18th International Conference on Structural Mechanics in Reactor Technology (SMiRT 18)
Beijing, China, August 7-12, 2005
SMiRT18-K11-7

SEISMIC PROTECTION OF SECONDARY SYSTEMS IN NUCLEAR POWER PLANT FACILITIES

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ABSTRACT

Numerical models of a sample nuclear power plant (NPP) reactor building, both conventionally constructed and equipped with seismic protective systems are analyzed for both Safe Shutdown and Beyond-Design-Basis earthquake shaking at two coastal sites in the United States. Seismic demands on secondary systems are established for the conventional and seismically isolated NPPs. The reductions in secondary-system acceleration and deformation demands afforded by the isolation systems are identified. The impact of isolation system choice on the response of the key secondary systems is presented.

Keywords: Nuclear power plant, secondary systems, seismic base isolation, viscous dampers.

1. INTRODUCTION

The United States Department of Energy (DOE) has identified a number of generic issues that could impact the viability of new nuclear power plant (NPP) construction in the United States, including economic competitiveness, nuclear safety, and management of spent nuclear fuel (SNF). Seismic protective systems, herein assumed to include seismic isolation and damping devices, can address each of these issues, enabling direct reductions in overnight capital cost and standardization of NPP and SNF facility designs and simplified design and regulatory review; facilitating design certification and the granting of early site permits and construction and operating licenses; and enhancing NPP and SNF safety at lower capital cost.

The primary focus of this paper is the characterization of seismic demands on secondary systems in NPP construction because the costs associated with analysis, design, construction, testing and regulatory approval of secondary systems can dominate the cost of NPPs. Substantial reductions in demands on secondary systems such as steam generators and piping systems will facilitate greater use of commercial-off-the-shelf equipment, simplify system and equipment qualification and design and regulatory review, and enhance NPP safety.

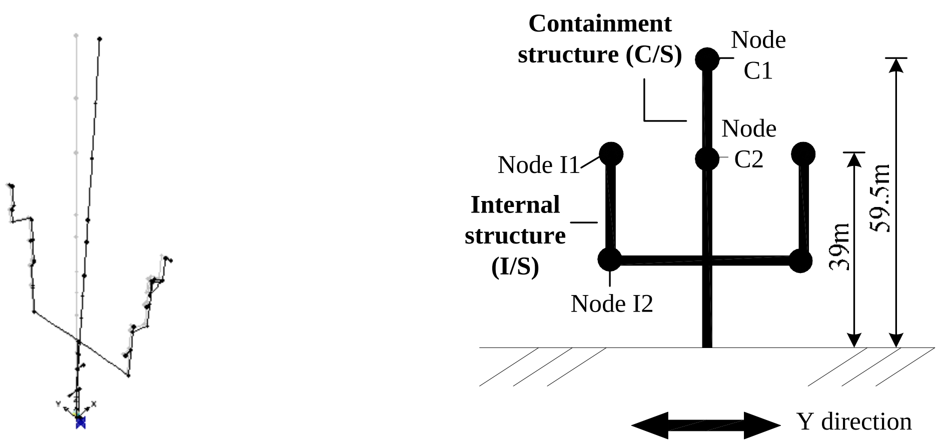
U.S. Nuclear Regulatory Commission (USNRC) Regulatory Guide RG 1.165 (USNRC, 1997) specifies that the Safe Shutdown Earthquake (SSE) for the seismic design of safety-related nuclear structures be based on a 5% damped median response spectrum with a return period of 100,000 years (or a median annual probability of exceedance of 1×10^{-5}). Chapter 1 of 10CFR (USNRC, 2005) writes that safety-related structures, secondary systems and components remain functional (undamaged) in the event of SSE shaking. For many NPP sites in the U.S., earthquake shaking associated with the SSE will result in high seismic acceleration and deformation demands in the stiff structural systems and extremely high demands on the safety-related secondary systems. Seismic isolation can substantially mitigate these high demands on primary structural components and secondary mechanical, electrical and piping systems by reducing the natural frequency of the NPP structure (by up to a factor of 20) and thus substantially separating the dominant frequencies of the secondary systems from the key natural frequency(s) of the primary system.

Seismic isolation and damping systems are widely used to protect mission-critical infrastructure. More than 3,000 buildings, bridges, and other types of structures have been isolated to date (Kelly, 2004). Seismic isolation has been incorporated into six reactor units, four in France and two in South Africa. In 1997, the Central Research Institute of Electric Power Industry in Japan issued isolation guidelines for applications to Fast Breeder reactors and Light Water reactors. In the United States, seismic isolation design procedures have also been included in the design code for safety-related nuclear structures (ASCE, 2000).

The potential benefits of applying seismic protective systems to the next generation of NPPs and SNF buildings prompted the study described in the remainder of this paper. Response-history analysis of numerical models of conventional and isolated reactor buildings was used to assess demands on structural and secondary systems. Herein, attention is focused on horizontal shaking only. Additional results will be available in Huang et al. (2006).

2. DYNAMIC MODELS OF THE FIXED-BASE AND BASE-ISOLATED REACTOR BUILDINGS

A lumped-mass stick model of a conventional next generation NPP reactor building was used to benchmark results from response-history analysis. (Information on the framing of the NPP reactor building was provided by a NPP supplier.) The model, shown in Figure 1, 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 analysis of the 3-D reactor building. The reactor building was assumed to be founded on *west coast* rock. The mass of the structure and the secondary systems was lumped at discrete locations at key levels in the reactor building. The discrete masses were connected to the frame elements through rigid links to account for torsional effects. The total height of the containment structure is 59.5 meters and its first mode period is approximately 0.2 second. 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 of the NPP reactor building is approximately 75,000 tons.



a. Model 1 and first mode shape　　b. Stick model sketch

Fig. 1. Model of the NPP building for response-history analysis

Nineteen numerical models were developed in the computer code SAP2000 Nonlinear (CSI, 2002) to study the impact of seismic protective systems on the response of the NPP reactor building and its secondary systems. The numerical models include representations of high damping rubber (HDR) bearings, lead-rubber (LR) bearings and Friction Pendulum™ (FP) bearings. Table 1 lists the properties of the isolation systems used in the models. Model 1 is the model of the conventionally framed NPP reactor building. Bilinear elements were used for the LRB and FP bearings; the key properties are shown in Figure 2. To study a wide range of isolator properties, the values of Q_d and F_y were set equal to either 3 or 6 percent of the supported weight and the second-slope period (related to K_d through the supported weight) was assigned values of 2, 3 and 4 seconds. The HDR bearings were modeled as linear viscoelastic elements and the problematic issues associated with scragging and modeling were not addressed. The linear viscoelastic models could equally well, and perhaps better, model an isolation system incorporating low damping rubber bearings and linear viscous dampers. Again, to study a wide range of isolator properties, consistent with isolators used in design practice, the dynamic properties of the HDR isolators included isolated periods of 2, 3 and 4 seconds and viscous damping ratios of 10% and 20% of critical. (Note that a period of 4 seconds is problematic for HDR and LR bearings due to isolator instability and that a damping ratio of 20% can only be realized in HDR isolators with supplemental damping devices.)

3. EARTHQUAKE HISTORIES USED FOR THE RESPONSE-HISTORY ANALYSIS

To judge the utility of seismic isolation for NPP and SNF applications, response-history analysis was performed using earthquake histories generated for east and west coast sites. The two bins of 10 pairs of earthquake histories were generated by Somerville et al. (1997) for firm soil sites in Boston and Los Angeles and a 2% probability of exceedance in 50 years. The Somerville seed motions were re-scaled for the response-history analysis reported herein. Each pair of earthquake histories was scaled to minimize the sum of the squared error between the target spectral values (developed from the USGS hazard maps for a 2% probability of exceedance in 50 years) and the geometric mean of the spectral ordinates for the pair at periods of 0.3, 2 and 4 seconds, where 0.3 second is representative of the natural period of a conventionally framed NPP and 2 and 4 seconds represent two of the isolated periods considered in the study. Acceleration spectra for the resulting 20 histories in each bin are shown in Figures 3a and 3c for the east and west coast sites, respectively. The variability in the spectral ordinates by bin and period is indicative of the scatter in recorded ground motions for a given combination of earthquake magnitude and distance. Figures 3b and 3d present the target spectrum, and median, 84th percentile and 16th percentile spectral ordinates for the 20 histories for the east and west coast sites, respectively. It should be noted that the spectral ordinates of the west coast (Los Angeles) histories are substantially larger in the long-period range than those of the east coast (Boston) histories.

The 2% probability of exceedance in 50 years corresponds to the return period of 2475 years: a return period much less than that required by RG 1.165 for SSE shaking. Although this could be considered a shortcoming of the analysis reported herein, the authors felt that it was more important to utilize a robust set of earthquake histories that accounted directly for randomness in the ground motion than to use motions matched to a site-specific spectrum. Importantly, the target spectra of Figures 3b and 3d could represent SSE shaking at sites on the east and west coast, although not in Boston or Los Angeles, respectively.

Safety assessment of NPP often requires a margins assessment, or an estimate of response for Beyond-Design-Basis (BDB) earthquake shaking, to ensure that shaking in excess of that associated with the SSE spectrum will not substantially compromise the safety and integrity of the NPP structure and secondary systems. No specific margin or multiplier is mandated by the USNRC for beyond-design-basis assessment. (The multiplier used in the United Kingdom is 1.4.) Per RG 1.165, SSE shaking is characterized by a 5% damped *median* spectrum. For this study, earthquake histories for the beyond-design-basis assessment were generated by amplifying the acceleration time series for each history from the *median* level to an *84th* percentile level. The acceleration time-series magnification factor was determined using ground-motion-attenuation relationships for east and west coast sites, recognizing that the ratio of the 84th and 50th (median) spectral ordinates (Y) is equal to the standard deviation of $\mathrm{Ln}Y$. The attenuation relationships of Campbell (2001) and Abrahamson and Silva (1997) were used to compute the multipliers for the east coast and west coast sites, respectively. Values for the multiplier were calculated for periods of 0.01 second, 0.15 second, 2.0 seconds and 4.0 seconds were computed. Results are shown in Fig. 4. For the beyond-design-basis assessment presented here, a magnification factor of 1.7 was used to scale up the earthquake acceleration histories.

Table 1. Description of response-history-analysis models

Model no.	Type of bearings	Description[1]
1	None	First mode periods of the containment and internal structures, in each horizontal direction, are (0.22 sec, 0.21 sec) and (0.14 sec, 0.13 sec), respectively
2	FP-isolated	$Q_d = 0.03W$; $T_d = 2$ seconds; $u_y = 1$ mm
3	FP-isolated	$Q_d = 0.03W$; $T_d = 3$ seconds; $u_y = 1$ mm
4	FP-isolated	$Q_d = 0.03W$; $T_d = 4$ seconds; $u_y = 1$ mm
5	FP-isolated	$Q_d = 0.06W$; $T_d = 2$ seconds; $u_y = 1$ mm
6	FP-isolated	$Q_d = 0.06W$; $T_d = 3$ seconds; $u_y = 1$ mm
7	FP-isolated	$Q_d = 0.06W$; $T_d = 4$ seconds; $u_y = 1$ mm
8	LR-isolated	$F_y = 0.03W$; $T_d = 2$ seconds; $K_u = 10K_d$
9	LR-isolated	$F_y = 0.03W$; $T_d = 3$ seconds; $K_u = 10K_d$
10	LR-isolated	$F_y = 0.03W$; $T_d = 4$ seconds; $K_u = 10K_d$
11	LR-isolated	$F_y = 0.06W$; $T_d = 2$ seconds; $K_u = 10K_d$
12	LR-isolated	$F_y = 0.06W$; $T_d = 3$ seconds; $K_u = 10K_d$
13	LR-isolated	$F_y = 0.06W$; $T_d = 4$ seconds; $K_u = 10K_d$
14	HDR-isolated	$T_i = 2$ seconds; $\xi_i = 0.10$
15	HDR-isolated	$T_i = 3$ seconds; $\xi_i = 0.10$
16	HDR-isolated	$T_i = 4$ seconds; $\xi_i = 0.10$
17	HDR-isolated	$T_i = 2$ seconds; $\xi_i = 0.20$
18	HDR-isolated	$T_i = 3$ seconds; $\xi_i = 0.20$
19	HDR-isolated	$T_i = 4$ seconds; $\xi_i = 0.20$

1. See Figure 2 for definitions of Q_d, F_y, K_d, K_u; T_d is related to K_d through the supported weight; T_i is the isolated period based on a rigid superstructure; ξ_i is the damping in the HDR isolation systems.

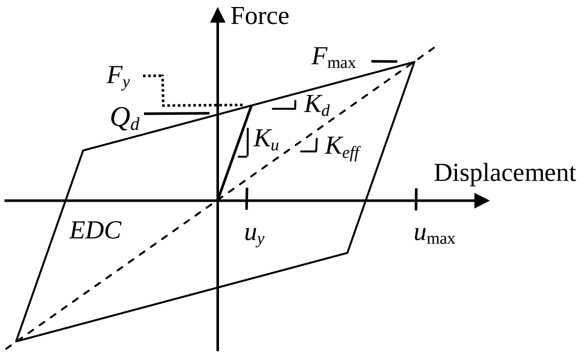


Fig. 2. Assumed properties of the LR and FP-bearings

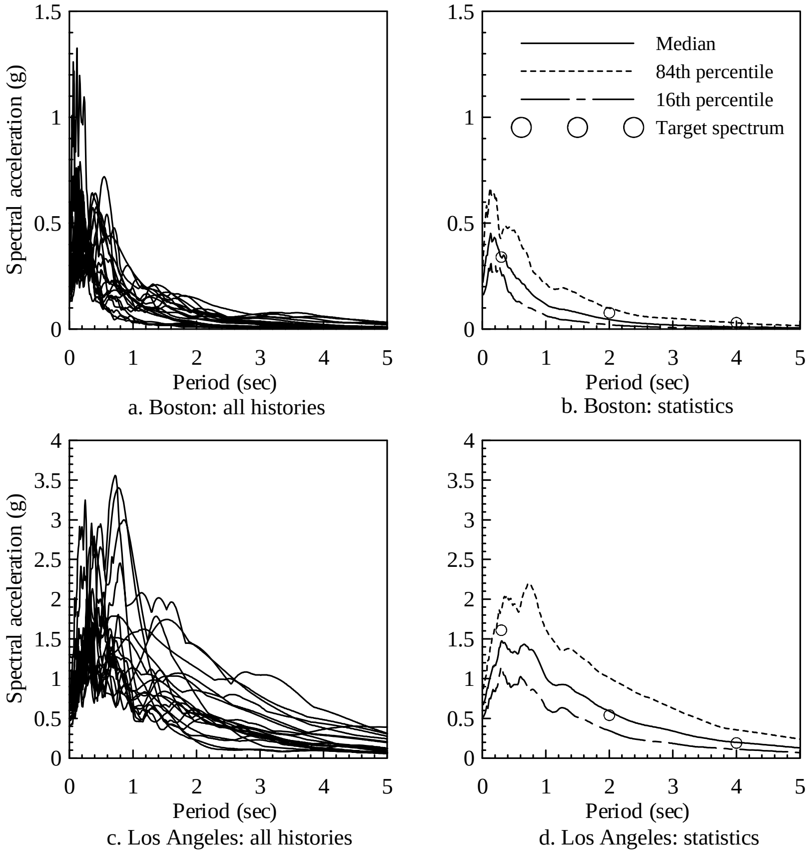
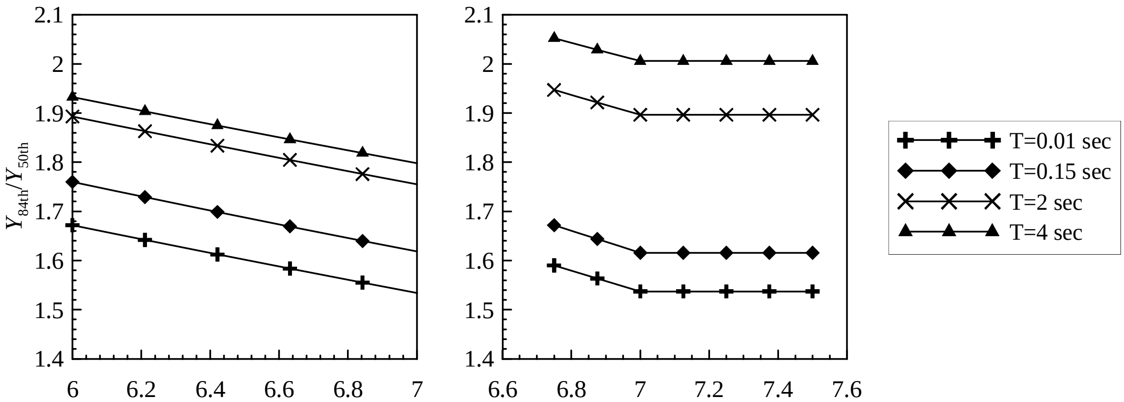


Fig. 3. Spectral accelerations of the ground motion bins for Boston and Los Angeles



a. Campbell (2001)
for Boston bin

b. Abrahamson & Silva (1997)
for Los Angeles bin

Fig. 4. Beyond-Design-Basis acceleration time-series magnification factors

4. SEISMIC DEMANDS ON SECONDARY SYSTEMS

4.1 Introduction

Response-history analysis was performed for each model and for three combinations of ground shaking and superstructure strength. Table 2 presents the combinations. Two levels of ground shaking were considered: SSE shaking and Beyond Design Basis (BDB) shaking, where BDB shaking is 170% of SSE shaking. Three levels of internal superstructure (exclusive of the isolators) strength were considered: infinite (elastic response), V_y and $1.20V_y$, where V_y in each frame element in each horizontal direction was taken as the median shear force demand of the 20 earthquake histories for SSE shaking on the conventionally framed NPP. Bilinear shear hinges with 10% post-yield stiffness were assigned to all frame elements in the internal-structure *stick*. The containment structure was assumed to remain elastic for SSE and BDB shaking. The strength of $1.20V_y$ represents a lower bound to the strength of the internal structural components. For each case, the same superstructure strength was employed for each model.

Table 2. Combinations of earthquake shaking and superstructure strength

Case	Ground motion	Yield strength of superstructure
I	SSE shaking	∞
II	BDB shaking	V_y
III	BDB shaking	$1.20V_y$

Results of the response-history analysis are presented below for the internal structure only because the secondary systems are supported primarily on this structure and not the containment structure. Unidirectional horizontal earthquake histories were applied to each model along each horizontal axis for a total of 40 simulations per model per bin. The statistical analysis of Section 4.2 below utilizes peak response data from each of the 40 simulations.

4.2 Response-History Analysis Results

Figures 5 and 6 present the maximum acceleration statistics at node I1, the top of the internal structure, as shown in Fig. 1b) and the drift ratio (relative displacement divided by height) statistics between node I1 and node I2 for all models, three cases and two bins of earthquake shaking. Accelerations at node I1 characterize peak demands at piping-system attachment points at the top of the internal structure. Node I1 and I2 drift ratios represent peak demands on deformation-sensitive components such as steam generators. Median, 16th percentile and 84th percentile

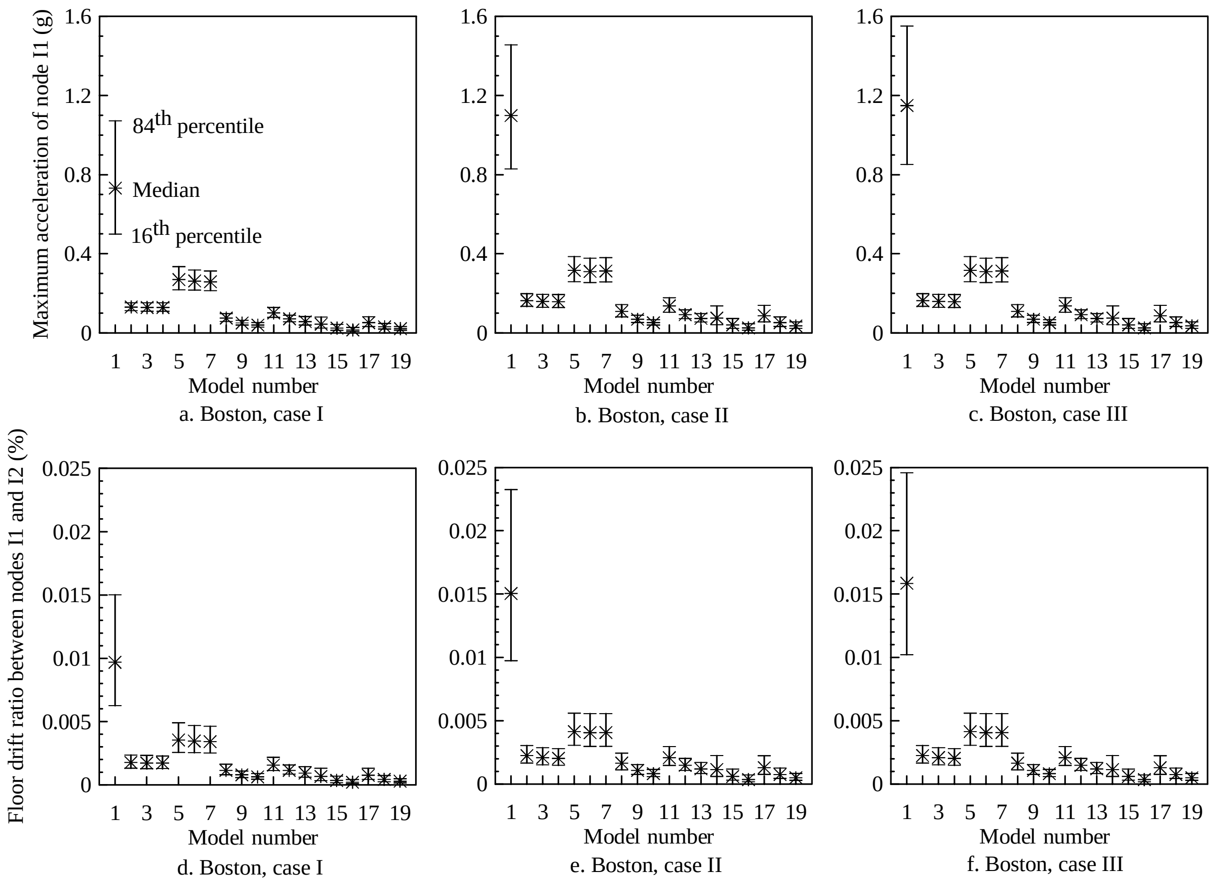


Fig. 5. Sample acceleration and drift ratio demands for the Boston earthquake histories

values are presented assuming that maximum values obtained from the response-history analysis are lognormally distributed. Table 3 lists the median response ratios of Figs. 5 and 6, calculated as the median response of the isolated NPP model divided by the median response of the conventional NPP model (Model 1).

Some general conclusions can be drawn on the basis of the data presented in Figs. 5 and 6 and Table 3. The use of seismic isolation leads to very significant reductions in acceleration and drift demands. The response reduction is greatest for the long-period isolated NPP (4 seconds), which is intuitive. Much greater percentage reductions are realized for the east coast (Boston) histories because the west coast (Los Angeles) histories have a much higher long-period content (see Figs. 3b and 3d). Importantly, the spectral accelerations of some of the Los Angeles histories are greater at a period of 2 seconds than at 0.14 second: the first mode period of internal structure.

For the Boston bin of earthquake histories, the demands on the secondary system are greater for the isolation systems with $Q_d = 0.06W$ (Models 5, 6 and 7) than for $Q_d = 0.03W$ (Models 2, 3 and 4). (The displacements across the isolation interface are smaller for Models 5, 6 and 7 than for Models 2, 3 and 4, respectively.) This result is due to larger forces developing in the $Q_d = 0.06W$ isolation systems at small displacements and the higher pre-yield stiffness of the $Q_d = 0.06W$ isolation systems (because the yield displacement was set at a constant value of 1 mm for all six FP isolation systems). Improved models of the FP bearings, accounting for the velocity dependence of the coefficient of sliding friction, will reduce the demands on the secondary systems. The trend observed for the FP bearings and the Boston ground motions does not hold for higher levels of earthquake shaking (i.e., the Los Angeles ground motions) for which comparable secondary-system demands are observed for the $Q_d = 0.06W$ and $Q_d = 0.03W$ isolation systems: see Table 3. For BDB shaking (Cases II and III), the increase in

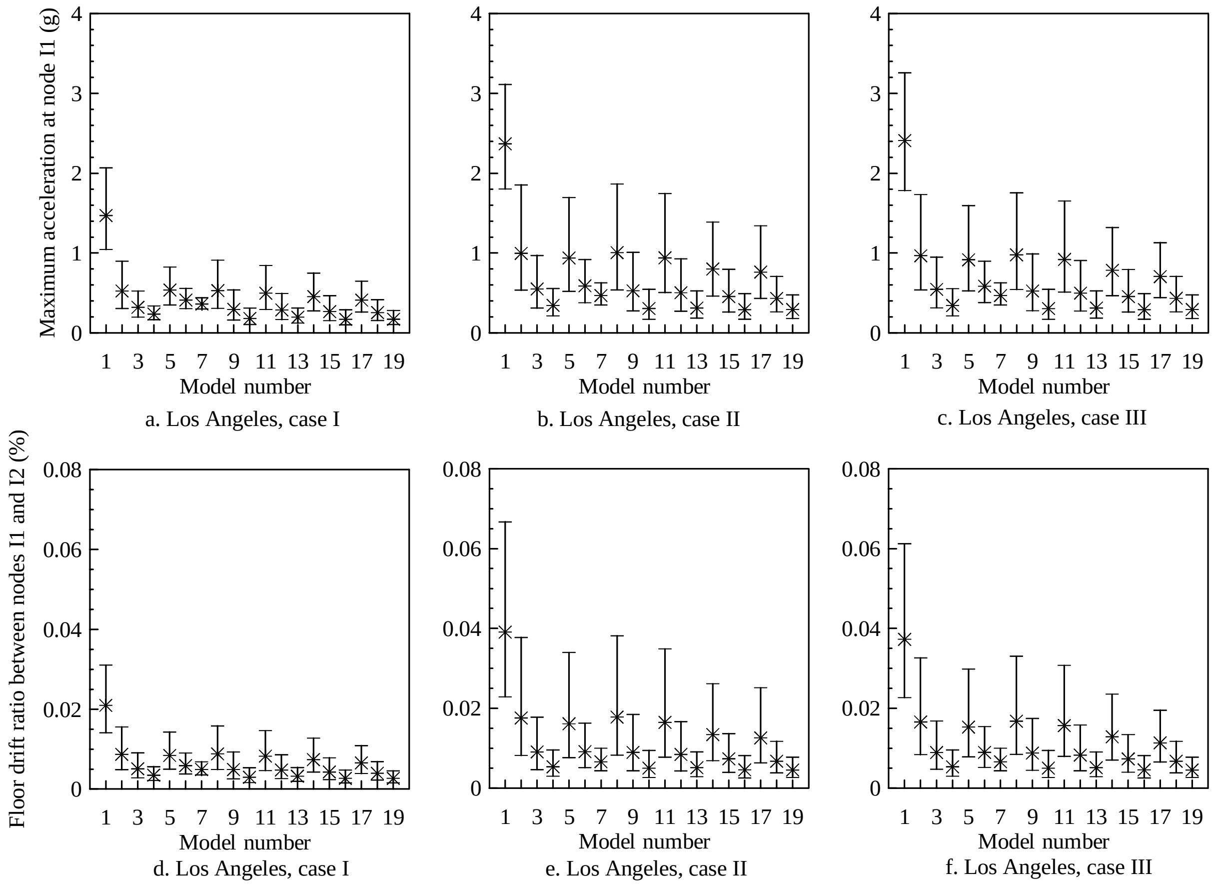


Fig. 6. Sample acceleration and drift ratio demands for the Los Angeles earthquake histories

median and 84th percentile accelerations and drift ratios is significant (of the order of 1.5) for the conventional NPP but small for the isolated NPPs, showing that an increase in the shaking intensity does not produce a similar increase in the demands on the secondary systems for the isolated NPPs. For the Boston earthquake histories, all of the isolated superstructures remained elastic for BDB earthquake shaking.

For the Los Angeles bin of BDB histories and case II yield strength (see Table 2), the superstructures of the isolated models with a period of 2 seconds yield (i.e., are damaged) for 10 of the 20 earthquake histories. (The number of instances of yielding decreases with an increase in isolation period.) Superstructure yielding for some earthquake histories produces the large differences in the 16th and 84th percentile responses of the 2-second isolated models in Fig. 6. Nonetheless, the introduction of an isolation system reduces substantially the median response of the NPP building and its secondary systems and the absolute dispersion of the responses are far smaller for the isolated NPPs than for the conventional NPP.

Floor Response Spectra (FRS) have been widely used in the nuclear industry (USNRC, 1978) to estimate demands on flexible secondary systems (Chen and Soong, 1998). In this study, absolute acceleration response histories were recorded at node I2 to develop representative FRS and to identify the difference in spectral responses for the conventional and isolated NPPs. (For the sample NPP under consideration, node I2 is located at the base of a tall steam generator.) Fig. 7 shows the median and 84th percentile floor acceleration response spectra for models 1, 3, 9 and 15 subjected to the Boston and Los Angeles SSE bins of ground motions, assuming that the peak responses are lognormally distributed. The frequency range of 5 to 33 Hz encompasses the frequencies of most NPP secondary systems. The reductions in floor spectra response resulting from the use of seismic isolators range between 3 and 15+ for the selected isolation systems and the frequency range of 5 to 33 Hz. The largest reductions in response are realized for the east coast (Boston) earthquake histories.

Table 3. Ratios of median responses in conventional and isolated NPPs

Bearing type	Model number	Boston motions						Los Angeles motions					
		Acceleration at node I1			Drift ratio between nodes I1 and I2			Acceleration at node I1			Drift ratio between nodes I1 and I2		
		Case I	Case II	Case III	Case I	Case II	Case III	Case I	Case II	Case III	Case I	Case II	Case III
FP	2	0.18	0.15	0.14	0.18	0.15	0.14	0.36	0.42	0.40	0.42	0.45	0.44
	3	0.18	0.14	0.14	0.18	0.14	0.13	0.22	0.23	0.23	0.24	0.23	0.24
	4	0.18	0.14	0.14	0.18	0.13	0.13	0.16	0.15	0.14	0.17	0.14	0.14
	5	0.37	0.29	0.28	0.37	0.27	0.26	0.37	0.40	0.38	0.40	0.41	0.41
	6	0.36	0.28	0.27	0.36	0.27	0.26	0.28	0.25	0.24	0.28	0.23	0.24
	7	0.35	0.28	0.27	0.35	0.27	0.26	0.25	0.20	0.19	0.23	0.17	0.18
LR	8	0.10	0.10	0.09	0.12	0.11	0.10	0.36	0.42	0.41	0.42	0.46	0.45
	9	0.07	0.06	0.06	0.08	0.07	0.07	0.20	0.22	0.22	0.23	0.23	0.24
	10	0.05	0.05	0.04	0.07	0.05	0.05	0.12	0.13	0.13	0.14	0.13	0.13
	11	0.14	0.12	0.12	0.16	0.14	0.13	0.34	0.40	0.38	0.39	0.42	0.42
	12	0.10	0.08	0.08	0.12	0.10	0.09	0.20	0.21	0.21	0.23	0.22	0.22
	13	0.08	0.07	0.06	0.10	0.08	0.07	0.13	0.13	0.13	0.15	0.13	0.14
HDR	14	0.06	0.07	0.07	0.07	0.08	0.07	0.31	0.34	0.33	0.35	0.34	0.35
	15	0.03	0.04	0.03	0.04	0.04	0.04	0.18	0.19	0.19	0.20	0.19	0.20
	16	0.02	0.02	0.02	0.02	0.02	0.02	0.12	0.12	0.12	0.13	0.12	0.12
	17	0.07	0.08	0.08	0.08	0.09	0.08	0.28	0.32	0.29	0.31	0.32	0.30
	18	0.04	0.05	0.04	0.04	0.05	0.05	0.17	0.18	0.18	0.19	0.17	0.18
	19	0.03	0.03	0.03	0.03	0.03	0.03	0.12	0.12	0.12	0.13	0.12	0.12

Figure 8 presents median, 16th and 84th percentile floor acceleration spectral values averaged over the frequency range from 5 to 33 Hz for each model, each case and each bin of histories. The trends of Fig. 8 are similar to those of Figs. 5 and 6. Table 4 lists the ratios of the median responses of the isolated and conventional NPPs shown in Fig 8.

4.3 Performance Spaces

The performance space is an alternate method for comparing demands on secondary systems and explicitly accounting for uncertainty and randomness in demand and response prediction (Astrella and Whittaker, 2004). Herein, the performance space is defined by two response quantities (e.g., acceleration and drift) and an 84% probability that the responses will lie within the performance space. For example, Fig. 9b presents the performance space associated with the drift ratio of node I1 and peak ground acceleration for the Boston SSE bin of earthquake histories. The choice of response quantities or indices used to define the performance space should be secondary-system dependent; typical quantities include maximum or spectral acceleration, velocity and displacement. The position of performance space indicates the overall level of structural response, the area of the space captures the randomness and uncertainty, and the shape of the space identifies the relationship (e.g., linear dependence) between the two chosen indices.

The relationship between the two chosen indices of interest must first be defined to characterize the performance space. Herein, the joint probability density function of the two indices is assumed to be a jointly lognormal. Consider Fig. 9b that is used to illustrate the calculation procedure. Figure 9b plots all 40 response points from the response-history analysis of the conventional NPP subjected to the case I Boston ground motions. The peak ground

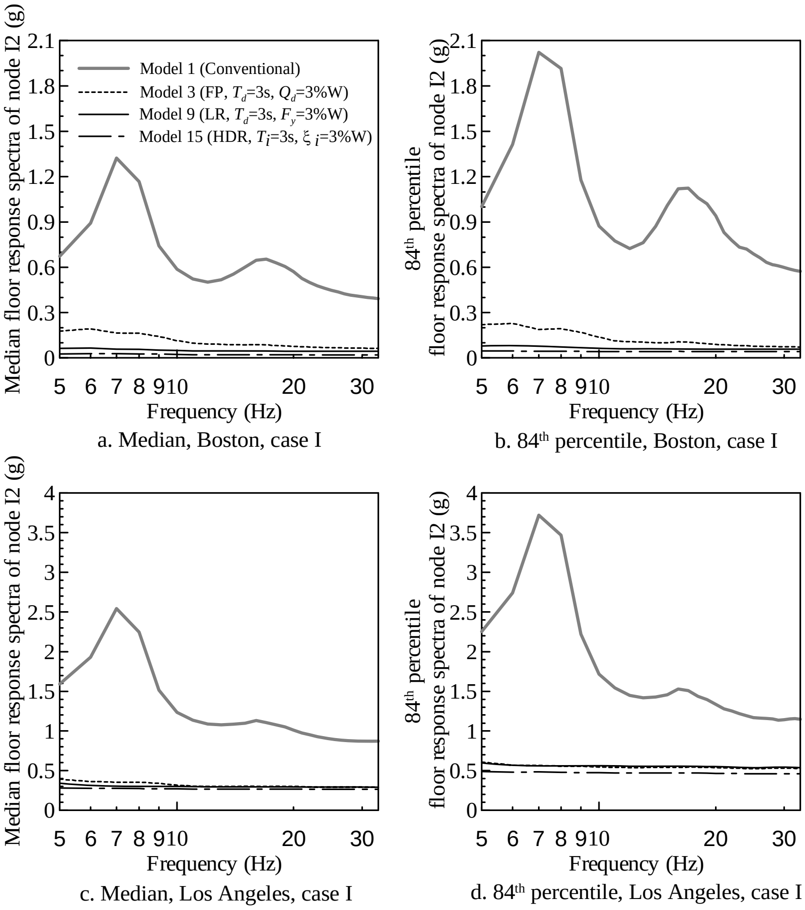


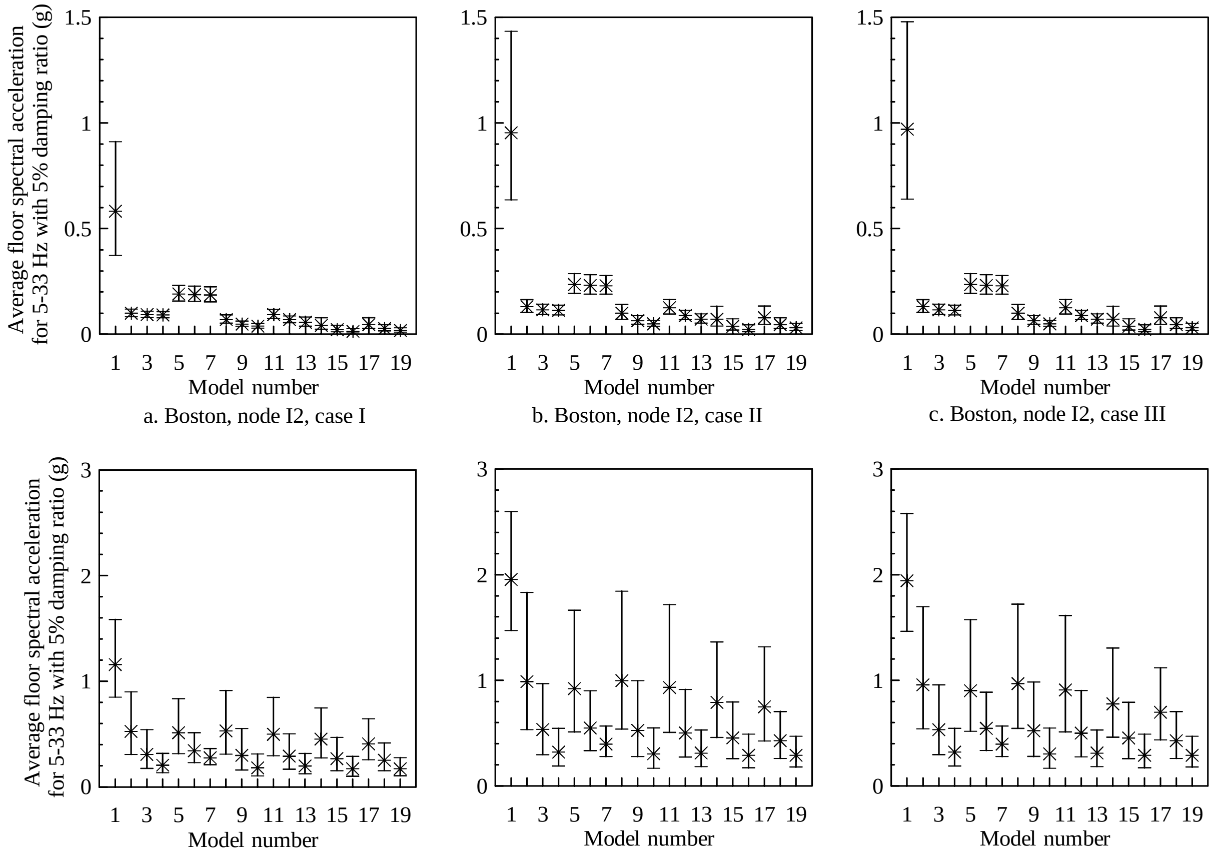
Fig. 7. Median and 84th percentile floor response spectra at node I2 for case 1

acceleration and peak drift ratio at node I1 are denoted as two random variables, *X* and *Y*, respectively. The joint probability density function of *X* and *Y* is calculated by the following equation

$$f_{XY} = \frac{1}{2\pi \cdot xy \cdot |\Lambda|^{0.5}} \exp\left[-0.5 \cdot (\tilde{z} - \tilde{m})^T \Lambda^{-1} (\tilde{z} - \tilde{m})\right] \quad (1)$$

where $\tilde{z} = [\ln x \;\; \ln y]^T$ and $\tilde{m}$ and Λ are the mean vector and covariance matrix of random variables ln*X* and ln*Y*; $\tilde{m}$ and Λ can be estimated from the sample points shown in the figure. Figure 9a shows the joint probability density function for this case. The total volume under the surface of the function is 1; the area in the XY plane corresponding to 84% of the total volume is the performance space of Fig. 9b.

Figure 10 performance spaces defined by the 5% damped average floor spectrum acceleration at node I2 over a frequency range of 5 and 33 Hz and the drift ratio between nodes I1 and I2. This performance space captures force demands on acceleration-sensitive components and systems attached at node I2 and the deformation demands on displacement-sensitive components and systems attached at nodes I1 and I2. The performance spaces associated with the isolated NPP models are much closer to the origin and smaller in area than those of the conventional NPP (Model 1), indicating much smaller seismic demands of reduced variability on the secondary systems in the isolated NPPs. The performance spaces of Fig. 11 permit a comparison of seismic demands on secondary systems for the conventional NPP and one isolated NPP (Model 13, lead-rubber isolators: see Table 1) at the Boston and Los Angeles sites for the three cases of Table 2. For the conventional NPP, the increase in demands (and absolute dispersion) on the secondary systems from SSE to BDB shaking is most significant compared with the changes in demand and dispersion for the isolated NPPs.



a. Boston, node I2, case I
b. Boston, node I2, case II
c. Boston, node I2, case III
d. Los Angeles, node I2, case I
e. Los Angeles, node I2, case II
f. Los Angeles, node I2, case III

Fig. 8. Average floor spectral accelerations at node I2

5. CLOSING REMARKS

Nineteen numerical models of a conventional and seismically isolated NPP reactor building of modern construction were studied to judge the effectiveness of protective systems to mitigate earthquake-induced demands on secondary systems. Response-history analysis of linear elastic and simple bilinear models of the reactor building was performed using earthquake histories associated with SSE and BDB earthquake shaking for east and west coast sites in the United States. (Nonlinear action was permitted in the internal structure only; the containment structure was assumed to remain elastic for all levels of shaking.) Randomness in the seismic input at each site was considered by not spectrally matching the earthquake histories and only horizontal earthquake shaking was addressed. Approximate models of sliding, FP, and elastomeric, LR and HDR bearings (or more accurately, low damping rubber bearings and linear viscous dampers), were employed for the analysis. The mechanical properties of the isolators and the reactor building were not varied for the study reported herein.

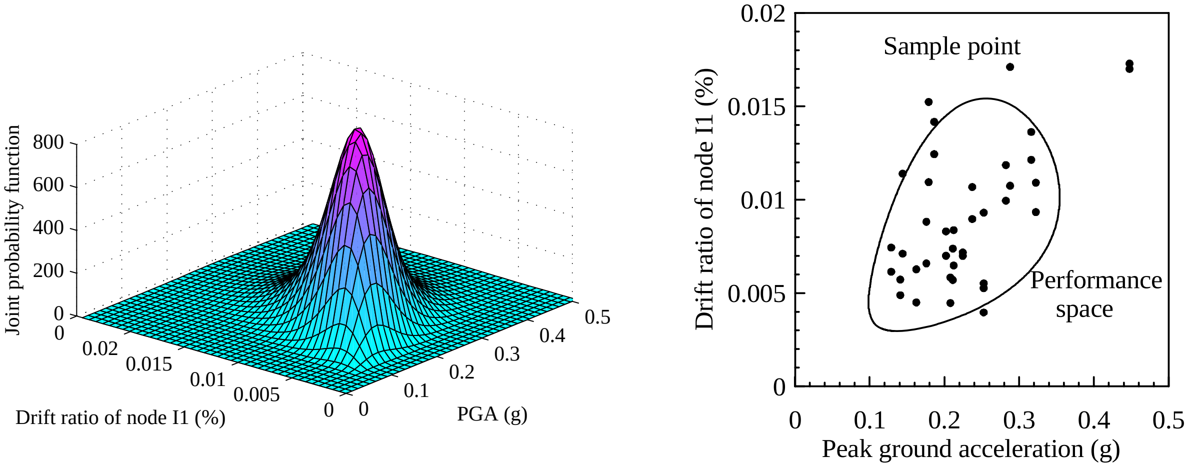
Seismic demands on secondary systems were presented for sample locations (points of attachment of a steam generator and piping) in the reactor building for each site, SSE and BDB shaking, and for linear and nonlinear response in the internal structure. For each suite of analyses, the introduction of protective systems, regardless of type, led to substantial reductions in the seismic demands on the sample secondary systems. The greatest percent reductions in demand were realized for a) the east coast site (Boston ground motions), and b) the isolation systems with a period of 4 seconds (based on elastic stiffness for the HDR models and post-yield stiffness for the FP and LR models). Greater reductions in secondary-system demand were observed for the east coast ground motions because the ratios of the ground-motion spectral ordinates at the fundamental period of the internal structure and the isolated periods of 2, 3 and 4 seconds were substantially larger for the east coast motions. Although the use of the 4-second

Table 4. Ratios of median FRS responses in conventional and isolated NPPs

Bearing type	Model number	Boston			Los Angeles		
		Case I	**Case II**	**Case III**	**Case I**	**Case II**	**Case III**
FP	2	0.17	0.14	0.13	0.45	0.51	0.49
	3	0.16	0.12	0.12	0.27	0.27	0.27
	4	0.16	0.12	0.12	0.18	0.16	0.17
	5	0.33	0.25	0.24	0.44	0.47	0.46
	6	0.32	0.24	0.24	0.30	0.28	0.28
	7	0.32	0.24	0.24	0.24	0.20	0.20
LR	8	0.12	0.10	0.10	0.46	0.51	0.50
	9	0.08	0.07	0.07	0.26	0.27	0.27
	10	0.07	0.05	0.05	0.16	0.16	0.16
	11	0.16	0.13	0.13	0.43	0.48	0.47
	12	0.12	0.09	0.09	0.25	0.26	0.26
	13	0.10	0.08	0.07	0.17	0.16	0.16
HDR	14	0.07	0.08	0.07	0.39	0.40	0.40
	15	0.04	0.04	0.04	0.23	0.23	0.23
	16	0.02	0.02	0.02	0.15	0.15	0.15
	17	0.08	0.08	0.08	0.35	0.38	0.36
	18	0.05	0.05	0.05	0.22	0.22	0.22
	19	0.03	0.03	0.03	0.15	0.15	0.15

isolators minimized the seismic demands on the secondary systems, other factors must be considered in the selection of isolator period and isolator type for NPP and SNF structures, including a) the maximum permissible displacement across the isolation interface, b) the feasibility of fabricating *long-period* elastomeric bearings, c) Operating Basis Earthquake demands on the secondary systems, and d) the effects of vertical earthquake shaking.

Performance spaces were introduced as one means of judging the seismic demands on secondary systems in NPP reactor buildings. The sample performance spaces clearly demonstrated the benefits of seismically isolating NPP and SNF buildings for both SSE and BDB shaking: median demands and absolute dispersions are reduced by factors ranging between 2 and 20.



a. Joint probability density function

b. Performance space and all sample points

Fig. 9. Performance space data for the Boston SSE bin of earthquake histories

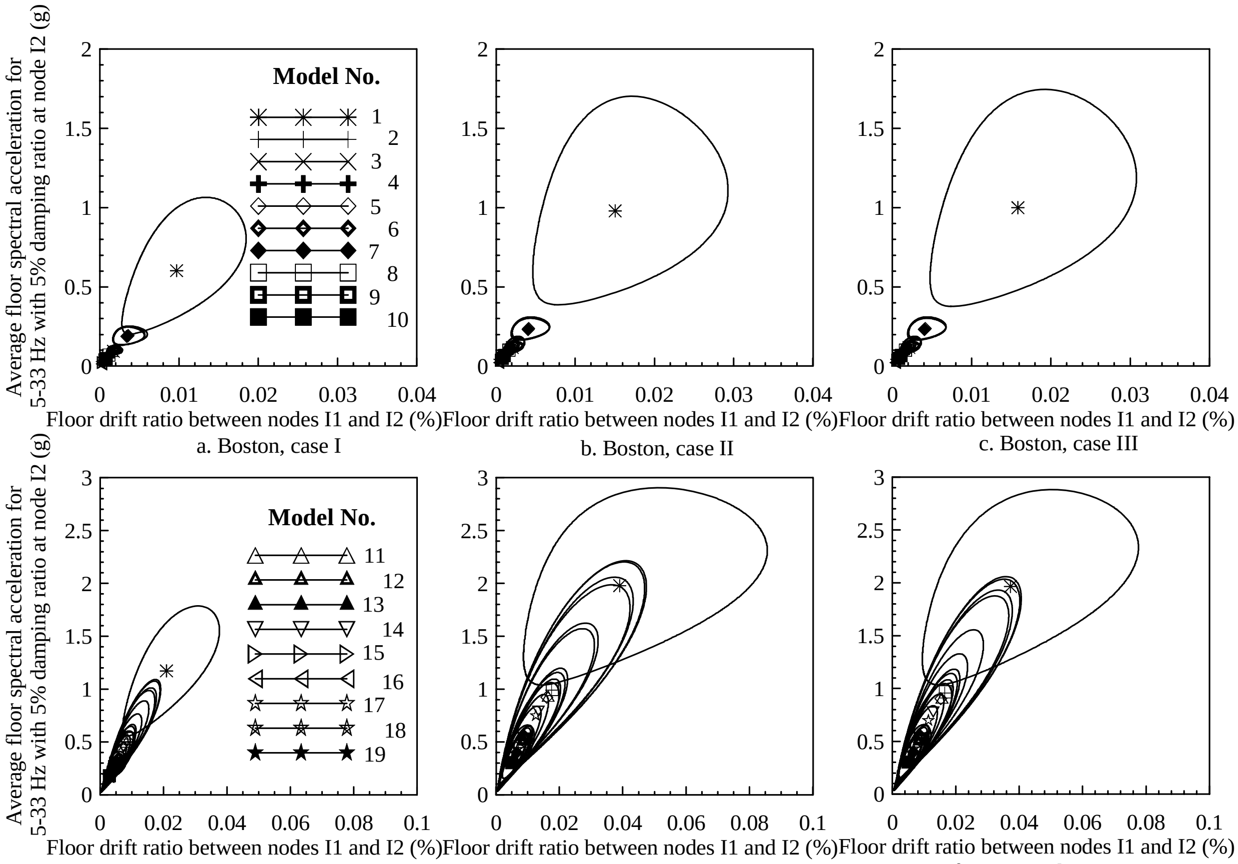


Fig. 10. Performance spaces for all models and SSE motions for Boston and Los Angeles

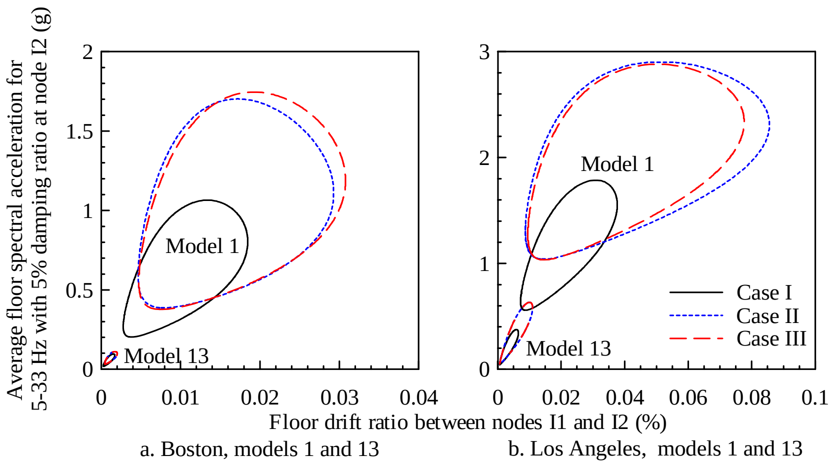


Fig. 11. Performance spaces of models 1 and 13 and cases I, II and III

ACKNOWLEDGEMENTS

Financial support for the studies described in this paper was provided in part by the Multidisciplinary Center for Earthquake Engineering Research (MCEER) through grants from the Earthquake Engineering Centers Program of the National Science Foundation (NSF), Award Number EEC-9701471, and the State of New York. The opinions, findings, conclusions expressed in this paper are those of the authors and do not necessarily reflect the views of the sponsors or the Research Foundation of the State University of New York. Dr. Ayman Saudy and Mr. Medhat Elgohary of Atomic Energy of Canada Limited (AECL) provided invaluable information on NPP reactor-building construction and provided review comments on the paper. The MCEER/NSF and AECL support is gratefully acknowledged.

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