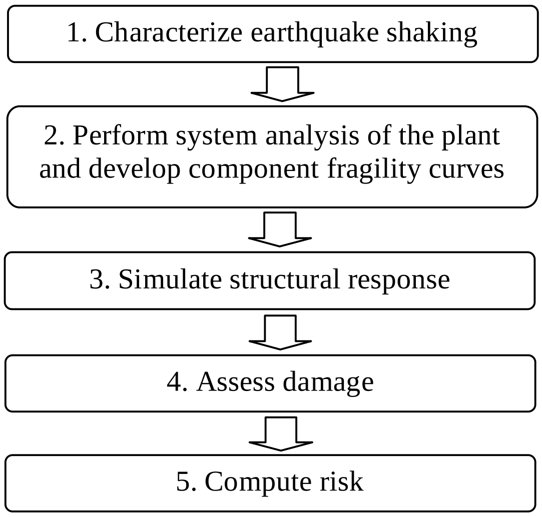
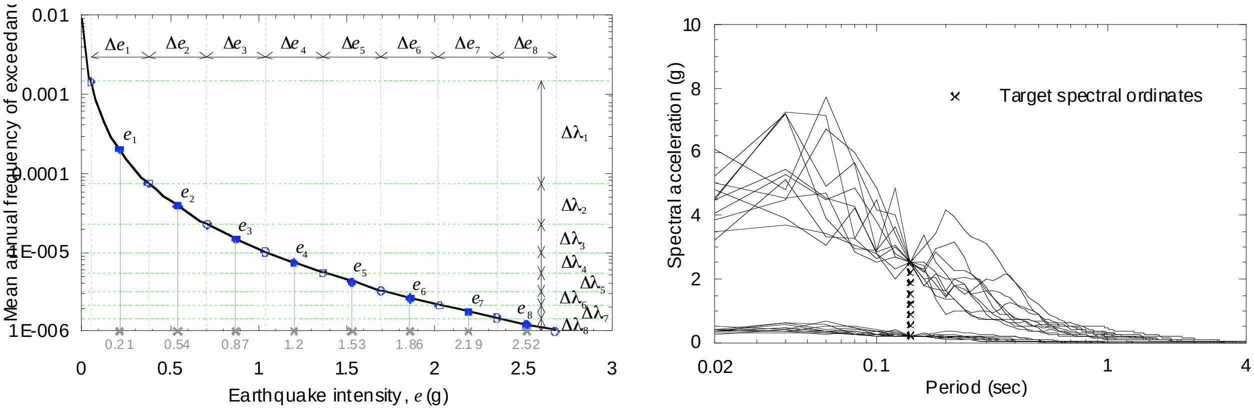


Figure 4. The new procedure for seismic performance assessment of NPPs

4.2 Step 1: Characterize earthquake shaking

A seismic performance assessment is performed as a series of intensity-based assessments. Each intensity-based assessment is associated with a target spectral intensity (e_i) at the fundamental period of the NPP and an annual frequency ($\Delta\lambda_i$) for the target spectral intensity. Both e_i and $\Delta\lambda_i$ are determined from a seismic hazard curve. Figure 5a presents the 0.14-second mean seismic hazard curve for the sample NPP site used in the assessment of the conventional sample NPP. The range of spectral acceleration between 0.05 and 2.68 g was split into 8 equal intervals. The range of spectral acceleration was selected to capture all significant risk to the sample NPP. The parameter e_i for each intensity-based assessment was determined by the midpoint spectral acceleration in each interval; $\Delta\lambda_i$ was determined by the interval of annual frequency of earthquake intensity in the range of Δe_i. The values of e_i and $\Delta\lambda_i$ are presented in Figure 5a and the second column of Table 1, respectively.



a. Computation of target spectral ordinates

b. Response spectra of the ground motions in Bins TC1 and TC8

Figure 5. Scaling ground motions for the performance assessment of the conventional sample NPP

Table 1. Computation of annual frequency of unacceptable performance of the conventional and base-isolated NPPs

Conventional NPP				Base-isolated NPP			
G.M. bin	$\Delta\lambda_i$	P_i	$\Delta\lambda_i \times P_i$	**G.M. bin**	$\Delta\lambda_i$	P_i	$\Delta\lambda_i \times P_i$
TC1	1.35E-03	0	0	TI1	1.35E-03	0	0
TC2	5.18E-05	0.16	8.29E-06	TI2	4.57E-05	0	0
TC3	1.24E-05	0.71	8.80E-06	TI3	1.07E-05	0	0
TC4	4.63E-06	0.96	4.44E-06	TI4	3.85E-06	0	0
TC5	2.23E-06	0.99	2.21E-06	TI5	1.87E-06	0	0
TC6	1.08E-06	1	1.08E-06	TI6	1.03E-06	0	0
TC7	6.90E-07	1	6.90E-07	TI7	6.09E-07	2.00E-05	1.22E-11
TC8	4.59E-07	1	4.59E-07	TI8	4.05E-07	7.50E-05	3.03E-11
Annual frequency of unacceptable performance			2.60E-05				4.25E-11

Another task of step 1 is to select and scale ground motions used in response-history analysis for each intensity assessment. Due to the lack of earthquake histories from large magnitude events in the Central and Eastern United Sates (CEUS), the computer code "Strong Ground Motion Simulation (SGMS)" (Halldorsson 2004) was used to generate CEUS-type ground motions appropriate for the sample NPP site. Eleven SGMS-generated ground motions were amplitude scaled to each e_i identified in Figure 5a at a period of 0.14 second. The resultant eight bins of ground motions were denoted Bins TC1 through TC8. The response spectra of the ground motions in Bins TC1 and TC8 are presented in Figure 5b. The analysis presented in this subsection was repeated for the base-isolated NPP using a 2-second hazard curve for the sample NPP site. The resultant eight bins of ground motions were denoted Bins TI1 through TI8. More information for the hazard curves used and ground motions developed in Step 1 can be found in Huang et al. (2008).

4.3 Step 2: Perform system analysis of the plant and develop component fragility curves

The purpose of system analysis of a NPP is to determine possible accident sequences leading to the unacceptable performance. A robust way for this task is to use event trees and fault trees (see Reed and Kennedy 1994 for more information). The unacceptable performance of the sample NPP evaluated in this study was defined as the failure of any of the secondary systems at Nodes 201, 1009 and 216. This unacceptable performance can be defined by the fault tree of Figure 6 where the "OR" gate defines the occurrence of the event right above the gate (the unacceptable performance) as the occurrence of one or more failure events immediately below the gate (the failure of secondary systems).

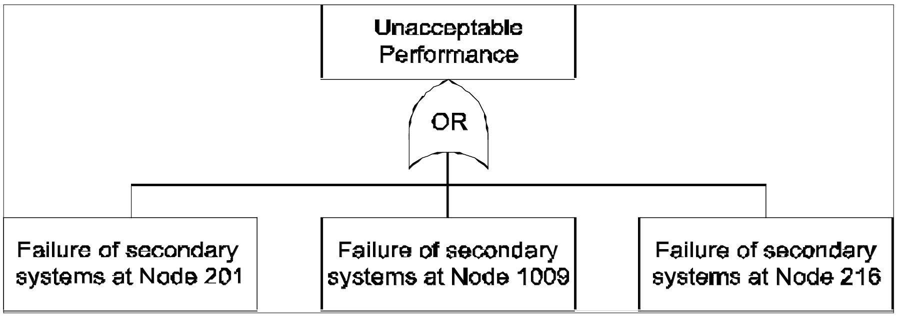


Figure 6. A fault tree for the unacceptable performance of the sample NPP

Fragility data are required in this performance-assessment procedure for all basic failure events at the lowest level of the fault tree. In this study, the demand parameter used to develop fragility curves for the secondary systems in the sample NPP is Average Floor Spectral Acceleration over a frequency range from 5 to 33 Hz, termed AFSA herein, since the seismic demands on secondary systems in NPPs are characterized typically using a floor response spectrum and the frequencies of most secondary systems are in the range of 5 to 33 Hz. (For a project specific application, any demand parameter could be used but fragility curves would have to constructed for that parameter.) Figure 7 presents the three fragility curves used in this study. The median AFSA values are 2.26, 3.15 and 7.02 g for the curves for Nodes 201, 1009 and 216, respectively, and the logarithmic standard deviation for the three curves is 0.43. More information for the development of the fragility curves of Figure 7 can be found in Huang et al. (2008, 2009b).

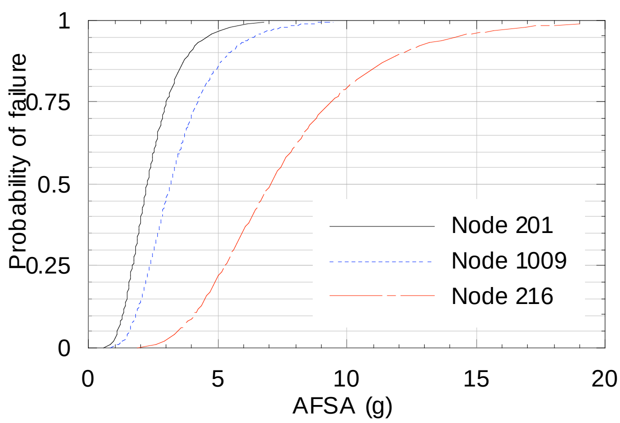


Figure 7. Mean fragility curves for the secondary systems at Nodes 201, 1009 and 216

4.4 Step 3: Simulate Structural Response

Unidirectional nonlinear response-history analyses were performed for the conventional NPP subjected to the Bins TC1 through TC8 ground motions and for the base-isolated NPP subjected to the Bins TI1 through TI8 ground motions in the X and Y directions. Sample results for the conventional NPP subjected to the Bin TC8 ground motions and the base-isolated NPP subjected to the Bin TI8 ground motions are presented in Figure 8 using black dots. The X and Y axes of Figure 8 are the values of AFSA at Nodes 201 and 216, respectively. The demands on the secondary systems in the base-isolated NPP are significantly smaller than those in the conventional NPP.

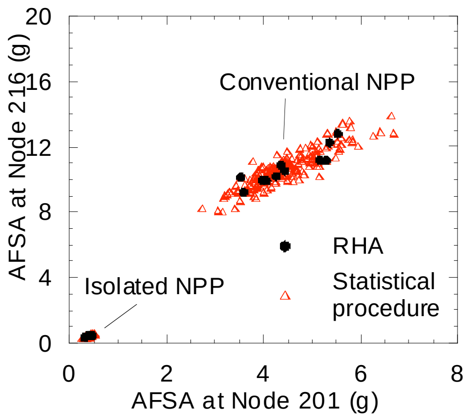


Figure 8. AFSA at Nodes 201 and 216 for 1) the conventional NPP to the Bin TC8 ground motions and 2) the isolated NPP to the Bin TI8 ground motions in the X direction

Table 2 presents a demand-parameter matrix using the results of response-history analysis for the conventional NPP subjected to the Bin TC8 ground motions. In the procedure introduced herein, all demand parameters used in the fragility curves developed in Step 3 are required to be included in the demand-parameter matrix. In the case of Table 2, each row vector of the matrix includes six values for the AFSA values at Nodes 201, 1009 and 216 in the X and Y directions from the analysis using a ground motion in Bin TC8. The number of row vectors in the demand-parameter matrix, which is 11 in this case, is determined by the number of ground motions in a bin. For each NPP model and ground-motion bin, a demand-parameter matrix, similar to that of Table 2, was generated and used in the performance assessment.

Table 2. Demand-parameter matrix for the conventional NPP subjected to the Bin TC8 ground motions

GM No.	AFSA in the X direction (g)			AFSA in the Y direction (g)		
	Node 201	Node 1009	Node 216	Node 201	Node 1009	Node 216
1	5.36	6.14	12.26	5.06	6.27	14.49
2	4.06	5.29	9.93	4.12	5.99	12.93
3	4.36	6.18	10.86	3.97	5.25	13.22
4	5.53	6.96	12.78	5.19	6.99	14.55
5	3.98	5.40	9.94	3.30	4.68	10.60
6	4.44	6.37	10.51	3.78	5.41	12.89
7	3.59	4.71	9.19	4.10	4.83	10.72
8	4.26	5.36	10.19	4.23	5.73	12.12
9	5.17	6.22	11.19	5.48	7.58	14.28
10	3.54	5.02	10.13	3.52	4.80	12.13
11	5.29	5.88	11.14	5.55	6.81	14.07

4.5 Step 4: Assess Damage of NPP Components

A Monte Carlo type procedure was used to assess the failure events at the lowest level of the fault tree of Figure 6, i.e., the failure of secondary systems at Nodes 201, 1009 and 216. For example, the value of AFSA at Node 201 in the X direction for the conventional NPP subjected to GM1 in Bin TC8 is 5.36 g, as shown in the demand-parameter matrix of Table 2. Per the fragility curve of Figure 7 for Node 201, the probability of failure is 0.98 at an AFSA of 5.36 g. A random number generator that generates random numbers uniformly distributed between 0 and 1 was used to select the damage state for the secondary system at Node 201. If the realization generated by the random generator is smaller or equal to 0.98, the secondary system is considered to have failed; and if the realization is greater than 0.98, the secondary system is considered to have passed. This procedure was repeated for all other five AFSA values in the row vector for GM1 to determine the success or failure of each basic event at the lowest level of the fault tree of Figure 6. According to the accident sequence defined in the fault tree, if any of the basic failure events occur, unacceptable performance is assumed for the sample NPP subjected to GM1. This analysis was repeated for all other row vectors in the demand-parameter matrix.

4.6 Step 5: Compute the Risk

The use of Monte Carlo procedures requires a large set of simulations so that the probability and annual frequency of unacceptable performance can be estimated with high confidence. The large set of simulations can be generated by two procedures, 1) directly by a large number of response-history analyses, or 2) indirectly by statistical manipulation of the results of a smaller number of analyses. The *draft ATC-58 Guidelines* presents one procedure, which was also used in this study, for generating a large number of simulations through statistical manipulation of a relatively small number of structural analyses (ATC 2009; Yang et al. 2006). The Yang et al. procedure can be used to increase the number of row vectors in a demand-parameter matrix without performing additional response-history analysis. Figure 8 presents the distribution of AFSA obtained using response-history analysis (11 row vectors) and that generated per the Yang et al. procedure (200 row vectors, for presentation only). The results show that the AFSA values generated per the Yang et al. procedure preserve the magnitude and correlation in AFSA obtained using response-history analysis.

The Yang et al. procedure was used to increase the number of row vectors in the demand-parameter matrix for each of the 8 intensity-based assessments from 11 to 2000 and 200,000 for the conventional and isolated sample NPPs, respectively. Step 4 was repeated for each new demand-parameter matrix and the probability of unacceptable performance (P_i) associated with the new demand-parameter matrix was computed as the ratio of the number of row vectors with unacceptable performance to the total number of row vectors in the matrix. The values of P_i are presented in the third and seventh columns of Table 1 for the conventional and isolated sample NPPs. The mean annual frequency of unacceptable performance for each model was computed by summing the eight products of P_i and $\Delta\lambda_i$. Per Table 1, the mean annual frequency of unacceptable performance of the base-isolated NPP (4.25E-11) is nearly six orders of magnitude smaller than that of the conventional NPP (2.60E-05).

5 CLOSING REMARKS

A new procedure is summarized for probabilistic seismic performance assessment of NPPs. The procedure is based on the current probabilistic risk assessment procedures in the nuclear industry and new tools developed for the performance-based earthquake engineering of buildings.

The new procedure involves five steps. Step 1 characterizes the seismic hazard, determines the target spectral acceleration at different intensity levels, and selects and scale pairs or sets of ground motions. Step 2 involves plant system analysis to determine possible accident sequences that would contribute to unacceptable performance, such as core melt and radiation release, and develops component fragility curves using structural response parameters. Nonlinear response-history analysis of NPP models is performed in Step 3 using the ground motions of Step 2. Step 4 uses Monte-Carlo-based procedures to generate a large number of response data that are statistically consistent with the response data of Step 3 and assesses the possible distribution of damage to structural and nonstructural components of the NPP for each set of simulations. In Step 5, the probability of unacceptable performance at each shaking intensity is computed using the corresponding damage distribution from Step 4 and the accident sequences of Step 2 to establish the annual frequency of unacceptable performance of a NPP.

The seismic performance of a sample conventional and base-isolated NPP reactor building was evaluated using the new procedure. The mean annual frequencies of unacceptable performance of the conventional and base-isolated sample NPPs are 2.60E-05 and 4.25E-11, respectively.

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