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Reliability Assessment of Indian Point Unit 3 Containment Structure Under Combined Loads

H. Hwang, M. ASCE*, M. Shinozuka, M. ASCE**, J. Kawakami*, M. Reich*

Abstract

In the current design criteria, the load combinations specified for design of concrete containment structures are in the deterministic format. However, by applying the probability-based reliability analysis method developed by BNL to the concrete containment structures designed according to the criteria, it is possible to evaluate the reliability levels implied in the current design criteria. For this purpose, the reliability analysis is applied to the Indian Point Unit No. 3 containment. The details of the containment structure such as the geometries and the rebar arrangements, etc., are taken from the working drawings and the Final Safety Analysis Report. Three kinds of loads are considered in the reliability analysis. They are, dead load, accidental pressure due to a large LOCA, and earthquake ground acceleration. This paper presents the reliability analysis results of the Indian Point Unit 3 containment subjected to all combinations of loads.

1. Introduction

Concrete containment structures in the United States are currently designed according to the ASME code[1] and other supplementary requirements such as Standard Review Plan (SRP)[12], etc. The load combinations specified in these criteria are in the deterministic format and the reliability levels implied in the load combinations are not stated explicitly. However, it is important to evaluate these reliability levels to insure the safety of the nuclear structures.

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The Structural Analysis Division of Brookhaven National Laboratory (BNL) has been developing a probability-based reliability analysis methodology for nuclear structures, particularly for concrete containment structures.[4,7,10,11] An important feature of this methodology is that the 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 of nuclear structures under various static and dynamic loads in terms of limit state probability.

By applying the reliability analysis method to the concrete containment structures designed according to the criteria mentioned above, it is possible to evaluate the reliability levels implied in the current design criteria. For this purpose, the reliability analysis is applied to the Indian Point Unit No. 3 containment structure. The results of the reliability analysis are presented in this paper.

2. Containment Description

The Indian Point Unit No. 3 nuclear power plant employs a pressurized water reactor (PWR) nuclear steam supply system furnished by Westinghouse Electric Corporation. The containment structure consists of a vertical right cylinder with a hemispherical dome on the top. The cylinder-dome system is built on a basemat of thickness 9'-0" (2.75m). Thus, the cylinder-dome system is considered to be fixed at the base in the present analysis for simplicity. The thickness of the dome is equal to 3'-6" (1.07m), whereas the thickness of the cylindrical wall is 4'-6" (1.40m). The inside radius of the dome and the cylinder is equal to 67'-6" (20.59m). The height of the cylindrical wall is 148'-0" (45.14m) and the total height of the containment is 219'-0" (66.80m). The containment wall is reinforced with hoop, meridional and diagonal rebars. The details of rabar arrangement for the cylinder and the dome of the containment are tabulated in Tables 1 and 2, respecitvely.

The containment structure is subjected to various static and dynamic loads during its lifetime. In this study, only three types of loads are taken into consideration. They are: dead load, accidental pressure and earthquake ground acceleration. From reviewing the

drawings and the Final Safety Analysis Report (FSAR)[8], it is found that there is no live load acting on the containment structure.

The dead loads arise mainly from the weights of the dome and the cylinder. The weight density of the reinforced concrete is taken to be 150 lb/ft^3 (23.55 kN/m^3). The accidental pressure is assumed to be caused by a large Loss of Coolant Accident (LOCA). The accidental pressure is considered as a quasi-static load and is uniformly distributed on the containment wall. The design value of the pressure is 47 psi (0.32MPa). From FSAR, the design value of the ground acceleration for the Operating Basis Earthquake (OBE) is 0.1 g applied horizontally and 0.05 g applied vertically. Additionally, the ground acceleration for the Design Basis Earthquake (DBE) is 0.15 g horizontally and 0.10 g vertically.

3. Containment Modelling

In order to utilize the finite element analysis results in computing the limit state probabilities, the containment modelling should be made in such a way that the local coordinates of the elements have the same directions as those of the rebars. This is very easy to achieve in this study, since all the rebars are in the hoop and meridional directions.

The finite element utilized in the analysis is the shell element as described in the SAPV computer code. A three-dimensional finite element model is used for the structural analysis of the containment. A detailed cross-sectional view of the containment model is shown in Fig. 1. As can be see from this figure, the containment is divided into 23 layers. Except at the top of the dome, each layer has 24 elements such that the nodal points are taken every 15° in the circumferential direction. This discretization requires a total of 553 nodes and 540 elements. The boundaries of the elements are made such that it matches the change of the reinforcements. Hence, the amount of reinforcements in most of the elements is the same as shown in tables 1 and 2. However, the meridional rebars in the dome portion are varied in different elevations, an average value of the meridional reinforcement in these ele-

ments is used. It is noted that the diagonal rebars, which provide in-plane seismic shear resistance, are not included in the present analysis. The liner is also disregarded as a load carrying structural component in the analysis. Furthermore, such other complications as penetrations, personal lock and equipment hatches are not included in the study.

4. Material Properties

In order to perform a reliability analysis, it is necessary to determine the actual material properties. In the present study, the mean values of the material properties are used in the analysis. The variation of material properties will be included in the sensitivity studies in the future. The properties for the concrete and rebars are summarized as follows:

Concrete

The minimum compressive strength of concrete at 28 days used for the Indian Pont Unit No. 3 containment is 3000 psi (20.7MPa). However, the mean value of the compressive strength is estimated to be 4896 psi (33.78MPa) from test data. The weight density of the concrete is taken to be 150 lb/ft^3 ($23.55 kN/m^3$). Young's modulus and Poisson's ratio are 3.1×10^6 psi (21390MPa) and 0.2, respectively.

Reinforcing Bars

As can be seen from Tables 1 and 2, No. 18 rebars are the main reinforcement used in the containment structure. Hence, the statistics for No. 18 rebars is used to represent all other types of rebars. Young's modulus and Poisson's ratio are taken to be 29.0×10^6 psi (200100MPa) and 0.3, respectively. From the test data the mean value of the yield strength f_y are estimated to be 71.8 ksi (495.42MPa).

5. Probabilistic Models for Loads

Various static and dynamic loads act on the containment structure

during its lifetime. These loads may be caused by normal operating, environmental and accidental conditions. Since the loads intrinsically involve random and other uncertainties, an appropriate probabilistic model for each load must be established.

Dead Load

As mentioned in Section 2, the dead load primarily arises from the weights of the containment wall. It is noted that there are some uncertainties as to the actual magnitude of the dead load.[6] For the purpose of this analysis, however, 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 ($23.55 kN/m^3$).

Accidental Pressure

The accidental pressure is considered as a quasi-static load and it is uniformly distributed on the containment wall. The accidental pressure is idealized as a rectangular pulse and will occur in accordance with the Poisson law during the containment life. Under these assumptions, three parameters are required to model the accidental pressure: the occurrence rate λ_p (per year), the mean duration μ_{dp} (in seconds) and the intensity P. For the Indian Point Unit No. 3 containment, the mean duration is taken to be 1200 seconds. This value is obtained from the approximation of the time history. According to the Indian Point Probabilistic Safety Study[9], the mean occurrence rate for a large LOCA is 2.16×10^{-3}/yr. The intensity of the accidental pressure is treated as a Gaussian random variable. The consensus survey of the nuclear structural loads, which was carried out by BNL, indicates that the ratio of the mean value to the design value is 0.89 and the coefficient of variation is 0.12.[5,6] Since the design value of the accidental pressure is 47 psi, (0.32MPa), the mean value, $\overline{P}$ and the standard deviation, σ_p, are 41.83 psi (0.29MPa) and 5.02 psi (0.0345 MPa), respectively.

Earthquake Ground Acceleration

The earthquake ground acceleration is assumed to act only along the global x direction. The ground acceleration is 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:

$$S_{gg}(\omega) = S_o \frac{1 + 4\zeta_g^2(\omega/\omega_g)^2}{\left[1 - (\omega/\omega_g)^2\right]^2 + 4\zeta_g^2(\omega/\omega_g)^2} \tag{1}$$

where the parameter S_o represents the intensity of the earthquake. ω_g and ζ_g are the dominant ground frequency and the critical damping, respectively, which depend on the soil conditions of a site. The soil condition of the Indian Point Power Plant is determined as rock.[8] For such a soil condition, Ref. 3 recommends that ω_g and ζ_g in Eq. 1 are taken to be 8πrad/sec and 0.6, respectively. The mean duration μ_{dE} of the earthquake acceleration is assumed to be 15 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 which is assumed to be 3.0. The standard deviation of the ground acceleration, σ_g, computed by integrating the Kanai-Tajimi spectral density function with respect to ω, is

$$\sigma_g = \sqrt{\pi\omega_g(\frac{1}{2\zeta_g} + 2\zeta_g)} \;\; \sqrt{S_o} \tag{2}$$

The peak ground acceleration A_1, given an earthquake, can be rewritten as

$$A_1 = \alpha_g\sqrt{S_o} \tag{3}$$

where

$$\alpha_g = P_g\sqrt{\pi\omega_g(\frac{1}{2\zeta_g} + 2\zeta_g)} \tag{4}$$

If the earthquake occurs in accordance with the Poisson law at a rate λ_E per year, it is easy to show that the probability distribution $F_A(a)$ of the annual peak ground acceleration A is related to the

probability distribuiton $F_{A_1}(a)$ of A_1 in the following fashion.

$$F_A(a) = \exp\left\{-\lambda_E[1 - F_{A_1}(a)]\right\}$$

or (5)

$$F_{A_1}(a) = 1 + \frac{1}{\lambda_E}\left[\ln F_A(a)\right]$$

Therefore, if a_o indicates the minimum peak ground acceleration for any ground shaking to be considered an earthquake, $F_{A_1}(a_o) = 0$ and hence, $\lambda_E = -\ln F_A(a_o)$. Assuming that $F_A(a)$ is of the extreme distribution of Type II, i.e.,

$$F_A(a) = \exp[-(a/\mu)^{-\alpha}] \tag{6}$$

where α and μ are two parameters to be determined. By least square fitting to a harzard curve given in the PRA study[9], we find $\alpha = 3.14$ and $\mu = 0.0135$.

From Eqs. 5 and 6, we obtain:

$$F_{A_1}(a) = 1 - (a/a_o)^{-\alpha} \qquad a \geqslant a_o \tag{7}$$

Under these conditions, one finds that $\lambda_E = 1.64 \times 10^{-2}$/year for $a_o = 0.05$ g. Combining Eqs. 3 and 7, and writing Z for $\sqrt{S_o}$, we further obtain the probability distribution and density functions of Z in the forms, respectively,

$$F_Z(z) = 1 - (\alpha_y z/a_o)^{-\alpha}$$

for $z \geqslant a_o/\alpha_y$ (8)

$$f_Z(z) = \alpha(\alpha_y/a_o)(\alpha_y z/a_o)^{-(\alpha+1)}$$

The information about the maximum earthquake ground acceleration, a_{max}, which represents the largest earthquake possible to occur at a particular site, is needed in order to determine the limit state probability. In this study, a_{max} is chosen to be equal to 0.7? g.

6. Finite Element Analysis

Static Analysis

As mentioned in Section 5, dead load and accidental pressure are considered to be static loads acting on the containment. Using the finite element model described in Section 3, a static analysis of the containment due to dead load alone is performed. Similarily, the analysis of the containment due to unit accidental pressure alone is also carried out. These results are going to be used in the reliability analysis.

Dynamic Analysis

For dynamic analysis of structures, modal analysis is employed. Hence, the dynamic characteristics of the structures are represented by the natural frequencies and associated mode shapes. Using the model described in Section 3 and one half of the stiffness of the uncracked section, the first 20 natural frequencies and corresponding mode shapes are evaluated. It is important to choose the significantly participating modes for the reliability analysis. In this study, only the first and second pairs of bending modes are included in the analysis.

7. Limit State For The Containment

A limit state essentially represents a state of undesirable structural behavior. In general, it will depend on the characteristics of the structures and the loadings that act on the structures. For a particular structural system, it is possible that more than one limit state may be considered. 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 for containments is defined according to the ultimate strength theory of the reinforced concrete. It is described as follows: At any time during the serivce life of the structure, the state of structural response is considered to have reached the limit state if a 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 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. [2] A typical limit state surface is shown in Fig. 2. In this figure, point "a" is determined from a stress state of uniform compression and point "e" from uniform tension. Points "c" and "c'" are the so-called "balanced point", 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.

8. Reliability Analysis Results

The reliability analysis methodology used in this study is summarized in Ref. 4. Based on this reliability analysis method, the structural model, loading conditions and the limit state described in the preceding sections, the reliability analysis for the Indian Point Unit No. 3 containment structure is carried out. The results are presented in this section.

Dead Load and Accidental Pressure (D+P)

The load characteristics of the dead load and the accidental pressure due to a large LOCA are described in Sections 5. The conditional limit state probability of the critical elements is 3.99×10^{-7}. The critical elements are the elements denoted as 289 to 312, which are located in the first layer of the dome section just above the spring line. Since the structure and the loads are both axisymmetrical in this case, all the elements located at the same level have the same limit state probability. The limit state is reached as the hoop rebars in the critical elements are yielded. When the conditional limit state probability is multiplied by the expected number of such simultaneous occurrences during the containment service life of forty years, the unconditional limit state probabilities $P_f(D+P)$ under the simultaneous action of D and P is obtained, and is equal to 3.46×10^{-8}.

Dead Load and Earthquake Ground Acceleration (D+E)

The analytical idealization of the earthquake ground acceleration is presented in Section 5. For the combination of the dead load and earthquake ground acceleration, the critical elements are elements 6, 7, 18 and 19. These elements are located in the lowest finite element layer and immediately adjacent to the global x axis. In this case, the limit state is reached as the meridional reinforcing bars in the critical elements are yielded. The locations of the critical elements and the manner in which the limit state is reached are obviously consistent with the structural and loading symmetry with respect to the x-axis under this particular load combination. The lower and upper bounds of the conditional limit state probability $P^{(D+E)}$ are found to be very close and equal to 1.02×10^{-8}. Multiplying by the expected number of simultaneous occurrences of D and E, the unconditional limit state probability during the 40 year lifetime is 6.72×10^{-9}.

The fragility curves are used in seismic probabilistic risk assessment (PRA) studies for nuclear power plants. The fragility is defined as the conditional limit state probability for a given peak ground acceleration. Using the BNL method for generating fragility curves[4], a fragility curve for the flexural limit state of the containment is presented in Fig. 3 and Table 3 shows the corresponding numerical values. Since all the data used in the analysis are taken to be best estimate values (or mean values), this fragility curve may be interpreted as the mean fragility curve. It can be seen from Table 3, the peak ground acceleration corresponding to the median of the curve is 1.2g.

Dead Load, Earthquake Ground Acceleration and Accidental Pressure (D+E+P)

The probabilistic characteristics of the loads are described in Section 5. Under the combination of these loads, the critical elements are found to be elements 294, 295, 306 and 307, which are located immediately adjacent to the global x-axis (when projected onto a horizontal plane) at the first layer above the springline. This is the same level at which the critical elements are found under the D+P load combination. The manner in which the limit state is reached is also the same as for

the D+P combination (i.e., yielding of the hoop rebars). The lower and upper bounds of the conditional limit state probability $P^{(D+E+P)}$ under this load combination are also very close and the average is 1.62 x 10^{-5}. Finally, multiplying by the expected number of simultaneous occurence of these loads, the unconditional limit state probability $P_f(D+E+P)$ under this load combination during the containment life of forty years is 8.85 x 10^{-13}.

Comparing the limit state probabilities under the load combination D+P with those under the combination D+E+P, it is observed that (1) the mode in which the limit state is reached (yielding of the hoop rebars) in the critical elements is the same and (2) the critical elements under the load combination D+E+P comprise the four elements which are most stressed by the additional earthquake load among those critical elements under the load combination D+P. These observations suggest that the accidental pressure P is a dominant factor in controlling the conditional limit state probability. The substantial reduction in the unconditional probability $P_f(D+E+P)$, as compared with $P_f(D+P)$, is primarily attributable to the fact that the probability of simultaneous occurrence of three loads, D+E+P, 5.46 x 10^{-8}, is much smaller than that of two loads D+P, 8.64 x 10^{-2}.

Overall limit State Probability

The limit state probabilities evaluated in the preceding section are those at the critical elements within the containment under various load combinations. While the limit state probability of the containment as a whole, or the system limit state probability, under a certain load combination is always larger than that of the critical elements, the author's experience in structural reliability analysis suggests that the difference between the system limit state probability and the limit state probability of the critical elements is tolerable for the type of load-structure system under consideration. Therefore, for the sake of analytical simplicity and computational economy, the present study approximates the containment limit state probability under each load combination by the critical element limit state probability. Under the assumption that the containment will not fail under dead load alone, the

overall containment limit state probability p_f is then obtained as the sum of the limit state probabilities under all these (mutually exclusive) load combinations. Hence, the containment limit state probability is 4.13×10^{-8} for its lifetime of forty years. The reliability analysis results of the containment are summarized in Table 4.

9. Concluding Remarks

This paper presents the reliability analysis results of the Indian Point Unit No. 3 containment structure under dead load, accidental pressure and earthquake ground acceleration. The reliability analysis method for concrete containment structures developed by BNL is employed in the analysis. This is the first attempt to carry out the reliability analysis for the existing containment structures in order to evaluate the reliability levels implied in the design criteria. It is noted that the estimated reliability levels are affected by the judgements made in the design process and the assumptions made in the reliability evaluation. In order to reasonably assess the reliability levels implied in the design criteria, it is necessary to continue the efforts by carrying out the reliability analysis for other existing containment structures designed according to these criteria.

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Table 1. Cylinder Reinforcement.

Elevation	Hoop	Meridional		Diagonal
		Primary	Secondary	
0 to 25'-0"	2#18 @ 14 in	1#18 @ 12 in	1#18 @ 12 in	1#18 @ 30 in
25'-0" to 45'-5"	"	"	1#11 @ 12 in	"
45'-5" to 50"-3"	"	"	2#11 @ 36 in	"
50'-3" to 54'-10"	"	"	1#11 @ 36 in	"
54'-10" to 110'-6"	"	"		"
110'-6" to 148'-0"	"	"		1#18 @ 60 in+ 1#14 @ 60 in

Table 2. Dome Reinforcement.

Angle From Spring Line (Degrees)	Hoop	Meridional	Diagonal
0°-9.5°	1#14 { @ 81/4" outside @ 8.0" inside	1#18 @ 0.796°	1#18+1#14 @ 4° (horiz. dist.)
9.5°-18.5°	1#14 @ 8.0 in	"	"
18.5°-35°	"	"	1#11 @ 2° (horiz. dist.)
35°-55°	"	"	
55°-60°	1#14 @ 9.0 in	"	
60°-75°	"	1#18 @ 1.593°	
75°-83°	"	1#18 @ 3.186°	
83°-86°	"	1#18 @ 6.37°	
86°-90°	"	1#18 @ 12.74°	

Table 3. Fragility Curve.

PGA(g)	p(D+E)	PGA(g)	p(D+E)
0.60	2.15 E-6	1.205	0.50
0.65	3.79 E-5	1.25	0.61
0.70	2.45 E-4	1.30	0.71
0.75	1.11 E-3	1.35	0.80
0.80	3.86 E-3	1.40	0.86
0.85	1.09 E-2	1.45	0.92
0.90	2.58 E-2	1.50	0.95
0.95	5.34 E-2	1.55	0.97
1.00	9.84 E-2	1.60	0.98
1.05	0.18	1.65	0.99
1.10	0.27	1.70	1.00
1.15	0.38		
1.20	0.49		

Table 4. Limit State Probabilities.

Load Combinations	Expected Number of Occurrences	Conditional Limit State Probability	Unconditional Limit State Probability	Critical Elements
D+P	8.64×10^{-2}	3.99×10^{-7}	3.46×10^{-8}	289,290, ...,312
D+E	6.56×10^{-1}	1.02×10^{-8}	6.72×10^{-9}	6,7,18, 19
D+E+P	5.46×10^{-8}	1.62×10^{-5}	8.85×10^{-13}	294,295, 306,307
Overall	---	---	4.13×10^{-8}	---

NOTE: Assuming the containment will not fail under dead load alone.

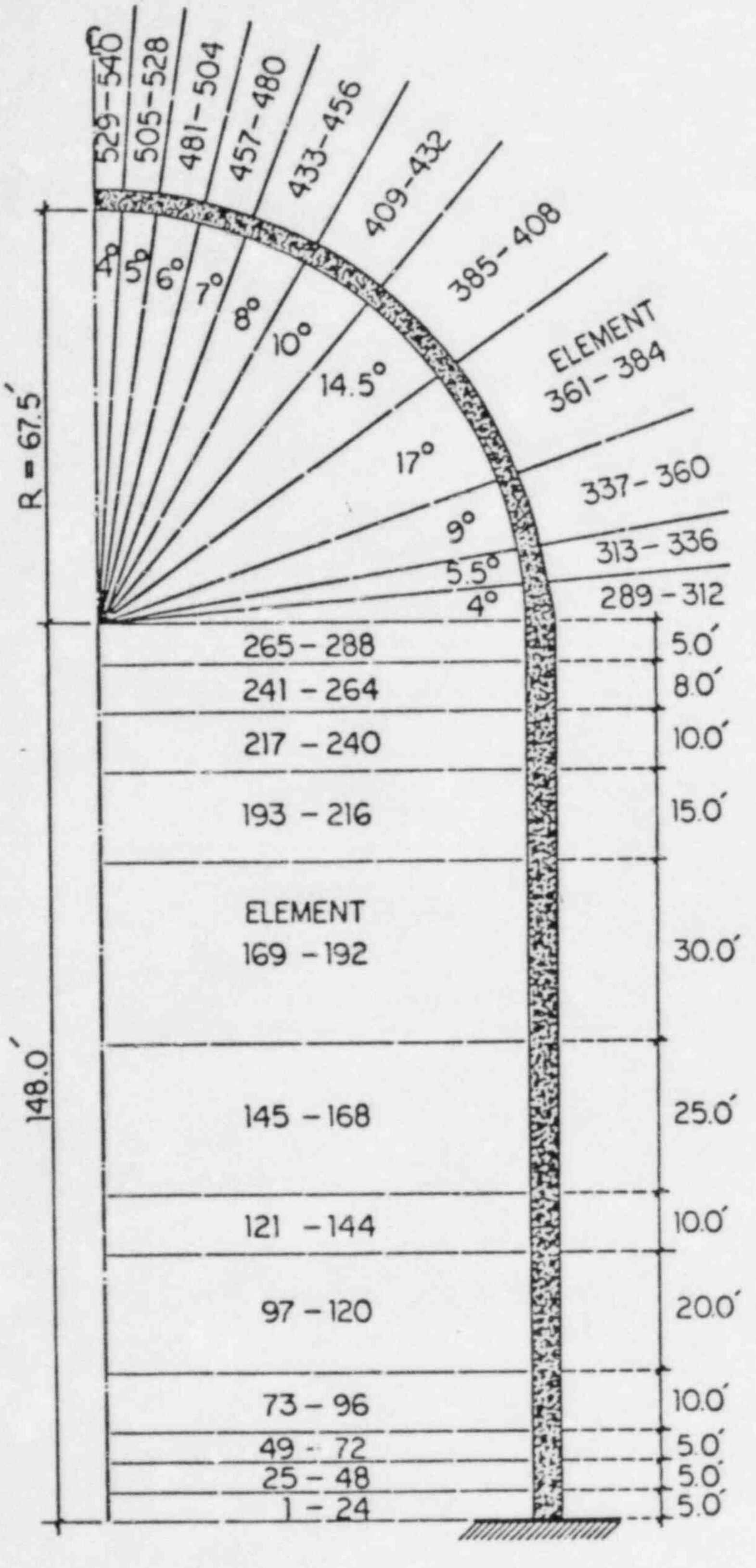

Fig. 1. Cross Section of Containment Model.

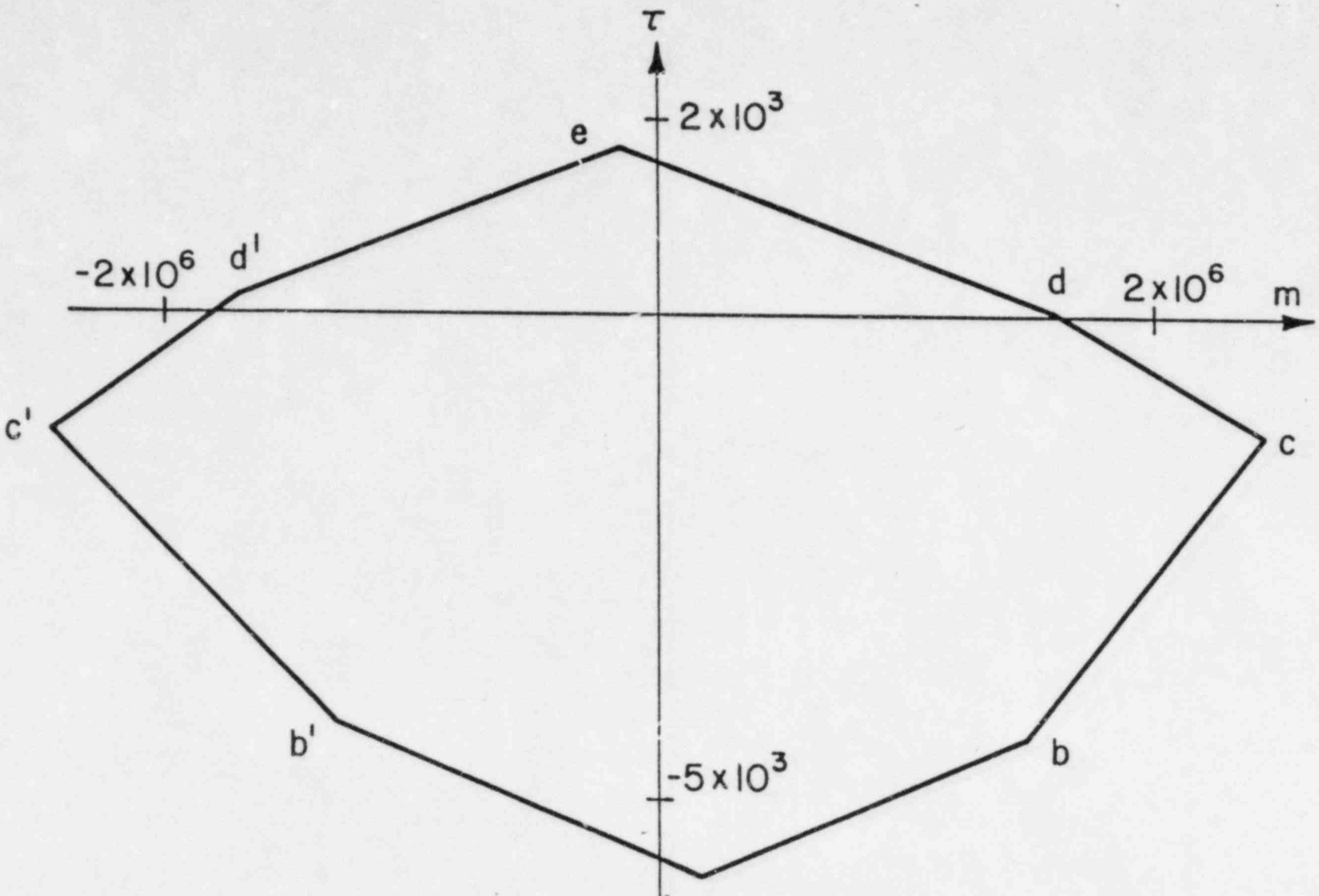

Fig. 2. Limit State Surface.

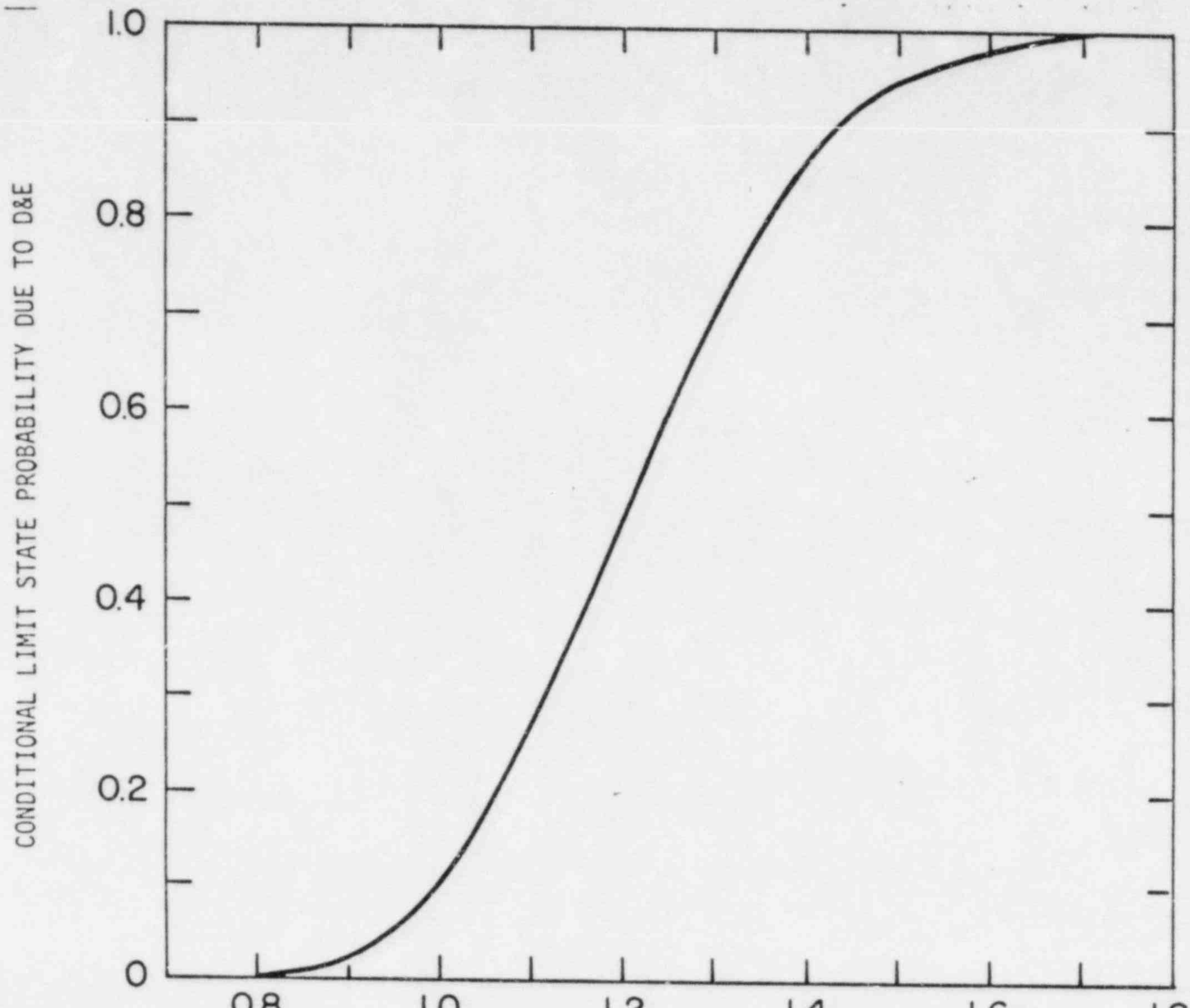

Fig. 3. Fragility Curve.

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