Fragility Evaluation for a PRA Study

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Abstract

On the basis of a reliability analysis, it is possible to analytically generate the fragility curves for PRA studies. In this paper, using a realistic reinforced concrete containment as an example, a reliability analysis of the containment subjected to ground earthquake acceleration is performed and a fragility curve for PRA studies is constructed. The earthquake ground acceleration is represented by a segment of stationary Gaussian process with a zero mean and a Kanai-Tajimi spectrum. All the possible seismic hazards at the site represented by the hazard curve is also taken into consideration. The limit state of the structure is analytically defined and the corresponding limit state surface is used for the analysis.

1. Introduction

Probabilistic risk assessment (PRA) is becoming an important tool for the safety evaluation of nuclear power plants. Previous PRA studies such as WASH-1400 by the US NRC [1] mainly concentrated on internal events. Recently, the method has been extended to deal with external events,in particular, the seismic event. As a result of some recent PRA studies such as those carried out for the Zion and Indian Point nuclear power plants, respectively by Commonwealth Edison Co. [2] and the Power Authority of the State of New York [3], it was concluded that a seismic event could be a dominant contributor to the risk. The major steps in performing a seismic PRA study are as follows: (1) seismic hazard analysis; (2) response analysis and fragility determination; (3) plant system evaluation; (4) consequence evaluation. It is recognized that all of the above steps involve uncertainties.

For the evaluation of structural fragility, the approach used in industrial PRA studies is primarily based on subjective judgment. The main features in the approach are (1) an assumption of lognormal distributions for all variables; and (2) a multiplication scheme to predict a median value of the overall safety factors to be determined subjectively. Use of lognormal distributions for all the variables is purely for mathematical expedience. Furthermore, the subjective inputs and multiplication scheme do not appear to be a good combination, since the combination produces fragility curves which are quite sensitive to the subjective inputs. Consequently, fragility curves estimated by different groups of engineers may vary considerably, and final PRA results are indeed open to question. An alternate approach to the industrial PRA method is to evaluate the structural response and fragility analytically on the basis of probabilistic structural mechanics.

In recent years, a probability-based reliability analysis methodology for nuclear structures has been developed jointly at Columbia University and the Structural Analysis Division of Brookhaven National Laboratory (BNL); e.g., see Hwang et al [4]. An important feature of this methodology is that finite element analysis and random vibration theory have been incorporated into the reliability analysis. By utilizing this method, it is possible to evaluate the safety margins of nuclear structures under various static and dynamic loads and to generate the structural fragility curves for PRA studies. In this paper, such a reliability analysis method is performed on a reinforced concrete containment structure in order to construct its fragility curve.

2. Containment Description

The reinforced concrete containment structure, as shown in Fig. 1, represents a realistic containment in the US. The containment consists of a circular cylindrical wall with a hemispherical dome on the top. The dome-cylinder system is fixed at the base. The dimensions of the containment are also shown in Fig. 1. The containment wall is reinforced with hoop, meridional and diagonal rebars. The details of the rebar arrangement are found in Ref. 4. The mean values of the material properties are used in the analysis. The variations of the material properties will be included in a sensitivity study in the future. The minimum compressive strength of concrete at twenty-eight days is specified as 4000 psi. However, the mean value is estimated to be 6085.6 psi. The weight density of the concrete is taken to be 150 lb/ft^3. Young's modulus and Poisson's ratio are 3.6×10^6 psi and 0.2, respectively. No. 18 rebars are the main reinforcement used in containment structures. Hence, the statistics for No. 18 rebars are used to represent all other types of rebars. Young's modulus E_s and Poisson's ratio are taken to be 29.0×10^6 psi and 0.3, respectively. From the test data, the mean value of the yield strength f_y is estimated to be 71.1 ksi.

3. Finite Element Analysis

Finite element analysis is used to obtain the static structural responses and the dynamic characteristics of the structure such as the natural frequencies and the associated mode shapes, etc. The finite element utilized in this analysis is the shell element described in the SAP V computer code. A three-dimensional finite element model is used for the structural analysis of the containment. As shown in Ref. 4, the containment is divided into twenty layers. Except for the top layer of the dome, each layer has twenty-four elements such that the nodal points are taken at every 15° in the circumferential direction. This discretization requires a total of 481 nodes and 468 elements.

For the dynamic analysis of structures, modal analysis is employed. Using the model described, the first twenty natural frequencies and corresponding modes are evaluated. It is important to choose the most significantly participating modes for the reliability analysis. Thus, in this study, only the first and second pairs of bending modes are chosen for the analysis.

4. Probabilistic Representation of Loads

The dead load arises primarily from the weight of the containment wall. It may have small variations due to the weight density of concrete. In this analysis, the dead load is assumed to be deterministic and is equal to the design value, which is computed based on the weight density of reinforced concrete as 150 lb/ft^3.

Because several floors are connected to the containment structure, some live loads act on the containment at those locations where the floors are connected to the containment. The locations and design values of the corresponding live loads are as follows: 0.707 (k/f) at elevation 856', 3.0 (k'f) at 828.25', 0.94 (k/f) at 803.25', 1.02 (k/f) at 778' and 0.93 (k/f) at 775' with k/f = kip/ft. For the purpose of the present analysis, however, the live loads are also assumed to be deterministic and equal to the design values.

The earthquake ground acceleration is assumed to act only along the horizontal (x) direction. It is further assumed that the ground acceleration can be idealized as a segment of a stationary Gaussian process with mean zero and a Kanai-Tajimi spectrum. The Kanai-Tajimi spectrum has the following expression:

$$S_{ggxx}(\omega) = S_0 \frac{1 + 4\xi_g^2(\omega/\omega_g)^2}{[1 - (\omega/\omega_g)^2]^2 + 4\xi_g^2(\omega/\omega_g)^2} \qquad (1)$$

where the parameter S_0 represents the intensity of the earthquake, and ω_g and ξ_g are the dominant ground frequency and corresponding critical damping ratio, respectively. In this study, it is assumed that $\omega_g = 9\pi$ rad/sec and $\xi_g = 0.6$. The mean duration μ_{dE} of the earthquake acceleration is assumed to be 10 seconds.

The peak ground acceleration A_1, given an earthquake, is assumed to be $A_1 = P_g\sigma_g$ where P_g is the peak factor and taken to be 3.0 in this study. The standard deviation of the ground acceleration σ_g associated with the Kanai-Tajimi spectral density function is:

$$\sigma_g = \sqrt{\pi\omega_g\left(\frac{1}{2\xi_g} + 2\xi_g\right)}\ \sqrt{S_0} \qquad (2)$$

It is easy to show that, if the earthquake occurs in accordance with the Poisson law at a rate λ_E per year, the probability distribution $F_A(a)$ of the annual peak ground acceleration A is related to the probability distribution $F_{A_1}(a)$ of A_1 in the form $F_A(a) = \exp\{-\lambda_E[1 - F_{A_1}(a)]\}$. It then follows that $\lambda_E = \ell n\, F_A(a_0)$, if a_0 indicates the minimum peak ground acceleration for any ground shaking to be considered as an earthquake. Assuming that $F_A(a)$ is of the extreme distribution of Type II, $F_A(a) = \exp[-(a/u)^{-\alpha}]$ with $\alpha = 2.61$ and $u = 0.01$, one finds that $\lambda_E = 1.50 \times 10^{-2}$/year for $a_0 = 0.05g$. Finally, writing Z for $\sqrt{S_0}$, one obtains the probability distribution of Z as

$$F_Z(z) = 1 - (a/a_0)^{-\alpha} \qquad \text{for } z > a_0/\alpha_g \qquad (3)$$

where $\alpha_g = P_g(\omega_g)^{1/2}[1/(2\xi_g) + 2\xi_g]^{1/2}$. The maximum earthquake ground acceleration a_{max} which represents the largest earthquake possible to occur at a particular site, is chosen to be equal to 0.71g.

5. Limit State

A limit state essentially represents a state of undesirable structural behavior. For a particular structural system, it is possible that more than one limit state exists. Limit states must also be related to the response quantities obtainable from the selected structural analysis method, e.g., the finite element method adopted in this study. In this paper, the flexural limit state is used for the containment. This limit state is defined as follows: the state of structural response is considered to have reached the limit state if the maximum compressive strain at the extreme fiber of the cross-section is equal to 0.003, while the yielding of rebars is permitted. Based on this definition of the limit state and the ultimate strength theory of reinforced concrete, for each cross-section of a finite element, a limit state surface was constructed by Chang et al [5] in terms of the membrane stress and bending moment, which is taken about the center of the cross-section. A typical limit state surface is shown in Fig. 2. In this figure, point "a" is determined from the stress state of uniform compression and point "e" from uniform tension. Points "c" and "c'" are the so-called "balanced points" at which a concrete compression strain of 0.003 and a steel tension strain of f_y/E_s are reached simultaneously. Furthermore, lines abc and ab'c' in Fig. 2 represent compression failure and lines cde and c'd'e represent tension failure.

6. Reliability Analysis and Fragility Curves

Analytically, the eight straight lines of the limit state surface shown in Fig. 2 are expressed as follows:

$$R_j - \{A_j\}^T\{\tau^{(e)}\} = 0 \qquad j=1,2,\ldots,8 \tag{4}$$

where $\{\tau^{(e)}\} = \{\tau^{(e)}\}_0 + \{\tau^{(e)}\}_d$ is the element stress vector, and R_j and $\{A_j\}$ are constants and the constant vectors, respectively. Vector $\{\tau^{(e)}\}_0$ is the stress vector due to dead and live loads and is time-invariant and deterministic. Vector $\{\tau^{(e)}\}_d$ is the stress vector due to earthquake acceleration and it can be computed as $\{\tau^{(e)}\}_d = Z[c^{(3)}]\{v_0\}$ with $[c^{(e)}] = [B^{(e)}][\phi^{(e)}][L_q]$. In this expression, $[B^{(e)}]$ and $[\phi^{(e)}]$ are such that $\{\tau^{(e)}\} = [B^{(e)}]\{u^{(e)}\}$ and $\{u^{(e)}\} = [\phi^{(e)}]\{q\}$ with $\{q\}$ being the generalized coordinate vector, respectively. Vector $\{v_0\}$ is obtained from the linear transformation $\{q_0\} = [L_q]\{v_0\}$ such that the covariance matrix $[V_{v_0v_0}]$ of $\{v_0(t)\}$ becomes $[I_m]$ = an mxm identity matrix (m = number of modes considered). Vector $\{q_0\}$ is the generalized coordinate vector when $Z = \sqrt{S_0} = 1\sqrt{in^2/sec^3}$. One then obtains from eq. (4)

$$r_j^{(e)} - Z\{n_j^{(e)}\}^T\{v_0\} = 0 \tag{5}$$

where $\{n_j^{(e)}\} = \{\bar{A}_j^{(e)}\}/|\bar{A}_j^{(e)}|$ with $\{\bar{A}_j^{(e)}\}^T = \{A_j\}^T c^{(e)}$ and $r_j^{(e)} = (R_j - \{A_j\}^T\{\tau^{(3)}\}_0)/|\bar{A}_j^{(e)}|$. Let $X_{mj}^{(e)}$ be max $|\{n_j^{(e)}\}^T\{V_0\}|$ in $0 < t < \mu_{dE}$. The probability distribution of $X_{mj}^{(e)}$ can be given in approximation by

$$F_{X_{mj}^{(e)}}(x) = \exp\{-\nu_{j0}^{(e)}\mu_{dE}\exp(-1/2\ x^2)\} \tag{6}$$

In eq. (6), $\nu_{j0}^{(e)} = \frac{1}{2\pi}\sqrt{\sum_{a=1}^{m}\sum_{b=1}^{m} n_{aj} n_{bj} E[\dot{v}_{0a}\dot{v}_{0b}]}$ where $x > \sqrt{2\ln\nu_{j0}^{(e)}\mu_{dE}}$ and $n_{aj}^{(e)}$ is the a-component of $\{n_j^{(e)}\}$ and $E[\dot{v}_{0a}\dot{v}_{0b}]$ is the a-b component of the covariance matrix $[V_{\dot{v}_0\dot{v}_0}]$ of $\{\dot{v}_0(t)\}$. The limit state probability with respect to line j, $P_j(e)$, conditional to the occurrence of an earthquake is then obtained as

$$P_j^{(e)} = \Pr\{r_j^{(e)} - ZX_{mj}^{(e)} < 0\} . \tag{7}$$

The fragility is defined here as the conditional limit state probability, given a peak ground acceleration A_1. Hence, referring to eqs. (6) and (7), the fragility is determined in approximation as:

$$P(A_1) = \max_{j \text{ and } (e)} \{1 - \exp[-\nu_{j0}^{(e)}\mu_{dE} \exp\{-1/2(\frac{\alpha_g\gamma_j^{(e)}}{A_1})^2\}]\} \tag{8}$$

where the maximum value with respect to j and (e) is taken. The fragility curve as a function of A_1 measured in g is presented in Fig. 3. Since all the data used in the analysis is taken to be the best estimate values (or mean values), this fragility curve may be interpreted as the mean fragility curve.

7. References

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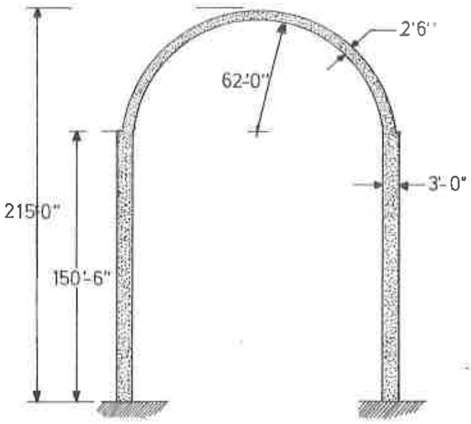
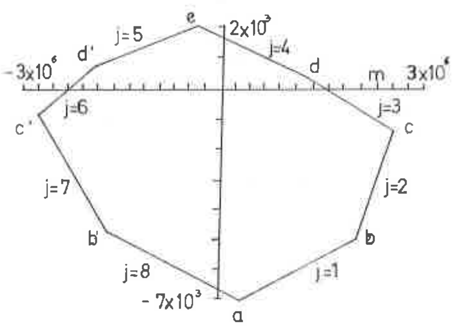
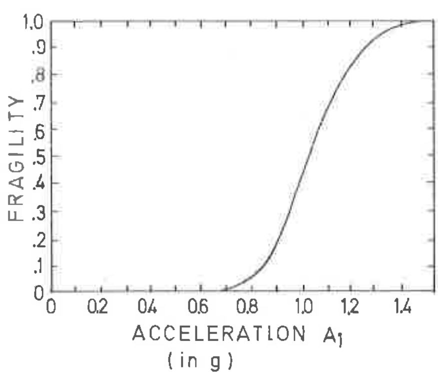


1. Containment Structure



2. Limit State Surface



3. Fragility Curve