*Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5*

# Seismic safety against ultimate failure of nuclear containment structures

J. Mizuno
*Nuclear Structure Design Division, Kajima Corporation, Tokyo, Japan*

A. H.-S. Ang
*University of California, Irvine, Calif., USA*

ABSTRACT: The significance of seismic events on the overall safety of nuclear power plants has been recognized by recent seismic safety studies. The safety of a complete structure, including nuclear structures, is determined by its reliability as a system, i.e. not limited to the safety of any one of its structural element, but rather of the entire structure. Reliable evaluation of seismic safety of structural systems, however, is still needed. To this end, the seismic structural reliability is examined further for reinforced concrete nuclear structures, specifically for a reinforced concrete (RC) containment structure. The system reliability against ultimate failure is evaluated on the basis of the stable configuration approach (SCA) and the three-dimensional finite element method (FEM) of stress analysis, considering both flexural and shear failure modes.

## 1 INTRODUCTION

The safety of a complete structure, including nuclear structures, is determined by its reliability as a system, i.e. not limited to the safety of any one of its structural element, but rather of the entire structure.

The ultimate failure (collapse) of a structure or structural element can be reasonably modeled as a brittle mode of failure in the sense that upon failure the load-carrying capacity drops to zero. The collapse of a nuclear containment structure can be similarly modeled. The ultimate failure of a typical PWR containment structure subjected to earthquake ground motion is evaluated on the basis of SCA, which has previously been shown to be effective for such brittle systems (Quek and Ang ,1991; Mizuno and Ang, 1991).

## 2 STABLE CONFIGURATION APPROACH

A stable configuration is defined as any damaged state of the structure that can still carry the applied loads. In general, the collapse probability of a structure is,

$$E = \bigcap_{i=1}^{n} \overline{C}_i \qquad \ldots(1)$$

where $C_i$ is the i-th stable configuration, and n is the number of all possible stable configurations. The complementary event of $C_i$ is the event of further damage to the current configuration. Each configuration can be expressed as,

$$\overline{C}_i = \bigcup_j B_{ji} \qquad \ldots(2)$$

which is the union of member failure events. For the intersection of events corresponding to the initial configuration and all the configurations with only one failed element can be expressed as (Quek and Ang, 1991);

$$P(\bigcap_{i=0}^{k} \overline{C}_i) = \sum_{i=0}^{k} P(\overline{C}_i) - \sum_{j=1}^{k} P(\overline{C}_0 \cup \overline{C}_j) \qquad \ldots(3)$$

in which $\overline{C}_0$ is the complement of the initial configuration (no damage state), and k is the number of configurations considered. The probabilities for unions of events are computed by the second-order bounds, and the geometric average of the bounds is used for calculating the intersection of events.

The probabilities of the intersections of events corresponding to configurations with more than one failed element are generally much less significant.

## 3 MODELING OF RC CONTAINMENT STRUCTURE

A RC containment structure, and associated material properties are shown in Fig. 1 and Table 1, following Shinozuka et al,1984.

Table. 1 Material Properties

| | |
|---|---|
| **Concrete** | |
| Compressive Strength | Mean 42.0MPa (428.1 kgf/cm²)<br>COV 0.106 |
| Young's Modulus | 2.48 x $10^4$ MPa (253.5 tonf/cm²) |
| Weight / unit volume | 2.35 x $10^4$ N/m³(2.40 tonf/m³) |
| Poisson's Ratio | 0.2 |
| **Reinforcement** | |
| Yield Strength | Mean 490MPa (5000 kgf/cm²)<br>COV 0.036 |
| Young's Modulus | 2.0 x $10^5$ MPa (2040tonf/cm²) |
| Poisson's Ratio | 0.3 |

For simplicity, the initial configuration of the containment structure is assumed to be axisymmetric with orthogonal reinforcements and fixed at the base.

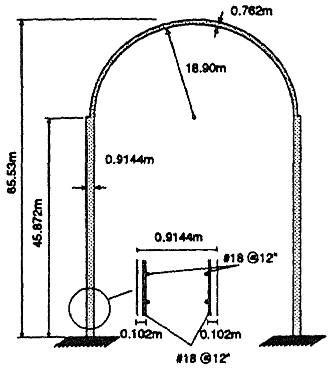


Figure 1 Reinforced concrete containment structure

The stable configurations are composed of three-dimensional FEM models with linear material constants, with the failed elements removed, and the stress redistribution taken into account.

The four node shell element, QUAD4 in the NASTRAN code, is used for modeling the RC containment. The FEM model and the shell element are shown in Fig.2. The full FEM model for the stress analysis of the various stable configurations must be used to evaluate the stress states of the asymmetric configurations.

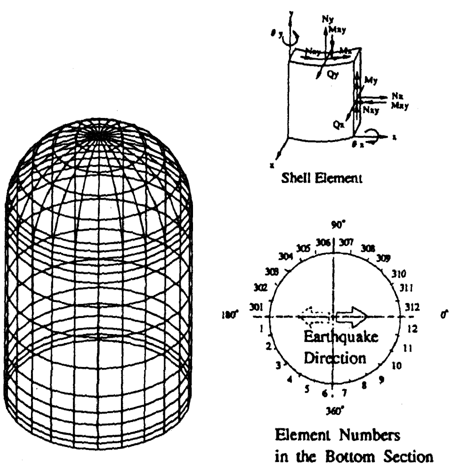


Figure 2 Finite element model

## 4 LIMIT STATE FOR ELEMENT FAILURE

Under severe ground motion there may be two major ultimate collapse pattern of the containment structure, i.e. flexural compressive failure and shear failure.

For the collapse of the structure due to flexural compressive failure, the critical portions of the structure are the elements in the vicinity of 0° or 180° in the bottom section in Fig. 2, where the compressive stress becomes the highest in the structure. For shear failure mode of the structure, the shear stress becomes the highest in the area of 90° and 270° at the bottom section.

In the critical section of the containment wall for flexural compressive failure, the entire section is very likely stressed in compression. The principal compressive stress in the critical section may be approximated by the vertical stress components of the shell element, $N_y$, since the stress states in the most stressed section in compression are close to uniaxial stress state under seismic loads. The performance function for flexural compressive failure may, then, be given as follows,

$$g_f(x) = fc - ( N_y / t + M_x / z_e) \qquad \ldots(4)$$

where,

- fc = 0.85 fc' ;
- fc' = cylinder compressive strength of concrete;
- $N_y$ = maximum normal stress of shell element per unit width in the local y-axis ;
- $M_x$ = maximum bending moment in the local x-axis per unit width;
- t = thickness of shell element;
- $z_e$ = effective sectional coefficient with reinforcement considered.

The performance function for shear failure may be expressed as,

$$g_s(x) = \tau_u - N_{xy} / t \qquad \ldots(5)$$

where $\tau_u$ is the shear strength of reinforced concrete wall, and $N_{xy}$ is the shear stress of an element per unit width.

In general, $\tau_u$ depends largely on concrete strength and reinforcement ratio as well as multi-dimensional stress states in a section. However, for heavily reinforced RC containment wall, the shear strength, on the basis of extensive experimental study of shear walls of nuclear structures, may be expressed as (Ogaki el, 1978),

$$\tau_u = 1.753 \sqrt{fc'} \quad (\text{in MPa}) \qquad \ldots(6)$$

## 5 MAXIMUM STRESS RESPONSE

The ground motion is modeled as a zero-mean stationary Gaussian process with a Kanai-Tajimi spectrum. The predominant frequency of the soil, $\omega_g$ and the soil damping factor, $\zeta_g$, are assumed to be $9\pi$ and 0.6, respectively. The duration of motion is assumed to be 10 seconds.

The stress responses are evaluated on the basis of linear random vibration. Given that the ground motion is a stationary Gaussian process, the variance of the stresses in the i-th element can be expressed as,

$$\sigma_{N\theta}^{i\,2} = \sum_j \sum_k r_j\, r_k\, \phi_{Nj}^{i}\, \phi_{Nk}^{i} \int_{-\infty}^{\infty} H_j(\omega)\, H_k^{*}(\omega)\, S_a(\omega)\, d\omega; \qquad \theta = x, y, xy \quad \ldots(7)$$

in which $\phi_{Nj}^{i}$ is the component of the j-th modal stress vector, $H_j(\omega)$ is the frequency response function of the j-th mode, $r_j$ is the participation coefficient, $S_a(\omega)$ is the power spectral density function of the earthquake.

Assuming that the maximum responses follow the Poisson occurrence law, the mean maximum stress response and its standard deviation can be expressed as;

$$\overline{N}_{\theta\,m}^{i} = \sigma_{N\theta}^{i} \sqrt{2 \ln ( \nu_{N\theta}^{i} t_d )} + 0.577215\, \sigma_{N\theta}^{i} \;/\; \sqrt{2 \ln ( \nu_{N\theta}^{i} t_d )} \quad \ldots(8)$$

$$\sigma_{N\theta\,m}^{i} = \frac{\pi}{\sqrt{6}} \frac{\sigma_{N\theta}^{i}}{\sqrt{2 \ln ( \nu_{N\theta}^{i} t_d )}} \quad \ldots(9)$$

where $\sigma_{N\theta}^{i}$ is the rms stress response evaluated by Eq. 2, using the dominant vibration modes; and $\nu_{N\theta}^{i}$ is the mean zero crossing rate of the stress. In this study, thirty vibration modes for each configuration are used to evaluate the maximum stresses.

## 6 ELEMENT FAILURE PROBABILITIES AND SELECTED STABLE CONFIGURATIONS

To identify the dominant stable configurations, first element failure probabilities are evaluated at the 2 SSE ( the peak ground acceleration is assumed to be 0.8g). Assuming that both the maximum stress responses and the concrete strength are lognormal variates with the mean and standard deviation evaluated by Eq. 8 and 9, the element failure probability may be evaluated by the first-order second-moment method. The computed probabilities of the critical elements are listed in Table 3.

The most critical elements for flexural compressive failure are 12 and 312, 1 and 301 respectively for each earthquake direction. For shear failure, the critical elements are 6, 7, 306 and 307. The dominant configurations with one element failed, are selected as those listed in Table 3. The contribution of the stable configuration with high probability of failure compared to that of the initial configuration is insignificant to the ultimate failure probability of a structural system, given by Eq. 1.

Since the critical elements for two collapse modes are well separated, the collapse probability of the structure may be expressed as,

$$P(E) = P(\bigcap_i {}_f\overline{C}_i\ ) + P(\bigcap_j {}_s\overline{C}_j\ ) \quad \ldots(10)$$

where,

${}_fC_i$ = stable configuration in flexure;

sCi = stable configuration in shear.

For the computation of the union and intersection of events, the correlations between element failures can be explicitly taken into account on the basis of the first-order second-moment method. Since concrete strength of containment wall at the same height may be highly correlated due to the same casting sequence, the correlation coefficient for concrete strength between elements is assumed to be 0.9.

Table 3 Element failure probabilities and selected stable configurations with one element failed

| Event | Failed Element | $P(B_{i10})$ | Corresopnding Configuration | Failure Mode |
|---|---|---|---|---|
| ${}_fB_{110}$ | 12 | $4.11\times10^{-5}$ | ${}_fC_1$ | Flexual Compressive Failure |
| ${}_fB_{210}$ | 312 | $4.11\times10^{-5}$ | ${}_fC_2$ | |
| ${}_fB_{310}$ | 11 | $1.02\times10^{-5}$ | ${}_fC_3$ | |
| ${}_fB_{410}$ | 311 | $1.02\times10^{-5}$ | ${}_fC_4$ | |
| ${}_fB_{510}$ | 10 | $3.83\times10^{-7}$ | ${}_fC_5$ | |
| ${}_fB_{610}$ | 310 | $3.83\times10^{-7}$ | ${}_fC_6$ | |
| ${}_sB_{110}$ | 6 | $1.46\times10^{-5}$ | ${}_sC_1$ | Shear Failure |
| ${}_sB_{210}$ | 7 | $1.46\times10^{-5}$ | ${}_sC_2$ | |
| ${}_sB_{310}$ | 306 | $1.46\times10^{-5}$ | ${}_sC_3$ | |
| ${}_sB_{410}$ | 307 | $1.46\times10^{-5}$ | ${}_sC_4$ | |
| ${}_sB_{510}$ | 5 | $2.77\times10^{-6}$ | ${}_sC_5$ | |
| ${}_sB_{610}$ | 8 | $2.77\times10^{-6}$ | ${}_sC_6$ | |
| ${}_sB_{710}$ | 305 | $2.77\times10^{-6}$ | ${}_sC_7$ | |
| ${}_sB_{810}$ | 308 | $2.77\times10^{-6}$ | ${}_sC_8$ | |

*For flexural compressive failure, failure events only for one direction are listed.

The collapse probabilities of each failure mode are evaluated as follows,

$$P( {}_f\overline{C}_{110} \cap \cdots \cap {}_f\overline{C}_{610} ) = 4.51 \times 10^{-5}$$

$$P( {}_s\overline{C}_{110} \cap \cdots \cap {}_s\overline{C}_{810} ) = 3.53 \times 10^{-5}$$

The probability of intersection, including stable configurations with one failed elements, are also computed. However, the difference between the results given by the initial configuration and by including more configurations is minimal. It may be noted that if the element failure probabilities are highly correlated, the initial configuration gives a reliable estimate of the collapse probability of a structure.

## 7 FRAGILITY FUNCTION AND ULTIMATE FAILURE PROBABILITY

The collapse probability of the structure for both flexural compressive failure and shear failure modes are assessed for different levels of peak ground acceleration, leading to the fragility function shown in Fig. 3. The probability of initial yielding is also evaluated, of which the critical elements are those on the opposite side of the critical elements for flexural compressive failure. The initial yielding of reinforcement between elements is assumed to be statistically independent whereas earthquake induced stresses are perfectly correlated among the elements.

The annual failure probability $P_E$ may be given by;

$$P_E = \int_{a_0}^{a_{max}} F_E(a)\, f_A(a)\, da \qquad ...(11)$$

in which $F_E(a)$ is the fragility function as a function of peak ground acceleration, a; and $f_A(a)$, is the probability density function of the peak ground acceleration, i.e. the derivative of a hazard function. The hazard function in Fig.4 ( Shinozuka et al,1984). is used in evaluating the annual failure probability given by Eq. 11.

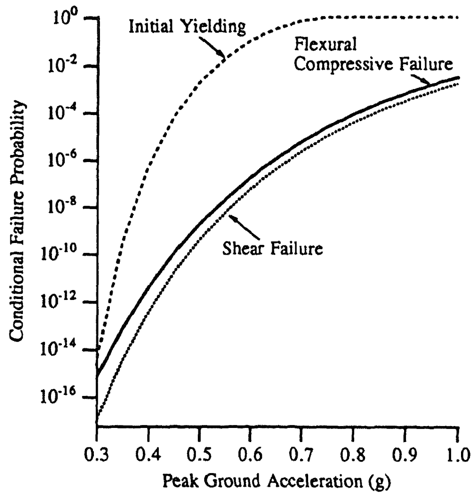


Figure 3 Seismic fragility function

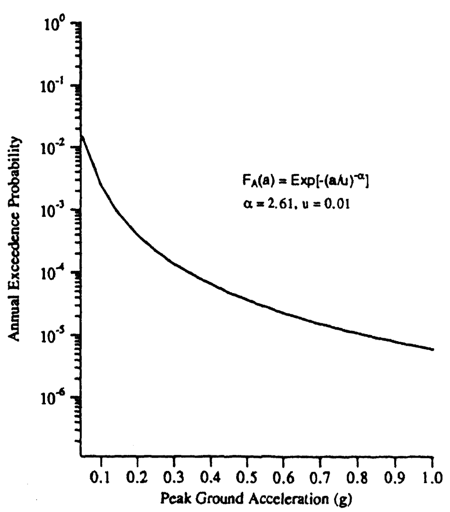


Figure 4 Seismic hazard function

The annual probabilities of collapse and initial yielding are summarized in Table 4. The maximum peak ground acceleration considered in the integration, $a_{max}$, is 1.0g. The probability of flexural compression failure of the structure is slightly larger than that of shear failure. In general, which failure mode governs the ultimate system failure of a structure may depend on several key parameters for flexural and shear capacity such as concrete strength, reinforcement ratio, and the proportion of a cylindrical wall ( ratio of height to diameter ).

Table 4 Structural system failure probability of RC containment

| | | System Failure Probabilities per year |
|---|---|---|
| Ultimate System Failure | Flexural Failure | $5.68 \times 10^{-9}$ |
| | Shear Failure | $2.57 \times 10^{-9}$ |
| | Total | $8.25 \times 10^{-9}$ |
| Initial Yielding | | $1.74 \times 10^{-5}$ |

## 8 CONCLUDING REMARKS

The seismic structural system reliability of a RC containment structure is evaluated on the basis of the SCA concept and FEM. The system collapse probability for the shear failure is explicitly evaluated as well as for the flexural compressive failure of a RC containment structure. The results show that both failure modes are equally significant in terms of annual collapse probability under seismic loading. The failure probability is of the order of $10^{-9}$, whereas the corresponding probability of first yielding of the reinforcement is of the order of $10^{-5}$. The difference between the probabilities of initial yielding and ultimate system failure may be interpreted as the reserved safety margin of the structure.

## REFERENCES

Mizuno, J. and A.H-S. , Ang,"Seismic Structural System Reliability of RC containment structures," SMiRT11, M09(J)/1, 1991,pp 169-174

Ogagi, Z. et al, "Experiment of 1/8 PC containment under Lateral Loading," Proc. of AIJ, 1978, pp 1851-1860 (in Japanese)

Quek, S-T. and Ang, A.H-S., "Reliability Analysis of Structural Systems by Stable Configuration," J. of Structural Engineering,, ASCE, Vol.116, Oct.1990, pp2654-2670

Shinozuka, M, Hwang,H. and Reich,M., "Reliability Assessment of Reinforced Concrete Containment structures, "J.of Nuclear Engineering and Design, North Holland, Vol.80, 1984, pp247-267