



DEVELOPMENT OF RESPONSE-BASED FRAGILITY CURVES FOR SEISMIC PROBABILISTIC RISK ASSESSMENT OF NUCLEAR POWER PLANTS

Chang-Ching Chang¹, Chin-Chien Chou², Ying-Hsiu Shen³, and Yin-Nan Huang⁴

¹ Assistant Researcher, National Center for Research on Earthquake Engineering, Taiwan
(ccchang@ncree.narl.org.tw)

² Graduate Student, Dept. of Civil Engineering, National Taiwan University, Taiwan

³ Assistant Researcher, National Center for Research on Earthquake Engineering, Taiwan

⁴ Assistant Professor, Dept. of Civil Engineering, National Taiwan University, Taiwan

ABSTRACT

Seismic probabilistic risk assessment (SPRA) determines the annual frequency of unacceptable performance of a nuclear power plant (NPP), such as core melt and release of radiation. In conventional SPRA methodologies, fragility curves express the conditional probability of failure of a structure or component for a given ground-motion parameter, such as peak ground acceleration (PGA) or spectral acceleration. However, damage and failure of NPP components are more closely tied to structural response parameters than to ground motion parameters. The use of ground-motion parameters greatly increases the dispersion in the fragility curves. In the new SPRA procedure proposed by Huang *et al.* (2011a, b), the fragility curves are defined in terms of structural response parameters, such as floor spectral acceleration and peak story drift. The use of structural response parameters in developing component fragility curves reduces the dispersion in the curves and enables the curves to be independent of seismic hazard and closely related to component capacity. In this study, the procedures to develop response-based fragility curves for structural components and equipment are proposed. Response-based fragility curves were developed using the proposed methodology for sample structural components and equipment in a sample NPP in Taiwan. The developed fragility curves were used in a SPRA for a sample NPP presented in the companion paper entitled "Seismic probabilistic risk assessment of nuclear power plants using response-based fragility curves" (Shen *et al.* 2013).

INTRODUCTION

Seismic probabilistic risk assessment (SPRA) determines the annual frequency of unacceptable performance of a nuclear power plant (NPP), such as core melt and release of radiation. In recent years, SPRA studies have been performed on a large number of nuclear power plants in USA. Seismic fragility analysis for structural and nonstructural components in NPPs are needed in the SPRA procedure. The methodology for evaluating seismic fragilities of structures and equipment is documented in the PRA Procedures Guide (USNRC, 1983) and is more specifically described for application to NPPs in Kennedy and Revindra (1984) and Reed and Kennedy (1994). In-depth guidelines for performing a fragility analysis are available in EPRI (2002). This document updates the fragility analysis methodology provided in EPRI TR-103959 (Reed and Kennedy, 1994).

Seismic fragility of a structure or equipment is defined as the conditional frequency of its failure for a given value of the seismic response parameter. In conventional SPRA methodologies, fragility curves express the conditional probability of failure of a structure or component for a given ground-motion parameter, such as peak ground acceleration (PGA) or spectral acceleration. However, damage and failure of NPP components are more closely tied to structural response parameters than to ground

motion parameters. The use of ground-motion parameters greatly increases the dispersion in the fragility curves. Moreover, the median and dispersion of a component fragility curve expressed in terms of ground-motion parameters are dependent on both the capacity of the component and the response (demand) of the structure where the component is attached to.

In the new SPRA procedure proposed by Huang *et al.* (2011a, b), the fragility curves are defined in terms of structural response parameters, such as floor spectral acceleration (FSA) and peak story drift and nonlinear response-history analysis of the NPP is performed to estimate the damage state (pass/failure) of structures, systems and components (SSCs). The use of structural response parameters in developing component fragility curves reduces the dispersion in the curves and enables the curves to be independent of seismic hazard and closely related to component capacity. In this study, the methodology to develop response-based fragility curves for SSCs will be proposed. The proposed methodology is partly based on that of Reed and Kennedy (1994) except that the fragility curve of each component is expressed in terms of the structural response parameter that best estimates damage to the component. A double lognormal model is still used to explicitly express aleatory randomness and epistemic uncertainty in fragility curves. Response-based fragility curves were developed using the proposed methodology for sample structural components and equipments in a sample NPP in Taiwan. The developed fragility curves were used in the SPRA of the sample NPP presented in the companion paper entitled “Seismic probabilistic risk assessment of nuclear power plants using response-based fragility curves” (Shen *et al.* 2013)

SEISMIC FRAGILITY ANALYSIS METHODOLOGY

Fragility Model

Seismic fragility curves for structural and nonstructural components in NPPs are needed in a SPRA to estimate the frequencies of occurrence of initiating events and the failures of different safety systems. The lognormal distribution has become the most widely used distribution for developing fragility curves. A lognormal distribution for a random variable can be fully defined by two parameters, the median and logarithmic standard deviation. The latter parameter represents the dispersion in the variable. The sources of the dispersion are distinguished into two types for developing fragility curves for structural and nonstructural components in NPPs: 1) epistemic uncertainty, for the variability due to the lack of knowledge for the procedure and variables used in the analysis process, for example, the variability in the strength of a shear wall, which could be tested to eliminate the uncertainty; and 2) aleatory randomness, for the variability that is inherent in the used variables and cannot be practically reduced, for example, the variability in peak ground acceleration for a given earthquake magnitude and distance.

Reed and Kennedy (1994) present a methodology for developing fragility curves for use in a SPRA and use a double lognormal model to consider the two types of variability separately. The model is

$$A = \bar{A} \cdot \varepsilon_R = A_m \cdot \varepsilon_U \cdot \varepsilon_R \quad (1)$$

where A is the random variable for the capacity of the component and the capacity is defined in terms of a ground-motion parameter, such as peak ground acceleration or spectral acceleration at a given period; \bar{A} , which is equal to $A_m \cdot \varepsilon_U$, is a random variable for the median capacity of the component; A_m is a deterministic value representing the median of \bar{A} ; and ε_U and ε_R are two lognormally distributed random variables with medians both equal to one and logarithmic standard deviations of β_U and β_R , respectively. Variables ε_U and ε_R represent the uncertainty and randomness in A , respectively. The model of (1) can be used to generate a family of fragility curves for each SSC in NPPs.

Failure Modes

The first step in generating fragility curves is to develop a clear definition of what constitutes failure for each of the critical elements in the plant. This definition of failure must be agreeable to both the structural analyst generating the fragility curves and the system analyst who must judge the consequences of component failure. Several modes of failure may have to be considered and fragility curves may have to be generated for each of these modes.

Estimation of Fragility Parameters

In conventional fragility analysis, such as the procedure of Reed and Kennedy (1994), the fragility parameters are often estimated using an intermediate random variable, also known as the factor of safety, F . The factor of safety, F , is defined as follows

$$F = \frac{A}{A_{SSE}} \quad (2)$$

where A is actual seismic capacity of a component and A_{SSE} is actual response of the component subjected to SSE shaking. Both A and A_{SSE} are characterized in terms of ground-motion-intensity parameters, such as peak ground acceleration and spectral acceleration. The random variable F is considered as a lognormal distribution with a median of F_m and logarithmic standard deviations of β_U and β_R identical to those for A . The parameter F_m is defined as

$$F_m = \frac{A_m}{A_{SSE}} \quad (3)$$

where A_m is the median of A .

In the SPRA procedure of Huang et al. (2011a, b), fragility curves are defined in terms of structural response parameters and nonlinear response-history analysis is used to estimate distributions of structural responses at given earthquake intensity levels. The fragility curves developed in this study were defined in terms of structural response parameters, such as stress, interaction ratio, and floor spectral acceleration. Therefore, Equation (2) is rewritten by

$$A_R = F_R \cdot A_{R,SSE} \quad (4)$$

where A_R is actual seismic capacity of a component and $A_{R,SSE}$ is actual response of the component subjected to SSE shaking. Both are characterized in terms of structural response parameters. Parameter F_R is the ratio of A_R and $A_{R,SSE}$. The procedure used in this study for the development of F_R is partly based on that of Reed and Kennedy (1994) for the factor F . Since the variability in ground motions and structural modeling are taken into account when performing response-history analysis, the median and the logarithmic standard deviations of the new factor of safety F_R is different from that of the conventional F . Fragility parameter values derived for the SSCs are discussed in the following subsections.

Fragility Analysis Variables for Structures

In Reed and Kennedy (1994), the factor of safety F for a structure is the product of capacity factors (F_C) and structural response factors (F_{RS}).

$$F = F_C F_{RS} \quad (5)$$

Table 1 lists the factors of safety for F_C and F_{RS} for structures. Factor F_C is computed using the product of a strength factor (F_S) and an inelastic energy absorption factor (F_μ).

The strength factor, F_S , represents the ratio of ultimate strength or strength at loss-of-function to the stress calculated for A_{SSE} .

$$F_S = \frac{S - P_N}{P_T - P_N} \quad (6)$$

where S is the strength of the structural element for a specific failure mode; P_N is the normal operating load (i.e. dead load, operating temperature load, etc); P_T is the total load on the structure.

The inelastic energy absorption factor, F_μ , accounts for the capability of structures and equipment to absorb energy beyond yield without loss-of-function.

The structural response factor (F_{RS}) is used to capture the difference between 1) the actual structural response for a given PGA level and 2) the structural response used in the design process for a specific deterministic ground-motion design parameter. The factor F_{RS} consists of a number of sub-factors as listed in Table 1 and modeled as:

$$F_{RS} = F_{SA} F_\delta F_M F_{MC} F_{ECC} F_{SSI} \quad (7)$$

where F_{SA} is the spectral shape factors representing variability in response due to difference in the median ground motion and design ground response spectra; F_δ is the damping factor representing variability in response due to difference in expected damping at failure and damping used in the analysis; F_M is modeling factor; F_{MC} is mode combination factor; F_{EC} is earthquake component combination factor; and F_{SSI} is the factor to account for effect of soil-structure interaction including the reduction of input motion with depth below the surface.

In the procedure of Huang *et al.* (2011a, b), the fragility curves are defined in terms of structural response parameters and the relationship between structural response and ground-motion intensity is obtained using nonlinear response-history analysis. Therefore, the randomness and uncertainty in structural responses for a given ground-motion intensity do not need to be included in the response-based fragility curves unless they are not properly considered in the process of response-history analysis and the numerical model of the structure.

In this study, the factor F_R of Eq. (4) is also modeled as the product of the capacity factors and structural response factors of Table 1. Ideally, all sub-factors for F_{RS} do not need to be included in the computation due to the reason described in the last paragraph. However, as presented in Table 1, β_R for F_{ECC} and β_U for F_δ and F_M are still included in the fragility functions developed in this study since the randomness and uncertainty for those factors are not addressed in the numerical models for the response-history analysis. The consideration for each sub-factor for F_{RS} is presented herein:

For the spectral shape factors (F_{SA}), since the earthquake record as seismic analysis input is selected by considering the difference between the site-specific response spectrum shape and the median design ground response spectrum, the $F_{SA}=1$, and uncertainty and randomness is not considered.

Factor F_{SSI} is the factor to account for effect of soil-structure interaction including the reduction of input motion with depth below the surface and ground motion incoherence factor. The dynamic analysis is based on the model considering soil-structure interaction effect so that it is reasonable using median factor of 1.0 and all uncertainty and randomness variability is not considered.

Uncertainty in modeling influences primarily the mode shapes and the modal frequencies of the actual structure. In this study, the different models are used in dynamic analysis by changing structural parameters, such stiffness or mass, and all uncertainty and randomness variability for F_M do not need be considered. However, if the model is considered to be the best estimate and the resulting dynamic characteristics are median-centered. The model factor is thus unity. The β_U due to mode shape is estimated to be 0.15 and the corresponding value for β_U in model frequency is 0.05.

Factor F_{MC} is mode combination factor accounting for variability in response due to the method used in combining dynamic modes of response. All uncertainty and randomness variability is not considered.

Factor F_{ECC} is earthquake component combination factor accounting for variability in response due to the method used in combining earthquake components. The effects of multi-directional earthquake excitation on structural response depends on the geometry, dynamic response characteristics, and relative magnitudes of the two horizontal and the vertical earthquake components. The design method is SRSS, according to RG 1.92, which is considered to result in median-centered response. F_{ECC} is therefore 1.0 if the design method is SRSS. In addition, since the actual building response could be higher or lower than calculated, some uncertainty due to randomness is assigned. The earthquake component combination effect on the wall design is thus estimated to have a logarithmic standard deviation of $\beta_R = 0.15$.

Table 1: Variables for fragility analysis for structures

Factor	F_R	β_R	β_U
Capacity			
Strength, F_S	Y	Y	Y
Inelastic energy absorption, F_μ	Y	Y	Y
Response			
Spectral shape factor, F_{SA}	*	*	*
Damping factor, F_δ	*	*	Y
Modeling factor, F_M	*	*	Y
Mode combination factor, F_{MC}	*	*	*
Earthquake component combination factor, F_{ECC}	*	Y	*
Soil-structure interaction effect, F_{SSI}	*	*	*

* The factors do not need to be considered in the response-based fragility curves.

Fragility Analysis Variables for Equipment by Analysis

In the methodology of Reed and Kennedy (1994), the factor of safety for equipment and components in NPPs, F , is the product of three factors, namely, a capacity factor (F_C), a structures response factor (F_{RS}), and an equipment response factor (F_{RE}):

$$F = F_C F_{RE} F_{RS} \quad (8)$$

The capacity factor for equipment (F_C) is computed as the product of the strength factor (F_S) and the inelastic energy absorption factor (F_μ). The equipment response factor (F_{RE}) is the ratio of equipment response calculated in design to the realistic equipment response, and both the responses are calculated for the design floor spectra. Factor F_{RE} is the factor of safety inherent in computation of equipment response. For equipment qualified by analysis, the important variables that influence response and variability are qualification method, spectral shape, modeling, equipment damping, combination of modal responses, and combination of earthquake components (Kennedy and Ravindra 1984). Since the procedures of Huang et al. (2011a, b) often involve the use of stick models where equipment is not modeled explicitly, the impact of the variables listed herein for F_{RE} on the fragility curves of equipment is considered in this study using the same procedure as that of Reed and Kennedy (1994).

The structural response factor, F_{RS} , is computed based on the response characteristics of the structure where the component (equipment) locates. The applicable variables are spectral shape, ground motion incoherence factor, damping, modeling, mode combination, and soil-structure interaction effect including the reduction with depth of seismic input, similar to the response factors of Table 1. Most of the structural response factors do not need to be considered since the fragility curves are developed in terms of structural response parameters rather than PGA and the randomness and uncertainty in the response

factors of Table 1 have been considered in the process of response-history analysis and the numerical model of the structure.

Basic fragility analysis variables for equipments by dynamic testing

For equipment qualified by shake table test, the following formulas are used to develop seismic fragility of the equipment in the methodology of Reed and Kennedy (1994):

$$A = \frac{TRS_C}{RRS_C} \cdot F_D \cdot F_{RS} \cdot A_{SSE} \quad (9)$$

$$TRS_C = TRS \cdot C_T \cdot C_I ; RRS_C = RRS \cdot C_C \cdot D_R \quad (10)$$

where A is the median capacity of the equipment in terms of ground-motion-intensity parameters; TRS is the equipment test response spectrum capacity; RRS is the equipment seismic demand in terms of required response spectrum; C_T is the clipping factor for narrow-banded TRS ; C_I is the capacity increase factor; C_C is the clipping factor for narrow-banded demand; D_R is the demand reduction factor; F_D is the broad frequency input spectrum device capacity factor; F_{RS} is the structural response factor of the building; A_{SSE} is the peak ground acceleration of the SSE design ground response spectrum.

In the Huang et al. SPRA procedure, fragility curves are defined in terms of structural response parameters. For equipment qualified by shake table test, the fragility for the equipment can be obtained directly from the testing data, i.e.,

$$A_R = TRS_C \cdot F_D \cdot F_{RS} \quad (11)$$

where A_R is the median capacity of the equipment in terms of floor spectral acceleration at the location of the component (equipment) support. If only the ratio of TRS_C to RRS_C , rather than TRS_C itself, is available, the following equation provides an alternative to estimate A_R :

$$A_R = \frac{TRS_C}{RRS_C} \cdot F_D \cdot F_{RS} \cdot A_{R,SSE} \quad (12)$$

where $A_{R,SSE}$ is the peak floor response spectral acceleration of enveloped floor response spectra for SSE.

Fragility Curves in terms of Interaction Ratio

The SPRA methodology of Huang et al. (2011a, b) requires response-history analysis, which is often conducted using stick models in which non-structural components and equipment are not modeled explicitly. For many types of equipment, the failure mode is governed by the failure of anchorage bolts of the equipment. A key parameter for this type of failure is the tension and shear in the bolts due to earthquake loading. Since the equipment is not modeled explicitly, the force demands in the bolts cannot be obtained directly from response-history analysis. This section provides an approach to develop fragility curves as a function of interaction ratio (IR) for tension and shear in anchor bolts of equipment in NPPs and the procedure to compute the IR value using the results of response-history analysis.

Equation (13) defines six parameters:

$$\alpha_{VX} = \frac{V_X}{S_{A,X_design}} ; \alpha_{VY} = \frac{V_Y}{S_{A,Y_design}} ; \alpha_{VZ} = \frac{V_Z}{S_{A,Z_design}} ;$$

$$\alpha_{TX} = \frac{T_X}{S_{A,X_design}} ; \alpha_{TY} = \frac{T_Y}{S_{A,Y_design}} ; \alpha_{TZ} = \frac{T_Z}{S_{A,Z_design}} \quad (13)$$

where V_X, V_Y , and V_Z (T_X, T_Y , and T_Z) are the shear (tension) in a bolt in the X, Y and Z directions, respectively, corresponding to the design floor response spectral accelerations S_{A,X_design} , S_{A,Y_design} ,

and S_{A,Z_design} in the X, Y and Z directions, respectively, at the location of the bolt at a frequency and damping of interest. The values of these parameters can be obtained either by performing dynamic analysis or by using existing data in the design documents for the equipment of interest.

For a given ground-motion set i , the floor response spectral accelerations at the location of the equipment at the frequency and damping of interest in the X, Y and Z directions (termed $S_{FA,X}$, $S_{FA,Y}$, and $S_{FA,Z}$, respectively) can be obtained using response-history analysis. The shear and tension in the bolt and the corresponding IR for the bolt subjected to the ground-motion set i are calculated by

$$T_i = \sqrt{(\alpha_{TX} \cdot S_{FA,X})^2 + (\alpha_{TY} \cdot S_{FA,Y})^2 + (\alpha_{TZ} \cdot S_{FA,Z})^2}; V_i = \sqrt{(\alpha_{VX} \cdot S_{FA,X})^2 + (\alpha_{VY} \cdot S_{FA,Y})^2 + (\alpha_{VZ} \cdot S_{FA,Z})^2} \quad (14)$$

$$IR_i = \left(\frac{V_i}{V_U} \right)^2 + \left(\frac{T_i}{T_U} \right)^2 \quad (15)$$

where V_U is the median shear capacity for the bolt; T_U is the median tension capacity for the bolt. The fragility curve of the equipment has a median value of IR equal to 1. The dispersion in the fragility curve can be estimated using the approach described in the previous section entitled “Fragility Analysis Variables for Equipment by Analysis”.

EXAMPLE I: DIESEL FUEL OIL DAY TANK

In this example, response-based fragility parameters for a diesel fuel oil day tank in a sample NPP plant in Taiwan were developed based on the information provided in the reports documenting the analysis and design of the tank. The diesel fuel oil day tank is a horizontal tank mounted on three saddles in the reactor building (RB) of the sample NPP at an elevator of 23.5 m. The seismic qualification of the tank was demonstrated by analysis. The equipment was designed to resist load cases arising from operating conditions, upset conditions, emergency conditions, and accidental conditions. The accidental condition (i.e., Service Level D) load case included dead weight, thermal load, nozzle loads from attached piping, SSE, design pressure, and SRV and LOCA loads. The demands of the tank subjected to static loads (i.e., dead weight, pressure, nozzle loads, and thermal expansion) and dynamic loads (i.e., OBE, SSE, SRV and LOCA) were computed using the finite element code ANSYS. The frequencies of the dominant modes are 22 Hz in the longitudinal (X) direction, 31 Hz in the transverse (Y) direction and 54 Hz in the vertical (Z) direction. The maximum membrane stress and membrane plus bending stress were reported for different elements (i.e., shell, support saddle and nozzles) for the service levels A, B, C and D. The service level D combination includes stresses from pressure, dead weight, thermal and nozzle loads along with the SRSS of stresses from SSE, LOCA and SRV.

We assume that the capacity of the anchor bolts of the tank is governed by the tensile failure of the bolt rather than by the concrete pull out. Based on the information provided in the design report for the tank, the median strength factor against yielding of the tank structure is 2.11 and the median strength factor for anchorage is 1.71. Therefore, the failure of anchorage is deemed to be the controlling failure mode. Since the failure mode considered herein for the anchor bolts of the tank is a brittle failure, the inelastic energy absorption factor is unity.

The equipment response factors for this equipment include qualification method factor, modeling factor, damping factor, modal combination factor and earthquake component combination factor. The qualification method factor is used to reflect the bias and uncertainty in the in-structure response spectra used in the equipment fragility analysis. In this case, the floor response spectra used in the fragility analysis were computed using dynamic analysis, of which the results are deemed the real demands. Therefore, the median value for the qualification method factor is 1.0 and the values of β_R and β_U are taken to be 0.

The 3%-damped spectral acceleration was used in the qualification analysis for the load case that includes the SSE. The median damping value for diesel fuel oil day tank is assumed to be 5%. The natural frequency of the sample tank in the longitudinal direction is 22 Hz. The widened and enveloped floor

spectral accelerations at a frequency of 22 Hz at the location of the tank are 1.62 g and 1.50 g for damping ratios of 3% and 5%, respectively. The median damping factor F_δ calculated as the ratio of these spectral values is 1.08. The β_U value is 0.06 calculated following the recommendation of Reed and Kennedy (1994) by taking the 3.5% damped spectral value as one standard deviation below the median damped spectral value.

Since the dynamic analysis was used to calculate the equipment response, the median modeling factor is unity. The uncertainty in the response due to modeling comes mainly from the uncertainty in the mode shape for which a β_U of 0.1 is used in this study.

The day tank was analyzed using modal analysis and ANSYS code. This analysis is judged to be median centered. Since the lowest fundamental frequency is 22 Hz and the other modes have frequencies higher than 30 Hz, the β_R for modal combination factor is estimated as 0.10. The SRSS method was used to combine loads from the three earthquake components for the anchored bolts of the tank. This is considered median centered. Based on the recommendation in Reed and Kennedy (1994), the logarithmic standard deviation representing the randomness in earthquake component combination factor, β_R , is taken as 0.18.

Table 2 summarizes the structural response factor F_{RS} estimated for the sample tank. Since the fragility function for the sample tank is expressed in terms of structural-response parameter, not PGA, the median values for the six factors of Table 2 are all equal to 1 and the dispersion values are either ignored or no greater than the values for a PGA-based fragility function.

Table 2: Structural response factors

Factor	Median	β_R	β_U	β_C
Spectral shape factors, F_{SA}	1.0	*	*	*
Damping factor, F_δ	1.0	*	0.15	0.15
Soil-structure interaction effect, F_{SSI}	1.0	*	*	*
Modeling factor, F_M	1.0	*	0.16	0.16
Mode combination factor, F_{MC}	1.0	*	*	*
Earthquake component combination factor, F_{ECC}	1.0	0.15	*	0.15
Total	1.0	0.15	0.23	0.28

Table 3: Seismic fragility of diesel fuel oil day tank

Factor	Median	β_R	β_U
Strength, F_S	1.71	0	0.13
Inelastic energy absorption, F_u	1.0	0	0
Qualification method, F_{OM}	1.0	0	0
Damping factor, F_δ	1.08	0	0.06
Modeling factor, F_M	1.0	0	0.1
Mode combination factor, F_{MC}	1.0	0.1	0
Earthquake component combination factor, F_{ECC}	1.0	0.18	0
Structure response factor, F_{RS}	1.0	0.15	0.23
Total factor (F_R)	1.85	0.25	0.29
$A_{R, SSE}$ (floor spectral acceleration)	1.62g		
A_R	2.99g		

In this example, the fragility curve for the sample tank is expressed in terms of floor spectral acceleration at a frequency of 22 Hz. That the floor spectral accelerations in the x and y directions is combined using the SRSS rule and the SRSS floor spectral acceleration due to SSE is 1.62 g. The median floor spectral acceleration capacity of the diesel fuel oil day tank is calculated using Equation (8) as 2.99g. The logarithmic standard deviation values are obtained as $\beta_R = 0.25$ and $\beta_U = 0.29$. The calculation is summarized in Table 3.

EXAMPLE II: SWING DIESEL GENERATOR BATTERY

This section presents an example for the development of a response-based fragility function for the battery of a sample swing diesel generator in a NPP in Taiwan. For equipment qualified by shake table test, Equation (12) is used to evaluate seismic fragility of the equipment for structural integrity and operational functionality. In this example, we assume that the equipment is tested to the IEEE requirements (IEEE, 1983), and the *TRS* envelops the *RRS* by 10% (IEEE 1983) across the entire frequency range of interest. The swing diesel generator battery is assumed qualified to the design floor response spectra at an elevation of 20m. In addition, it is required for testing functional qualification that seismic excitation generally has broad frequency content, and multi-frequency independent random inputs. Thus, it is assumed that the *TRS* is broad-banded such that the 10% over test requirement is at the peak of the *RRS*. At frequencies on either side of the peak, the over-test factor is likely to be greater. Thus,

$$TRS = 1.1 \cdot RRS \quad (17)$$

Since the *TRS* is expected to be broad-banded as discussed above, the clipping factor (C_T) for equipment capacity is unity, i.e., $C_T = 1$. The capacity increase factor (C_I) in Equation (10) is to account for the fact that it is unlikely that the peak in-structure spectral acceleration will line up with the lowest spectral acceleration capacity of the *TRS*. There are studies to quantify this conservatism. However, Reed and Kennedy (1994) provide a conservative estimate of the median factor of 1.1 and a value of 0.05 for uncertainty and variability in C_I .

The clipping factor (C_C) for narrow banded demand is a function of the bandwidth to central frequency ratio, B . The value of B is calculated using the raw (unbroadened) floor response spectra at El. 20 m following the procedure given in Reed and Kennedy (1994). The resulting value of C_C is 0.505. The demand reduction factor (D_R) is to account for the fact that using a single time series to generate seismic demand is conservative when compared with demand generated from multiple time histories. Reed and Kennedy (1994) suggest a median demand reduction factor of 0.92 with an uncertainty value of 0.04. Using the above values, we obtain a value of 2.606 for the ratio of TRS_C / RRS_C .

The factor F_D is to account for conservatism and variability in *TRS*. The median factor for equipment required to remain functional during earthquake and qualified by test is estimated to be 1.4 with a corresponding randomness variability value of 0.09 and uncertainty variability value of 0.22 (Reed and Kennedy, 1994).

The qualification method factor (F_{QM}) is calculated as 1.15 to account for the 15% increase of the enveloped floor response spectra generated from the RB seismic response analysis before the spectra were broadened.

The structural response factor, F_{RS} , of Table 2 can be used for the equipment located within the reactor building and was used herein for the sample equipment. The median floor spectral acceleration capacity of the swing diesel generator battery under functional failure is calculated using Equation (9) as 33.22 g. The logarithmic standard deviation values are obtained as $\beta_R = 0.157$ and $\beta_U = 0.387$. The calculation is summarized in Table 4.

Table 4: Seismic fragility of swing diesel generator battery

Factor	Median	β_R	β_U
Ratio of $TRSC$ to $RRSC$	2.606	-	-
Clipping factor for narrow-banded TRS , C_T	1.0*	0	0
Capacity increase factor, C_I	1.0*	0	0.05
Clipping factor for narrow-banded demand, C_C	1.0*	0	0.21
Demand reduction factor, D_R	1.0*	0	0.04
Broad frequency input spectrum device capacity factor, F_D	1.4	0.09	0.22
Qualification Method, F_{OM}	1.15	0	0
Structural response factor of the building, F_{RS}	1.0	0.15	0.23
Total factor	4.20	0.18	0.39
$A_{R,SSE}$ (peak floor spectral acceleration)	7.92g		
$A_R = F_R \cdot A_{R,SSE}$	33.22g		

CONCLUSION

In this paper, the methodology to develop response-based fragility curves for structural components and equipments is proposed. The proposed methodology is partly based on that of Reed and Kennedy (1994) except that the fragility curve of each component is expressed in terms of the structural response parameter that best estimates damage to the component. Response-based fragility curves is developed using the proposed methodology for sample structural components and equipments in a sample NPP in Taiwan. The use of structural response parameters in developing component fragility curves reduces the dispersion in the curves and enables the curves to be independent of seismic hazard and closely related to component capacity. The developed fragility curves have been used in the SPRA of the sample NPP presented in the companion paper entitled “Seismic probabilistic risk assessment of nuclear power plants using response-based fragility curves” (Shen *et al.* 2013).

REFERENCES

- Electric Power Research Institute (EPRI), (2002). *Seismic Fragility Application Guide*, TR-1002988. EPRI, CA, USA.
- IEEE (1983), “IEEE Standard for Qualifying Class 1E Equipment for Nuclear Power Generating Stations,” IEEE Std 323-1983, The Institute of Electrical and Electronic Engineers, Inc. New York, New York, 1983.
- Kennedy, R. P. and Ravindra, M. K., (1984). “Seismic fragilities for nuclear power plant risk studies”, *Nuclear Engineering and Design*, 79(1), 47-68.
- Huang, Y.-N., Whittaker, A. S. and Loco, N. (2011a). “A probabilistic seismic risk assessment procedure for nuclear power plants: (I) Methodology,” *Nuclear Engineering and Design*, 241(9), 3996-4003.
- Huang, Y.-N., Whittaker, A. S. and Luco, N. (2011b). “A probabilistic seismic risk assessment procedure for nuclear power plants: (II) Application,” *Nuclear Engineering and Design*, 241(9), 3985-3995.
- Reed, J. W. and Kennedy, R. P., (1994). *Methodology for Developing Seismic Fragilities*. TR-103959. Electric Power Research Institute, Palo Alto, California, USA.
- Shen, Y. -H., Chang, C. C., and Huang, Y. -N. (2013). “Seismic probabilistic risk assessment of nuclear power plants using response-based fragility curves,” *Proc., 22nd International Conference on Structural Mechanics in Reactor Technology*, San Francisco, California, USA.