Transactions, SMiRT-22
San Francisco, California, USA - August 18-23, 2013
Division IX (include assigned division number from I to X)

DESIGN FOR UNIFORM RISK TO STANDARDIZED NUCLEAR POWER PLANTS USING SEISMIC ISOLATION

Troy A. Morgan[1], Brian M. McDonald[2]

[1] Senior Engineer, Exponent, Inc., New York, NY (tmorgan@exponent.com)
[2] Principal Engineer, Exponent, Inc., Menlo Park, CA

ABSTRACT

The design of structures, systems and components of nuclear power plants (NPPs) in the United States is moving toward a risk-informed performance-based approach. Seismic risk is an important concern for NPPs due to the extremely rare levels of shaking that must be considered in design. In response to events at Fukushima Dai-ichi following the Great Eastern Japan Earthquake, renewed attention has been focused on the mitigation of seismic risk to NPPs. The goal of this paper is the introduction of a practical strategy for the design of seismic isolation systems for standardized NPPs who's structural and non-structural components have been certified to a particular level of seismic demand. Such a design may be considered a template of a standard NPP to be deployed in multiple geographic regions, all having varying levels of seismic hazard, fault mechanisms, soil types, etc. Whereas a conventionally-built standard NPP would be expected to have varying seismic reliability for each seismic hazard environment in which it is built, the introduction of a properly-tuned seismic isolation layer can modify the seismic effects such that the target performance objectives are met regardless of the original design basis. This paper presents a practical method to target bilinear isolation system properties (strength and stiffness) given a standard NPP design and a set of seismic hazard curves for a specific site and soil type. This method has the potential to simplify the design of new NPPs located worldwide, while providing the high level of reliability against failure that is required of such crucial facilities.

INTRODUCTION

The goal of this paper is the development of a practical approach to the design of seismic isolation systems for standardized nuclear power plants (NPPs.) In this context, "standardized" refers to an NPP who's structural and non-structural components have been certified to a particular level of seismic demand. Such a design may be considered a template of a standard NPP to be deployed in multiple geographic regions, all having varying levels of seismic hazard, fault mechanisms, soil types, etc. Whereas a conventionally-built standard NPP would be expected to have varying seismic reliability for each seismic hazard environment in which it is built, the introduction of a properly-tuned seismic isolation layer can modify the seismic effects such that the target performance objectives are met regardless of the original design basis of the superstructure and associated components. This idea has been investigated for small modular reactors by Blandford et al [2009.] The deployment of numerous small-scale, standard NPPs throughout the US is an attractive way to increase and distribute nuclear energy output with higher safety and less expense [The Economist, 2009.]

In this study, the standard NPP has been designed as a conventional structure at a site with very low seismicity, representing a practical minimum for seismic certification of the plants structures, systems, and components (SSCs.) Such an NPP could be constructed in any region with very low seismic hazard, and the structural system and non-structural components and contents could be identical for each NPP. The structural system, once designed to the original very low seismic criteria, has quantifiable force- deformation characteristics which may be adopted from the pushover curve and transformed into fragility curves for performance states applicable to NPPs. The non-structural systems, including electrical, mechanical, and plumbing equipment, and the associated distribution systems, would also be subjected to some floor response spectrum based on the initial design criteria. Once the structural and non-structural systems have been certified to the level of seismic demand, this design basis is used in the optimization of isolation systems considering a broad classification of seismic hazard, from the lowest to the most severe found in the US. The isolation systems considered in this study are assumed to be bilinear, and are applicable to either spherical sliding bearings or lead rubber bearings. However, the conceptual framework is easily extensible to other devices should they be considered.

Unless otherwise indicated, design of NPP structures, systems, and contents (SSCs) in this paper is based on the provisions and underlying philosophy described in ASCE 43-05 [2005.]

SEISMIC RESISTANCE OF STRUCTURAL SYSTEMS

The standard design of an NPP in this study is based on one constructed in a region of very low seismicity. The site is arbitrarily chosen as Houston, TX, since it is among the least active seismic regions in the US. Applying the provisions of ASCE 43-05 [2005], the design base shear for a structure at this site would be 0.21 *g*, assuming a rock site and stiff containment structure with a natural period around 0.25 sec. This base shear corresponds to an annual return period of 10,000 years, plus a modification to achieve uniform risk of failure. ASCE 43-05 also provides estimates of structural system over-strength based on the random strength having a 98% exceedance probability relative to the design strength, and an assumed dispersion of 0.35. This leads to an over-strength factor of 2.05. Assuming the NPP structure is reinforced concrete, a reasonable estimate of the pushover curve for our standard NPP is shown below in Figure 1. This pushover curve may be thought of as the seismic certification of the structure, since it is a measure of seismic resistance, and depends only on the structural geometry and detailing, and not on the assumed hazard environment.

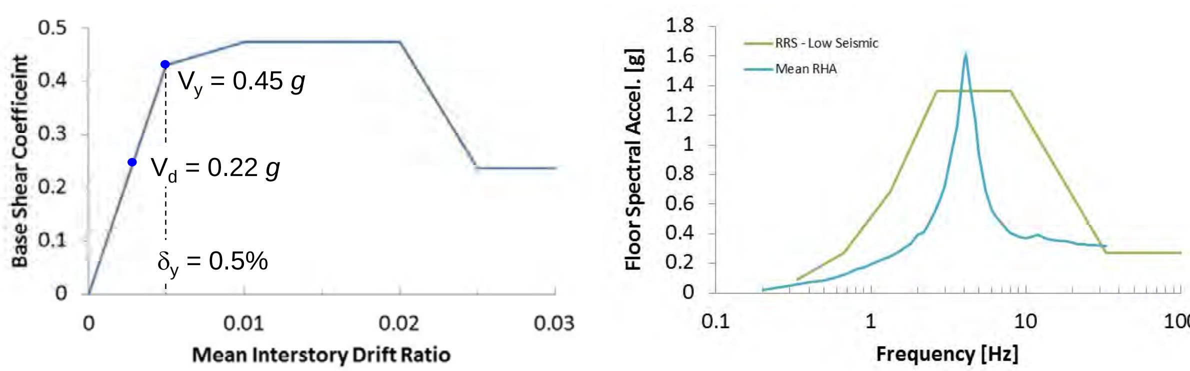

Figure 1. (left) Pushover curve for standard NPP structure (right) Comparison of mean floor spectrum from RHA with generic RRS for the standard NPP.

SEISMIC CERTIFICATION OF EQUIPMENT

The seismic certification of an equipment unit establishes the ability of that unit to withstand a particular intensity of seismic shaking and perform its function immediately following that shaking. Recognizing that earthquake shaking produces a broadband excitation, the intensity is generally represented not by a single parameter or even acceleration history, but by an acceleration response spectrum (ARS). This ARS is a demand imposed on an equipment unit by an earthquake, and is a function of many parameters: structural system supporting the equipment, earthquake magnitude, site-to-source distance, soil type, etc. The resistance of an equipment unit must also be described in terms of a spectrum, since harmonic shaking at a fixed frequency may not cause a failure, whereas the same or lower amplitude shaking at a different frequency may cause a failure due to the dynamic response of the internal components of the unit. To address this, equipment manufacturers develop what is known as the Required Response Spectrum (RRS) to describe the maximum amplitude of shaking across a range of frequencies the unit may experience while remaining functional. The ability of the equipment to function is established by testing, and if the tested response spectrum envelopes the RRS for the frequency range of interest, the equipment units are considered to be seismically certified for that RRS. Further information about the seismic certification of electrical equipment can be found in Wilkie et al [2009.]

Given the standard NPP structure, the in-structure floor spectra may be computed for a specific level of seismic hazard. To generate a spectrum, a set of spectrum-compatible ground motions was selected and scaled to the original Houston, TX site. The mean in-structure floor spectrum was then computed at the roof level based on response history analysis (RHA.) The roof is the location with highest acceleration demand. A generic spectrum was then generated based on the assumed peak roof acceleration from static analysis, and an amplification factor of 5 for peak roof spectral acceleration. The peak of the spectrum is based on the natural period of the standard NPP structure = 0.25 sec. The generic spectrum is compared with that obtained from RHA above in Figure 1. The generic spectrum is denoted the required response spectrum (RRS) since this spectrum forms the basis of seismic certification testing for all nonstructural systems and components.

Figure 2 describes the development of the test response spectrum (TRS) for equipment. Once testing establishes that the TRS exceeds the RRS for the relevant frequency range, the RRS may be used as a basis for damage assessment. If the in-structure floor spectrum from RHA exceeds the RRS within the relevant frequency range, a damage event (D) is assumed to have occurred for the ground motions being considered. Otherwise, the No Damage event (ND) has occurred.

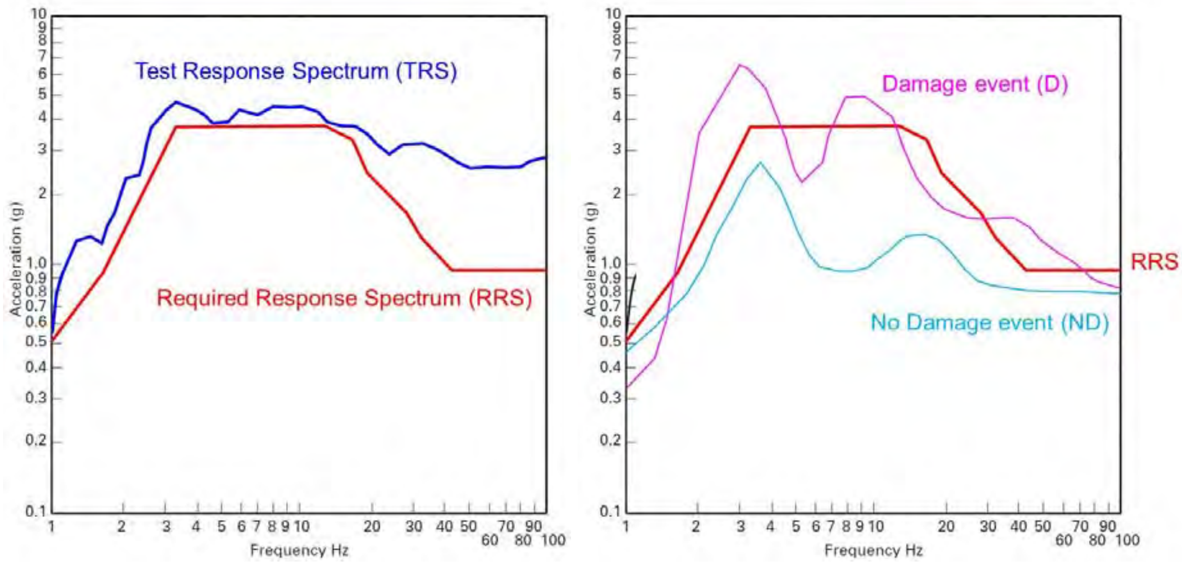

Figure 2. (Left) Comparison of TRS and RRS for acceleration-sensitive nonstructural equipment (Right) Definition of a Damage (D) or No Damage (ND) event relative to RRS

APPLICATION OF SEISMIC ISOLATION TO STANDARD NPP DESIGN

With the seismic resistance of the standard NPP structural and non-structural components defined, we apply seismic isolation technology to this standard design and enhance its performance capabilities so it may be constructed in any part of the US, regardless of the seismic hazard environment. To motive this study, three sites have been selected to represent large, moderate, and low levels of seismic hazard. These sites are located, respectively, in Los Angeles CA, St. Louis MO, and Boston MA, each with firm soil site conditions (shear wave velocity between 200 m/s and 400 m/s). The seismic hazard curves for each of these sites, assuming rock sites, are shown in Figure 3, where the intensity measure is taken to be the spectral acceleration at a period of 1-sec (commonly denoted S_{a1}.) This is an appropriate measure for isolated structures, where the period range is generally in the constant-velocity region.

In selecting the basic properties of the target isolation system, the adequate performance of the structure and non-structural components must be targeted. First considering the structure, the primary design parameter is the base shear demand. By limiting the design base shear of the isolated NPP to that of the standard NPP described previously, elastic behavior of both structures with the equivalent level of confidence is achieved. The basic procedure for this is to develop the constant velocity region of the design spectrum based on the 10,000 yr return period earthquake as

$$S_a = \frac{S_{a1}}{T_I B} \tag{1}$$

Where S_{a1} is the spectral acceleration at 1-sec with an MAFE = 1 x 10^{-4}, T_I is the isolation period, and B is a damping-based spectral reduction factor which can be taken as $B = 2.4\zeta^{0.3}$ (where ζ is the critical damping ratio of the isolation system.) By setting Eq. (1) equal to the design base shear coefficient 0.21 g for the standard NPP above, some combination of isolation period and damping can be determined. In the protection of nonstructural components, it is generally advised to use lower-damping isolators to the extent possible (Yang et al, 2010.) This process of selecting target isolation properties given a standard NPP pushover curve is described graphically in Figure 4.

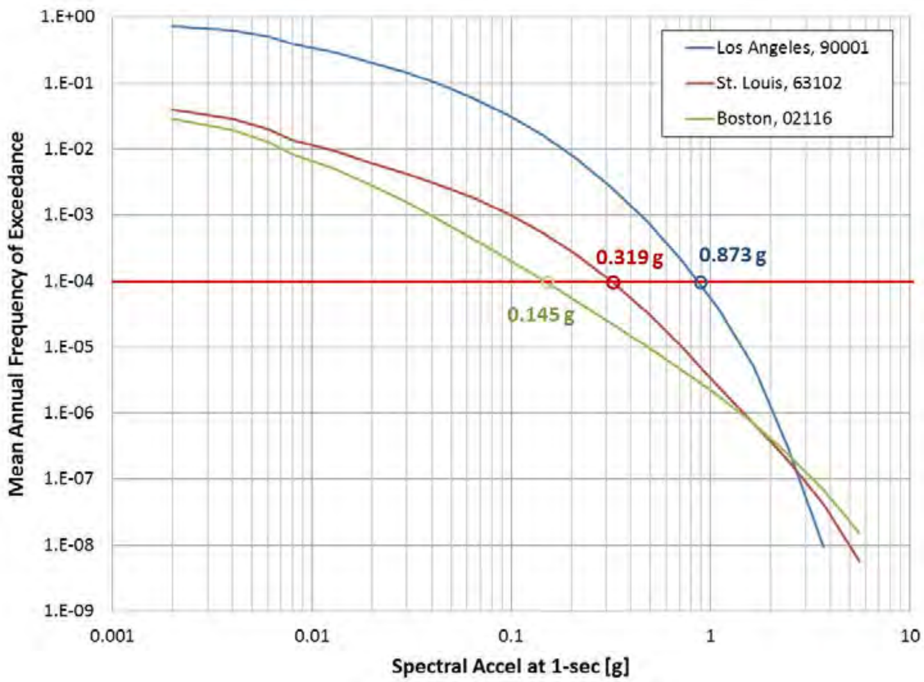

Figure 3. Comparison of seismic hazard curves for 1-sec spectral acceleration for Los Angeles, CA, St. Louis, MO, and Portland, OR, including demand at MAFE = $1x10^{-4}$.

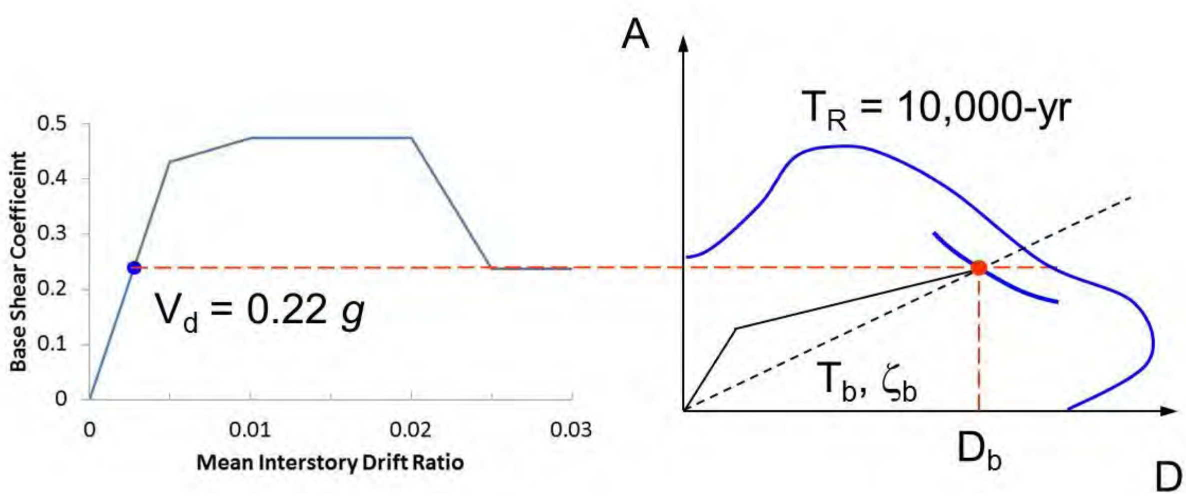

Figure 4. Procedure for determining target isolator properties given DBE spectrum

To facilitate comparison of in-structure floor spectra with the RRS for equipment, some generic in-structure spectrum must be developed based on the intensity of shaking at the floor levels of the isolated NPP. Since isolated structures may be treated as a single-degree-of-freedom system, this study considers a single representation of demand for all locations above the isolation layer. This assumption is valid for stiff superstructures having an isolation system with a substantial period separation (say fixed-to-isolated period ratio = 4.) The estimated in-structure ARS is based on the peak floor acceleration and an amplification factor to reach the peak spectral acceleration. This amplification factor depends on the hysteretic properties of the bearings, but can be as large as 4.5 (Yang et al, 2009.) For simplicity, a conservative amplification factor of 4.0 was adopted here, and nonlinear RHA supports its use over a range of isolator types.

The isolator properties are selected so that Eq. (1) is satisfied (for structural performance) and such that the in-structure demand ARS exceeds the RRS for all frequencies between 1 Hz and 35 Hz. A comparison of the ARS for the three sites with the RRS for the standard NPP is shown in Figure 5. A summary of isolator properties which provides the standard NPP with suitable performance for each of the three sites is given in Table 1. These properties are easily achievable in practice with either elastomeric or spherical sliding bearings.

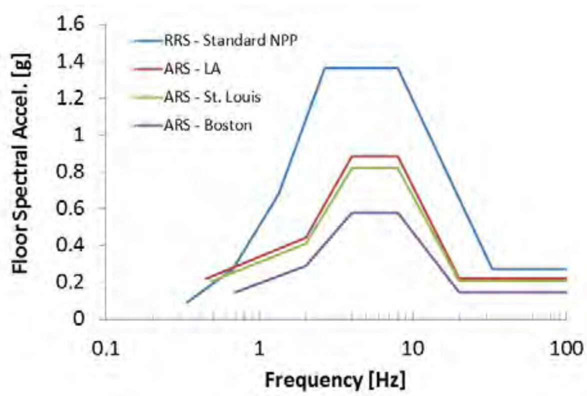

Figure 5. Comparison of floor acceleration response spectra (ARS) for three sites with the required response spectrum (RRS) for the standard NPP.

COMPUTATION OF RISK TO SEISMIC ISOLATED STANDARDIZED NPP

Once the properties of the isolation system have been selected for the standardized NPP, the risk of exceeding the target damage state may be computed based on the seismic hazard and the fragility functions for the structure, systems, and contents. To estimate the risk of damage, suitable fragilities must be developed for all structural and non-structural components. Since the goal of this study is equivalent damage risk for all standardized NPPs, fragilities were developed such that the conventional, low-seismic NPP achieves a target mean failure interval of 10,000 years. Once these fragilities are estimated, they are used in subsequent analysis of all seismic isolated NPPs.

Let the fragility for deformation-sensitive components be defined as $P[D|\Delta=\bar{\delta}]=F_D(\bar{\delta})$, or a likelihood of exceedance damage state *D* given the observance of drift demand $\bar{\delta}$. Similarly, we define the fragility for acceleration-sensitive components as $P[D|A=\bar{a}]=F_D(\bar{a})$, given the observance of acceleration demand $\bar{a}$. Here, the acceleration *A* is left general, and could be a floor spectral acceleration at any frequency, with peak floor acceleration simply being the special case of *A* at infinite frequency.

The goal of this risk assessment is to estimate a mean annual frequency of exceedance (MAFE) of some damage state *D*. This damage state is taken here to be onset of damage to the reactor core, but could be any damage state of interest. This annual risk (or MAFE) of exceeding damage state D is denoted λ_D. To compute this risk, it will be necessary to develop a fragility function which jointly considers the individual fragilities of all known structural and non-structural components. Here we consider the case with two components, one drift-sensitive, and the other acceleration-sensitive. This can be easily generalized to any number of components. Consider a bivariate fragility, where the definitions of drift Δ and acceleration *A* are as before:

$$P[D \,|\, (\Delta, A) = (\bar{\delta}, \bar{a})] = F_D(\bar{\delta}, \bar{a}) \tag{2}$$

It is possible to define this bi-variate fragility $F_D(\bar{\delta}, \bar{a})$ in terms of previously-defined uni-variate fragilities by recognizing that survival of the system depends on survival of all components. As a result, for an *N*-component system, the likelihood of survival, or a damage state of No Damage (ND), is

$$P[ND] = \prod_{j=1}^{N} \left[1 - F_j(\bar{x}_j)\right] \tag{3}$$

Where the uni-variate fragility for the jth component is $F_j(\bar{x}_j)$. The likelihood of damage is therefore

$$P[D] = 1 - \prod_{j=1}^{N} \left[1 - F_j(\bar{x}_j)\right] \tag{4}$$

Specializing this to the bi-variate case of interest, the fragility $F_D(\bar{\delta}, \bar{a})$ is computed as

$$F_D\left(\bar{\delta}, \bar{a}\right) = 1 - \left[1 - F_D(\bar{\delta})\right]\left[1 - F_D(\bar{a})\right] \tag{5}$$

A 3-D surface plot of this bi-variate fragility function is shown in Figure 6.

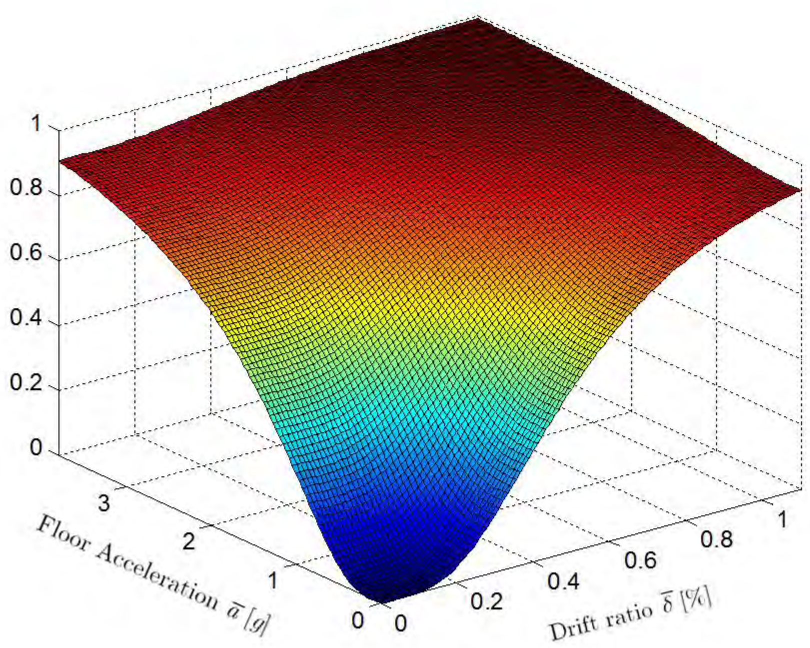

Figure 6. Surface plot of bi-variate fragility function for both drift- and acceleration-sensitive components

Once the bi-variate fragility has been defined, it is possible to compute of risk of exceeding damage state *D* using total probability. Given some spectral acceleration *S*, RHA can be used with a suite of scaled ground motions to estimate a joint distribution of both drift and acceleration. This joint distribution, $P\left\{(\Delta, A) = (\bar{\delta}_m, \bar{a}_n) \middle| S = s_j\right\}$, is generally assumed to be bi-variate lognormal, but could also be taken as an empirical distribution defined only by simulation data. In general, the practical computation of the risk λ_D is done numerically, and therefore the assumption of a distribution function is not necessary. This risk λ_D may be computed as

$$\lambda_D = \sum_{n=1}^{N}\sum_{m=1}^{M}\left\{P[D\left|(\Delta, A) = (\bar{\delta}_m, \bar{a}_n)\right]\sum_{j=1}^{J} P\left\{(\Delta, A) = (\bar{\delta}_m, \bar{a}_n)\middle| S = s_j\right\} P\left[S = s_j\right]\right\} \tag{6}$$

Where the random spectral acceleration takes on possible values $[s_1,s_J]$, the random drift takes on possible values $[\bar{\delta}_1, \bar{\delta}_M]$, and the random floor acceleration takes on values $[\bar{a}_1, \bar{a}_N]$. These are all chosen to be some practical range that includes all values that could be potentially observed. The mean failure interval can be computed simply as the reciprocal of the annual risk, or $1/\lambda_D$. For the isolation systems investigated in this study, the mean failure intervals are shown in Table 1. From this data, it is clear that the isolated standardized NPPs are at least as reliable as the conventional NPP located in the very low seismic region.

Table 1. Isolation system period, damping, and displacement demand for each of the three selected sites.

Site	T_b	ζ_b	Δ_b	Mean Failure Interval
LA	4.0 s	20%	34.6"	14,100 yrs
St. Louis	2.0 s	15%	8.04"	15,700 yrs
Boston	1.5 s	10%	3.19"	21,440 yrs
Houston	--	--	--	12,550 yrs

CONCLUSIONS

This paper concludes with practical method to target bilinear isolation system properties (stiffness and damping) given a standard NPP design and a set of seismic hazard curves for a specific site and soil type. The standard NPP was designed as a conventional structure assuming a site with very low seismicity. A procedure for developing optimal strength and stiffness of the isolators was presented starting with the seismic hazard environment and soil type, certified design strength of standard NPP, and required response spectra (RRS) for nonstructural components. This method therefore incorporates the performance both structural- and non-structural systems, and uses nonlinear response history analysis to validate the procedure presented. This design approach has the potential to simplify the design of new NPPs located in all parts of the US and around the world, while providing the high level of reliability required for such crucial facilities.

REFERENCES

ASCE/SEI 43-05, Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, American Society of Civil Engineers, 2005.

Blandford, E., Keldrauk, E., Laufer, M., Mieler, M., Wei, J., Stojadinovic, B., and Peterson, P.F. (2009). Advanced seismic base isolation for modular reactors. Report No. UCBTH-09-004, University of California, Berkeley, CA.

The Economist Technology Quarterly, Mini nuclear reactors: Thinking small. http://www.economist.com/node/17647651

Wilkie E, Paul FM, Ahmed MA, and Glen NM, Seismic White Paper SA12501SE, Eaton Corporation, August 2009.

Yang TY, Konstantinidis D, Kelly JM. The Influence of Isolator Hysteresis on Equipment Performance in Seismic Isolated Buildings. EERI Spectra, **26**(1), pp. 275–293, February 2010.