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RELIABILITY ANALYSIS OF RC CONTAINMENT STRUCTURES UNDER COMBINED LOADS

H.Hwang*, M. Reich* and S. Kagami**

* Brookhaven National Laboratory
Upton, NY 11973, USA
** Taisei Corporation
P.O. Box 4001, Shinjuku-ku
Tokyo, 160-91, Japan

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ABSTRACT

This paper discusses a reliability analysis method and load combination design criteria for reinforced concrete containment structures under combined loads. The probability based reliability analysis method is briefly described. For load combination design criteria, derivations of the load factors for accidental pressure due to a design basis accident and safe shutdown earthquake (SSE) for three target limit state probabilities are presented.

KEYWORDS

Concrete(Reinforced); Containment Structures; Design; Earthquake; Limit States; Loads; Reliability; Safety

1. INTRODUCTION

The safety of nuclear power plant structures is of primary concern to the regulatory agencies, the nuclear industry and the general public because of the serious socioeconomic consequences that could result from structural failures. To ensure the structural safety, nuclear structures must be designed to withstand all kinds of loads and load combinations that may be expected to occur during their lifetime. These loads include various static and dynamic loads, which are caused by operational, environmental and accidental conditions. It is recognized that the loads involve random and other uncertainties in nature. Similarly, the structural resistance also cannot be determined without uncertainties. The traditional methods of structural design attempt to account for the inevitable variability through the use of safety factors or load and resistance factors. These factors are specified in various codes such as ASME, ACI, AISC, etc., and the NRC Standard Review Plan (SRP). However, the subjective manner by which these safety factors have been determined may result in an unknown and nonuniform reliability. In view of randomness and uncertainty in loads, structural resistance etc., a probabilistic approach for assessment of structural safety and for development of load combination design criteria is a rational choice.

This paper discusses a reliability analysis method and load combination design criteria for reinforced concrete containment structures under combined loads. The probability-based reliability analysis method is briefly described. For load combinationdesign criteria, derivations of the load factors for accidental pressure due to a design basis accident and safe shutdown earthquake (SSE) for three target limit state probabilities are presented.

2. SAFETY ASSESSMENT OF CONTAINMENT STRUCTURES

For the safety evaluation of concrete containment structures under various static and dynamic loads, a probability-based reliability analysis method has been developed.[1,2] An important feature of this method is that finite element analysis and random vibration theory have been incorporated into the reliability analysis. In the method, an appropriate probabilistic model is established for each load. For example, accidental pressure is idealized as a rectangular pulse with random intensity and duration and is assumed to occur according to the Poisson arrival law. Earthquake ground acceleration is represented by a segment of a stationary Gaussian process with a zero mean and Kanai-Tajimi spectrum. Furthermore, all possible seismic hazards at a site, which is represented by a hazard curve, are also included in the analysis. The limit state of the structure is then analytically defined and the corresponding limit state surface is established. Finally, limit state probabilities for various load combinations are evaluated.

Currently, the reliability analysis method for concrete containments subjected to dead load, live load, accidental pressure, tornado, SRV load and ground earthquake acceleration has been established. This reliability analysis method has also been applied to selected existing containment structures in order to assess their safety margins under various load combinations.[1,3]

3. LOAD COMBINATION DESIGN CRITERIA

In principle, the reliability analysis method described above could be utilized directly in structural design. However, the probabilistic method requires expert judgement on the probabilistic models of loads and resistance, and on the target limit state probability etc., thus, it is not suitable for the routine design of nuclear structures.[4] Load combination criteria, which are in a deterministic format and yet reflect the probabilistic nature of the design parameters, are more appropriate for routine design purposes. The procedure for developing probability-based load combination criteria for the design of containments is as follows:[5]

1. Select an appropriate load combination format. (e.g., LRFD format)
2. Establish representative structures.
3. Select a target limit state probability.
4. Assign initial values for all parameters (load factors etc.) associated with the selected load combination format.
5. Design each representative structure.
6. Determine the limit state probability of each representative structure.
7. Compute the objective function measuring the difference between the target limit state probability and the computed limit state probability.

8. Determine a new set of parameters (load factors) along the direction of maximum descent with respect to the objective function.
9. Repeat steps 5 to 8 until a set of parameters that minimize the objective function is found.

4. LOAD FACTORS FOR ACCIDENTAL PRESSURE AND SAFE SHUTDOWN EARTHQUAKE

The derivation of the load factors for the accidental pressure due to a design basis accident (DBA) and safe shutdown earthquake (SSE) for three target limit state probabilities is discussed below.

4.1 Load Combination Format

The development of probability-based load combinations for designing nuclear structures requires the selection of an appropriate load combination format. Several different formats have been proposed.[4] The format that has been selected for this study is the "load and resistance factor design (LRFD)" format. The LRFD format is simple enough to be used in routine design while offering sufficient flexibility to achieve consistent reliabilities in various design situations.

If three loads, i.e., dead load, earthquake and accidental pressure are considered, the load combinations in the LRFD format are expressed as follows:

$$0.9D + \gamma_p P_a \leqslant \phi_i R_i \quad (1)$$

$$1.2D + \gamma_{ES} E_{ss} \leqslant \phi_i R_i \quad (2a)$$

$$0.9D - \gamma_{ES} E_{ss} \leqslant \phi_i R_i \quad (2b)$$

where

D = load effect due to design dead load
P_a = load effect due to design pressure
E_{SS} = load effect due to safe shutdown earthquake
γ_p = load factor for accidental pressure
γ_{ES} = load factor for safe shutdown earthquake
ϕ_i = resistance factor for the i-th limit state under consideration
R_i = nominal structural resistance for the i-th limit state under consideration

Dead load factor and resistance factor are preset to simplify the optimization. The mean value of the dead load is approximately equal to its nominal value and its variability is quite small. A dead load factor of 1.2 (or 0.9 when the dead load has a stablizing effect) has been found to be more than adequate to account for uncertainty in dead load.[4,6] Thus, in Eqs. 1 and 2, the dead load factor is preset to be 1.2 or 0.9. A proposed set of resistance factors for concrete design that are consistent with the A58 load requirements has been derived.[7] For axial tension or flexure with axial tension, $\phi = 0.85$. For axial compression or flexure with axial compression, $\phi = 0.65$. The ϕ value is increased linearly from 0.65 to 0.85 as axial compression decreases from 0.10 $f'_{cn} A_g$ to zero, where f'_{cn} is the specified concrete compressive strength and A_g is the area of cross section. These ϕ values will be used for this study.

4.2 Selection of Representative Containment Structures

An important requirement for codified structural design is that all the structures designed according to a code should meet the code performance objectives which are expressed in probabilistic terms. In order to test if this requirement is satisfied, a set of representative (sample) structures must be selected for evaluating the code. In this study, representative containments are determined from examining the inventory of PWR reinforced concrete containments in the United States. The ranges of the design parameters such as geometries, material strengths, and design loads are determined as shown in Table 1. For each design parameter, one, two or four representative values are selected to represent its range. These representative values are also listed in Table 1. The general PWR containment characteristics identified in Table 1 can be used, along with a Latin hypercube sampling technique, to construct the sample containments. A sample containment is identified by a sample vector, which consists of one of the representative values of each design parameter. Four sample containments thus selected are shown in Table 2. With the design variables in Table 2 specified, only one design variable still needs to be determined, that is, the required reinforcement.

Table 1. Design Parameters of PWR Reinforced Concrete Containment.

Design Parameters	Design Range	Recommended Value
inside radius	18.3 m to 22.86 m	18.3 m, 21.34 m
dome rise ratio	1.0	1.0
cylindrical height	44.2 m to 48.77 m	45.72 m
cylindrical wall thickness	1.07 m to 1.52 m	1.07 m, 1.37 m
dome wall thickness	0.76 m to 1.07 m	0.76 m, 1.07 m
concrete compressive strength	20.7 MPa to 34.5 MPa	27.6 MPa, 34.5 MPa
yield strength of steel rebars	414 MPa	414 MPa
accidental pressure	0.28 MPa to 0.41 MPa	0.29,0.32,0.36, 0.39 MPa
safe shutdown earthquake	0.10 g to 0.75 g	0.17 g, 0.25 g, 0.32 g, 0.50 g

4.3 Limit State

A limit state represents a state of undesirable structural behavior. In general, a limit state is defined from the actual structural behavior under loads. For a particular structural system, it is probable that more than one limit state may be considered. In this study, the limit state for containments is defined according to the ultimate strength theory of reinforced concrete. It is assumed that the containment can be detailed to prevent failures at local stress concentrations such as penetrations, equipment hatches, etc., and can be stiffened to prevent local buckling. Thus, the limit state can be defined by membrane stresses and bending moments in the containment wall. It is described as follows: at any time during the service life of the structure, the state of structural response is considered to have reached the limit state if a maximum concrete compressive strain at the extreme fiber of the cross-section is equal to 0.003, while the yielding of rebars is permitted. Based on the above definition of the limit state and the theory of reinforced concrete, for each cross-section of a finite element, a limit state surface can be constructed in terms of the membrane stress and bending moment, which is taken about the center of the cross-section.[3,8]

4.4 Design of Containment Structures

Each sample containment as shown in Table 2, is assumed to be fixed at the base and has to be designed according to the proposed load combination with trial load and resistance factors, design loads and nominal resistance. For design loads and nominal resistance, the current values specified in codes are generally used. However, certain modifications may be necessary in order to put the design values on a probabilistic basis.

For the structural analysis of containments, three-dimensional finite element models are used. The finite element utilized in the analysis is the shell element as described in the SAP V computer code. The element stress resultants for dead load and accidental pressure are obtained from static analysis. For seismic analysis, the response spectrum analysis method is employed. The horizontal and vertical response spectra used in this study are those specified in the Regulatory Guide 1.60. The damping ratio is taken to be 7 percent of criterial for SSE as specified in the Regulatory Guide 1.61. The square root of the sum of squares (SRSS) method is used to combine the

Table 2. PWR Reinforced Concrete Containment Samples.

Design parameters	Sample 1	Sample 2	Sample 3	Sample 4
inside radius	21.34 m	18.3 m	18.3 m	21.34 m
dome rise ratio	1.0	1.0	1.0	1.0
cylindrical height	45.72 m	45.72 m	45.72 m	45.72 m
cylindrical wall thickness	1.37 m	1.07 m	1.37 m	1.07 m
dome wall thickness	1.07 m	0.76 m	1.07 m	0.76 m
concrete compressive strength (MPa)	27.6	27.6	34.5	34.5
steel yield strength (MPa)	414	414	414	414
dead load (kN/m^3)	[illegible]	23.55	23.55	23.55
accidental pressure (MPa)	0.32	0.29	0.36	0.39
safe shutdown earthquake (g)	0.17	0.32	0.50	0.25
soil	Rock	Deep Cohesionless	Rock	Deep Cohesionless
earthquake duration (sec)	10	20	20	10

responses in three directions. On the basis of the ultimate strength design of reinforced concrete, the minimum required rebar area is determined. Designers usually provide rebar area ' ryer than the minimum requirement. In this study, however, the minimum required rebar area will be used in design and reliability analysis.

4.5 Probabilistic Models for Loads and Material Strength

Various static and dynamic loads act on a containment structure during its lifetime. Since the loads intrinsically involve random and other uncertainties, an appropriate probabilistic model for each load must be established in order to perform reliability analysis.

4.5.1 Dead Load The dead load primarily arises from the weight of the containment wall. There is some uncertainty as to the actual magnitude of the dead load. However, the large variabilities in earthquake and accidental pressure tend to overshadow the variability in dead load. As a result its effect on the limit state probabilities is minor. Thus, for the purpose of this analysis, dead load is assumed to be deterministic and is equal to the design value.

4.5.2 Accidental Pressure The accidental pressure is considered as a quasi-static load that is uniformly distributed on the containment wall. The accidental pressure is idealized as a rectangular pulse that occurs in accordance with the Poisson law during the containment life. Under these assumptions, three parameters are required to model the accidental pressure: the mean occurrence rate, the mean duration, and the intensity P, intensity P is considered as a random variable. In this study, the mean occurrence rate and the mean duration are taken to be 1.68×10^{-3} per year and 1200 sec, respectively. The intensity is assumed to be normally distributed with a mean over design value of 0.9 and coefficient of variation of 0.12.[5]

4.5.3 Earthquake Ground Acceleration The seismic hazard at a site of a nuclear power plant is described by a seismic hazard curve. In this study, the probability distribution $F_A(a)$ of the annual peak ground acceleration A is assumed to be the Type II extreme value distribution,[9]

$$F_A(a) = \exp[-(a/u)^{-\alpha}] \tag{3}$$

where α and μ are two parameters to be determined. The value of α for the U.S. is estimated to be 2.7.[5] The parameter μ is computed based on this α value and the assumption that the annual probability of exceeding the safe shutdown earthquake at a site is 4×10^{-4} per year.[10]

The lower and upper bounds of peak ground acceleration are required in the analysis. The lower bound, a_0, indicates the minimum peak ground acceleration for any ground shaking to be considered as an earthquake. a_0 is assumed to be 0.05 g. The upper bound, a_{max}, represents the largest earthquake possible at a site. In this paper, a_{max} is chosen to be $2a_{SSE}$.

Even though the structures are designed for three components of an earthquake, for reliability analysis the earthquake ground acceleration is assumed to act only along the global x direction. This simplification is made since the reliability analysis results from both assumptions are almost the same because of the symmetry of the containment structures. The earthqquake ground acceleration on the condition that an earthquake occurs, is idealized as a segment of a zero-mean stationary Gaussian process, described in the frequency domain by a Kanai-Tajimi power spectral density.

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

where the parameter S_0 is a random varible and represents the intensity of an earthquake. The distribution of S_0 can be determined as shown in Ref. 5. Parameters ω_g and ζ_g are the dominant ground frequency and the critical damping, respectively, which depend on the site soil conditions. For rack and deep cohensionless soil conditions, ωg, is taken to be 8π rad/sec and 5π rad/sec respectively. ζg is taken to be 0.6 for both soil conditions.[9] The mean duration of the stationary phase of the earthquake acceleration is assumed to be 10 or 20 seconds in this study.

4.5.4 Material Strength In the reliability analysis methodology, the geometry of the containments is assumed to be deterministic while the distributions of material strengths are included. Ellingwood[11] recommended that concrete compressive strength, f'_c, is normally distributed with coefficient of variation (CoV) of 0.14 and a mean value at 1 year, $\overline{f'}_c$,

$$\overline{f'}_c = 8.41 + 1.02\, f'_{cn} \text{ (MPa)} \qquad (5)$$

in which f'_{cn} = specified compressive strength of concrete. For yield strength f_y of ASTM A 615 Grade 60 deformed bar reinforcement, the lognormal distribution is recommended with a mean value of 490 MPa and CoV of 0.11.[11]

4.6 Reliability Assessment

For reliability assessments of these containments, the reliability analysis method developed by BNL is used. By utilizing this method, it is able to determine limit state probabilities for structures under various static and dynamic loads. The methodology can also evaluate the coincidence probabilities of various load combinations. This is important since it is on the basis of the coincidence probabilities that a decision may be made on whether or not a particular load combination (among all the possible mutually exclusive load combinations) is to be considered for design.

The limit state defined in Section 4.3 and the probabilistic models for loads and material strengths described in Section 4.5 are used in the reliability assessments of sample containments. The limit state probabilities for a reference period of 40 years are shown in Table 3.

4.7 Determination of Load Factors

The limit state probability is a quantitative measure of structural performance. The selection of a target limit state probability should consider many factors, e.g., the characteristics of the limit states, the consequence of failure, and the risk evaluation and damage cost. Hence, the target reliability may not necessarily be the same for different limit states.

If a target limit state probability $P_{f,T}$ is specified, the load and resistance factors can be determined such that the limit state probabilities of the sample containments are sufficiently close to the target limit state probability. The closeness is measured by an objective function defined as follows:

$$\Omega(\gamma_p, \gamma_{ES}) = \sum_{i=1}^{4} (\log P_{f,i} - \log P_{f,T})^2 \tag{6}$$

where N is the total number of representative containments and $P_{f,i}$ is the limit state probability computed for the i-th sample containment. w_i represents a weight factor for i-th sample containment. On the basis of the Latin hypercube sampling technique, it is assumed that each sample containment in Table 2 is equally representative, and thus, w_i = 1.0.

Table 3. Limit State Probability ($D+P_a$, $D+E_{SS}$).

Sample	γ_{ES}	P_f			
		γ_p = 1.0	γ_p = 1.1	γ_p = 1.2	γ_p = 1.3
1	1.2	3.502 -5	2.220 -7	3.830 -9	1.911 -10
	1.6	3.502 -5	2.220 -7	3.820 -9	1.911 -10
	1.8	3.502 -5	2.220 -7	3.830 -9	
	2.0	1.518 -5	2.220 -7	3.830 -9	1.911 -10
2	1.2	1.307 -4	1.306 -4	1.306 -4	1.306 -4
	1.6	1.036 -6	8.790 -7	8.788 -7	8.788 -7
	1.8	2.020 -7	4.463 -8	4.447 -8	
	2.0	1.593 -7	1.932 -9	1.777 -9	1.777 -9
3	1.2	9.781 -5	9.766 -5	9.766 -5	9.766 -5
	1.6	8.811 -7	7.262 -7	7.260 -7	7.260 -7
	1.8	1.971 -7	4.262 -8	4.193 -8	
	2.0	1.569 -7	1.955 -9	1.797 -9	1.797 -9
4	1.2	1.922 -5	6.935 -8	4.439 -11	6.959 -14
	1.6	1.922 -5	6.935 -8	4.439 -11	6.959 -14
	1.8	1.922 -5	6.935 -8	4.439 -11	
	2.0	6.681 -6	6.935 -8	4.439 -11	6.959 -14

NOTE: 3.502 -5 = 3.502×10^{-5}

The optimum load factors γ_{ES} and γ_p may be obtained using a minimization technique. For three target limit state probabilities, the optimum load factors are determined as shown below.[5]

Target limit state probability	Optimum load factors	
$P_{f,T}$	γ_{ES}	γ_p
1.0×10^{-5}	1.6	1.1
1.0×10^{-6}	1.7	1.2
1.0×10^{-7}	2.0	1.2

5. CONCLUDING REMARKS

This paper discusses reliability analysis and load combination design criteria for reinforced concrete containments under combined loads. For the safety evaluation of concrete containment structures under various static and dynamic loads, a probability-based reliability analysis method has been developed. An important feature of this method is that finite element analysis and random vibration theory have been incorporated into the reliability analysis. In the method, an appropriate probabilistic model is established for each load. The limit state of the structure is analytically defined and the corresponding limit state surface is established. Finally, limit state probabilities for various load combinations are evaluated.

A procedure for developing probability-based load combination criteria for design of containment structures has also been established. In this procedure, the proposed load combinations is in load and resistance factor design (LRFD) format which uses the principal load-companion load concept. The load and resistance factors are, in general, determined on the basis of limit states and target limit state probabilities. In this paper, the derivations of the load factors for accidental pressure due to a design basis accident and safe shutdown earthquake (SSE) for three target limit state probabilities are described.

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NOTICE

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