PROBABILISTIC ASSESSMENT OF SEISMIC RISK FOR NUCLEAR POWER PLANTS

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SUMMARY

Probabilistic approaches to the design, siting, and safety analysis of nuclear power plants have been proposed by Farmer, Wall, and Garrick. Farmer and Wall established a limit-line which delineates between acceptable and unacceptable risks. To implement the method, all accidental chains are systematically analyzed to determine their probability and associated fission product release magnitude; the combination is compared to the limit-line. For proper implementation, the seismic risk should be evaluated in a quantified manner.

Conceptually, this evaluation is made in two stages: the probability of an earthquake occurrence as a function of its intensity and, given a seismic intensity, the conditional probability of damage. This paper reports on initial study into the latter aspect.

The effect of uncertainty in several parameters which determine the response of a nuclear reactor building to earthquake forces is assessed. Probability distributions for material properties were determined from site measurements. These distributions were utilized for determining the building response and the damage criterion. A subjective probability density function for damping was assigned from the available information and the judgment of experienced engineers. Four accelerograms, El Centro N-S 1940, and three artificial earthquakes were used to represent the variability in the forcing functions. The uncertainty in the model idealization was assessed by evaluating three alternate models. A versatile computer program was developed to compute the response of the mathematical model to the forcing functions using matrix formulation and modal method of analysis. An exact solution, rather than numerical integration was used to obtain the dynamic response of the system in generalized coordinates.

The stresses within the reactor building are similar for different earthquakes when they are normalized to ground acceleration, indicating that the shape of the accelerogram and its frequency content are less significant than the magnitude of the maximum ground acceleration for the reactor building. The variation in modulus of elasticity for concrete had significant effect on the building response. Damping, in general, reduced the response, but in cases where the duration of an earthquake is short the effect was not very significant.

The normal design of the reactor building is deterministic and is based on a 0.2 g design basis earthquake. Using the results obtained by the parametric study on the variation of response, the probability of damage was estimated by a Monte Carlo analysis. It was estimated that, given the occurrence of a design basis earthquake, there is less than approximately 10^{-3} probability of cracking in the shear walls of the reactor building. The initiation of cracking in the concrete should not lead to significant release of contained fission products.

1. Introduction

Probabilistic approaches to the design, siting, and safety analysis of nuclear power plants have been proposed by Farmer [1], Wall [2, 3], and Garrick [4]. Farmer and Wall established a limit-line which delineates between acceptable and unacceptable risks. The limit-line, a complementary cumulative distribution function for fission product release magnitude, has been established for reactor sites by a statistical analysis of meteorology [3, 5]. To implement the method, all accident chains are systematically analyzed to determine their probability and associated fission product release magnitude; the combination is compared to the limit-line. The approach has been discussed in more detail elsewhere [1, 3], and for nuclear power plants, has been applied to seismic risk [6], aircraft crashes [7], tornadoes [8], floods [9], and loss-of-coolant accident [10].

For proper implementation, the seismic risk should be evaluated in a quantified manner. Conceptually, this evaluation is made in two stages: the probability of an earthquake occurrence as a function of its intensity, and given a seismic intènsity, the conditional probability of damage to the plant and release of fission products. This paper reports on an initial study into the latter aspect. The probability of cracking in the reinforced concrete of the reactor building is estimated as a function of the seismic intensity. A typical design for a boiling water reactor building is used as an example.

The dynamic response of a reactor building to earthquake forces includes many uncertainties: variability of material properties, system damping, input earthquake motions, and idealization of physical structure to generate mathematical model. In order to assess the probability of cracking, a parametric study was made encompassing these uncertainties. An additional benefit of such a study is the establishment of the relative importance of each uncertainty. A typical design of a boiling water reactor building is used as an example.

2. General method

A broad outline of the sequence of operation involved in the analysis is shown in Figure 1, starting from the physical structure and the earthquake forces and proceeding to the determination of the probability of damage. At each stage, the factors affecting damage probability are noted. The equation of motion of the reactor building under earthquake forces can be represented by the following matrix equation:

$$[M]\ [\ddot{U}] + [C]\ [\dot{U}] + [K]\ [U] = [F] \qquad (1)$$

where [M] is the mass matrix, [C] is the damping matrix, [K] is the stiffness matrix, [U] is the displacement vector, and [F] is the effective force vector due to earthquake accelerations.

The probability of damage is a function of the input parameters and their uncertainties and also the mathematical model utilized to represent the physical structure. Some of the important factors include the uncertainties in generating a mathematical model for the reactor building, accelerations due to earthquakes including the effect of site conditions, uncertainty distributions for physical properties and damping in the system, damage criteria, foundation stiffness, and inelastic behavior.

Other minor factors include uncertainties in physical dimensions and mass. However, in reactor containment structures these variations are minor and thus can be neglected.

2.1 Mathematical model of reactor building

The cross section and a typical sectional plan of the reactor building are shown in Figures 2 and 3. The mathematical model for the reactor building is shown in Figure 4; the masses are assumed to be concentrated at each floor level and range between 30000 and 100000 kips. There is a light, comparatively flexible roof structure attached to the massive, stiff reactor building. The mass of the roof structure, approximately 2700 kips, is concentrated at the roof level.

To compute the response of the reactor building, it is necessary to determine the stiffness matrices of the elements of the mathematical model. The stiffness matrices in turn must be derived from the cross-sectional plans and the deformation pattern of the building under earthquake forces. The reactor building in plan has essentially an outside wall, box-type structure, and two inside circular shields. The shields and the walls are connected by floors at elevations shown in Figure 2.

In the present study, lateral motions in each plane are studied independently and the effect of vertical accelerations due to earthquake motions is not considered. A preliminary investigation showed that the stresses due to vertical accelerations for the type of structure under consideration are not large compared to the stresses generated by lateral accelerations. The effect of vertical accelerations will be discussed in detail in another paper.

Figure 5 shows various types of behavior under lateral loads. It is assumed in Model A that the shields and outside walls are acting together, which means that the shields have axial deformations. This behavior is an extreme one. More accurate behavior would be represented by Model B in which shields and outside walls act separately. A further improvement is shown in Model C which includes the participation of the floors.

The stiffness matrix in eq. (1) is determined by a displacement method [11]; the inertial forces corresponding to rotations are neglected. The set of linear differential equations is uncoupled by using modal matrices; modal damping is assumed. The time history of the building response is computed exactly by an analytical matrix formulation [12]. The internal stresses in each major component are calculated. The time interval for calculation of the modal response is largely determined by the earthquake accelerogram and is 0.005 seconds. To minimize computation time, the building response and stresses are calculated less frequently than the modal response; and a time interval of

0.01 seconds is found optimum for a good estimate of the maximum stresses.

2.2 Earthquake forces

It is extremely difficult to predict the exact nature of a future earthquake because of the complexity of ground motions. Not only the peak acceleration but also the duration of the earthquake and its frequency content are very important in determining the response of the system. Recent studies showed that the influence of local site characteristics might be significant in determining the nature of earthquake motions that can be expected at that site. Since the reactor building under study is founded on competent rock, the effect of soil was not included.

For the present analysis, it was considered desirable to include the widely used strong motion earthquake record (El Centro 1940 N-S Component) which was also the accelerogram used for the normal seismic design analysis of the building considered in this example. However, to examine the sensitivity of the results to different earthquake motions, it was felt desirable to include the artificial earthquakes, B-1, C-1, and D-1, developed by Jennings, Housner, and Tsai [13] to represent varying earthquake motions. Earthquake B is expected to represent the motions in a Magnitude 7 earthquake while Earthquake C is designed to represent a Magnitude 5 or 6 earthquake. Earthquake D is designed to represent motion close to a fault.

A summary of the characteristics of the above four earthquake accelerograms is given in Table I. In the studies of the influence of intensities of earthquake motions on response and damage probabilities, the four accelerograms are normalized to peak accelerations of 0.1, 0.2, 0.3, 0.4, and 0.5 g.

The probability of occurrence of an earthquake as a function of local parameters is the subject of another study. As a point of reference, a preliminary examination of the historical record suggests that the probability of a design basis earthquake with a maximum acceleration of 0.2 g at the site can be no greater than 10^{-2} per year and is probably very much lower, e.g., 10^{-3} or 10^{-4}.

2.3 Uncertainties in physical properties of concrete

The density, strength, and elasticity of concrete will vary from batch to batch and from day to day at the reactor site even though all samples satisfy the quality control standards. To examine this variability, actual size data for the 28-day compression test results were analyzed for several hundred samples from 11 mixes. For one widely-used concrete mix, the density and 28-day compression strength data were available for 89 valid samples. A linear regression analysis of these data showed that the two parameters were highly correlated. The mean, standard deviations and correlation coefficient were:

28-day compression strength	f'_c	$= 4056 \pm 500$ psi
density	w	$= 141.5 \pm 3.0$ lb/ft^3
correlation coefficient	r	$= 0.848$

Truncated normal distributions were found to be fair approximations to both f'_c and w; the truncation was made at $\pm 3\sigma$. The specified minimum for the 28-day compression strength was 2650 psi (-3σ) so apparently the probability of deficiency is about 0.05 percent.

The modulus of elasticity for concrete was estimated from the following relationship:

$$E = 33\, w^{1.5} (f'_c)^{0.5} \tag{2}$$

By using a Monte Carlo analysis, described later, the frequency distribution for E was derived from the assumed normal distributions for f'_c and w. The value of E is 3538 ± 436 psi and a truncated normal distribution would be a fair approximation (not used in the later analysis) with truncation at $\pm 3.67\sigma$.

The response calculations are made using the modal method. There is uncertainty in the appropriate damping coefficient. For concrete under very low values of stress where no cracking is anticipated, a value of 1 percent damping is recommended. As the stress increases, cracking develops in concrete and the damping is increased. For reinforced concrete with slight cracking, a 2 percent damping is recommended. This value is increased to 5 percent for considerable cracking. A damping coefficient of 7 to 10 percent is recommended for reinforced concrete at higher levels of cracking near the yield point. The damping coefficient is increased from 10 to 15 percent where the stresses are greater than yield point strain [14]. Thus, it is desirable to use different probable damping values for different intensities of ground shaking. However, for the linear analysis considered here, the uncertainty in damping is encoded by the subjective probability distribution as shown in Figure 6 for all five levels of peak accelerations. As argued in a previous paper [2], this subjective concept of probability is the correct tool for consistent decision-making in the face of uncertainty.

2.4 Damage criteria

To determine the damage criteria for complex structures similar to nuclear reactor containment systems is very difficult. Experimental investigations are, in general, concerned with simple loadings. A damage criterion at this time is based by simply noting the maximum stress levels during earthquake. However, this may not give the estimate of true damage potential. If an earthquake lasts a long time, maximum stress levels may be reached several times during the earthquake thus increasing its potential to damage. During a shärp, short-duration earthquake, maximum stress levels may be reached only once and thus reducing its potential to damage. More information is needed about the actual behavior of the reactor buildings under earthquake forces to improve the judgment on the damage criterion.

Approximate calculations have been made to estimate the relative importance of shear and bending deformations for an idealized box-like, four-wall structure similar to the reactor building. These calculations indicate that, as expected, under a static horizontally applied force, the relative horizontal displacement between two adjacent floor levels is essentially all due to shear.

Clough and Benuska [15], in their investigation of the behavior of high-rise buildings subjected to earthquake forces, studied the results of 194 tests on beams without web reinforcement used in the generation of empirical formulas for the ACI Building Code and suggested the following relationship for the ultimate shear:

$$\tau_{ult} = 4.75 \sqrt{f'_c} \tag{3}$$

By using the truncated normal distribution for f'_c, a frequency distribution for τ_{ult} was estimated by the Monte Carlo analysis described later.

3. Parametric study of the response of reactor building

3.1 Effect of mathematical model

By observing Figure 5, it can be seen that the total moment of inertia of the section of Models A and B about the neutral axis is significantly different because of the independent action of the shells. However, the total shear area is assumed to be the same for both models. Figure 7 shows the dynamic characteristics of Models A and B. The only significant difference between these characteristics is about 6 percent in the second mode frequency. Thus, it is concluded that changes in moment of inertia do not significantly affect the dynamic characteristics of the model of the reactor building under study.

The results of Model C are not shown here but are almost identical to those of Model B which show that the contribution of floors to the element stiffness is negligible. For the parametric study reported next, Model B is assumed to represent the behavior of the reactor building under earthquake forces. The behavior of the roof is governed by the first two modes. The contribution of the first mode to the response of the reactor building (except the roof) is practically negligible, while the second and third modes contribute significantly. Selected values of maximum responses are given in Table II for two different earthquakes. It can be seen that while Model A gave lower response values for the El Centro type of motion, the same model gave higher response values for the C-1 type of motion. The differences in shear are close to 25 percent for the C-1 type of motion although the difference in frequency of the two models is only 6 percent.

One can use highly complex three-dimensional models with three components of accelerograms acting simultaneously. However, this type of model might get too complicated for analysis and expensive for parametric variation. One has to use judgment in these cases to determine where to put the effort, either too complex a model with many uncertainties and one analysis or simpler models with several analyses using different parameters. Additional data and information are needed to get reliable stiffness characteristics for these mathematical models specially in the ranges approaching yield levels.

3.2 Effect of modulus of elasticity on concrete, damping, and earthquakes

Sectional properties about the horizontal axis are used for a parametric study of the response of the reactor building as a function of the modulus of elasticity of concrete, modal damping, and the variation of the accelerogram record. The predicted dynamic characteristics of the nuclear reactor building for three values of E for concrete are shown in Figure 8.

It may be seen that increasing the modulus of elasticity reduces the building periods, as expected. The values of E represent the mean and "3σ" extremes of its frequency distribution. For this extreme variation in the modulus of elasticity, the first mode period being primarily associated with the superstructure remains unchanged; and the second, third, and fourth periods are altered about +23 and -14 percent about their modal values.

Tables III and IV present the maximum shear force and bending moments at the base of the reactor building for three values of damping and three values of modulus of elasticity of concrete. Values are given for four different accelerograms, all normalized to a maximum ground acceleration of 0.2 g.

The normalizing of the building response with reference to maximum ground accelerations is logical since the reactor structure is a relatively high frequency building and thus is more affected by the trace of ground acceleration than those of velocity and displacement [16, 17]. However, the effect of the shape of the accelerogram on the moment and shear responses of the reactor building is surprisingly small except for the extreme combination of 2 percent damping and modulus of elasticity of concrete of 2300 ksi, as can be seen from Tables III and IV. For the modal values of β and E (5 percent and 3500 ksi), the estimated maximum shear force and internal bending moment are within $\pm$ 8 and $\pm$ 12 percent, respectively, for four widely divergent accelerogram records. These variations are negligible when compared to the overall uncertainties in seismic design. For a particular earthquake, the maximum shear force and bending moment vary by factors of 1 to 2.4 for the extreme variations in damping and modulus of elasticity of concrete. Damping, in general, reduces the response; but the effect is small especially for short-duration earthquakes.

It is believed that a parametric study of the type presented in Tables III and IV gives a much better feeling of what could be expected in the event of an earthquake instead of relying on one number when one knows clearly that there are many uncertainties involved in several aspects of seismic design.

4. Probability of damage

Given an earthquake intensity, the conditional probability of damage was computed by a Monte Carlo analysis. Cracking of the reinforced concrete was assumed to occur when the shear stress exceeds the ultimate shear stress. In about 80 percent of the calculations, the maximum shear stress in the outside shear wall of the reactor building was found to occur between elevations of 500 and 517.5 feet. Accordingly, this component was designated as one of the critical components even though shear cracking at this location would not directly cause any damage or release of any fission products. The shear stress in this component was computed for 13 combinations of Young's modulus (E) and the damping coefficient (β) and for each of the four earthquake accelerograms. The values are stated in Table V. These data were fitted by a polynomial expression with errors ranging between 0.1 and 18 percent and averaging 4.6 percent. By using these polynomial expressions, the shear stress in the critical component could be estimated for any combination of E and β.

The Monte Carlo program generated sets of random values for f'_c, w, and β following their frequency distributions described earlier. The normal deviates were generated by the Box-Müller method [18]. By using eq. (2), a value for E was computed which, together with the value for β, was used in the polynomial to estimate the shear stress τ in the critical component. The ultimate shear stress τ_{ult} was computed from the same value of f'_c. If τ was equal to or greater than τ_{ult}, cracking was assumed to have occurred. Dependence of variables has been partially accommodated since both τ and τ_{ult} are computed from the same random value of f'_c and a correlated value of w. However, damping and modulus of elasticity were assumed independent. Since a linear analysis is being used, the value of τ is proportional to the scaled intensity of the earthquake and the same Monte Carlo trial can be used to evaluate the probability of cracking for a range of earthquake intensities. The conditional probability of cracking was computed by 10000 trials for each earthquake accelerogram.

The probability of cracking as a function of earthquake intensity is shown in Figure 9. The cracking probabilities are almost identical for each earthquake accelerogram. For the El Centro accelerogram, the probability of cracking in the lower outside wall of the reactor building is $\sim 10^{-3}$, 0.4 and 1.0 for peak accelerations of 0.2, 0.3, and 0.4, respectively. Since this component is thought to be the critical one, the probability of damage to other components of the reactor building are possibly lower. For perspective, it should be noted that for the reactor building used in this example, the operating and design basis earthquakes are 0.1 and 0.2 g, respectively.

5. Summary and conclusion

The objectives of probabilistic risk assessment are (a) to place each hazard in perspective and thereby allocate resources for an overall balanced design of a nuclear power plant and (b) to provide uniform criteria for different sites. The parametric analysis, necessary to estimate probability, provides insight into the relative contributions of uncertainties in the mathematical model, damping, physical properties of concrete, and the earthquake accelerogram to the uncertainty in the response for the expected values of damping and the modulus of elasticity. It is concluded that uncertainties in the properties of one dimensional mathematical model and the precise shape of the earthquake accelerogram do not cause significant uncertainties in the response of a typical reactor building at higher values of damping.

It is concluded that for the reactor building under consideration, given the occurrence of the design basis earthquake, there is less than 10^{-3} probability of cracking in the shear walls of the reactor building. The initiation of cracking in the concrete should not lead to a significant release of contained fission products. By combining this conditional probability with an assumed probability of a design basis earthquake and the associated small and probably zero fission product release and by comparing the combination of the proposed limit-line [2, 3], it is concluded that the seismic risk is negligible with respect to the reactor building. This linear analysis is limited to the estimation of the probability of cracking. The probability of building collapse should be substantially lower but a nonlinear analysis would be required for substantiation.

While the analysis suggests that seismic design practice is very conservative at least for the reactor building, further analyses of other components and parameters are necessary, e.g., piping, effects of nonlinear behavior, site characteristics, and foundation stiffness, in order to reach a firm conclusion. Of even greater importance would be a systematic analysis of the probability of occurrence versus seismic intensity for several reactor sites. A rational seismic design criterion would be to establish for each proposed reactor site the seismic intensity associated with a specified probability, e.g., 10^{-4} per year. The reactor building and other components would then be designed to a code for which prior analysis had demonstrated a low enough probability of damage. Unrealistically conservative seismic criteria lead to unnecessarily high equipment costs and exclude otherwise acceptable sites for nuclear power plants. Such a practice is not in the overall public interest.

Acknowledgements

This work was performed by Holmes & Narver, Inc., under contract to the General Electric Company's Nuclear Energy Division. The authors wish to thank Messrs. B.J. Garrick, R.P. Kennedy, O.C. Baldonado, and R.A. Williamson of Holmes & Narver and Messrs. J.E. Love, S.A. Wilson, J.R. Sasso, R.I. Post, J.J. Peterson, and P.P. Stancavage of the General Electric Company for their contributions and encouragement during the course of the study.

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Table I

Summary of the Properties of Earthquake Accelerograms

Earthquake	Type	Duration, Seconds: $T^{(1)}$ Record	Duration, Seconds: T Response	Maximum Acceleration(2) Ft/Sec^2	Maximum Velocity(2) Ft/Sec	Maximum Displacement(2) Ft	Richter Magnitude	Spectral Intensity(3)
B-1	Artificial	50 (4, 11, 35)	18	(0.375 g) 12.077 (9.35 sec)	1.472 (15.8 sec)	.966 (9.325 sec)		3.115
El Centro N-S 5/18/40	Real	30	8	(0.319 g) 10.286 (2.005 sec)	1.376 (2.11 sec)	.93 (2.575 sec)	7.0	2.71
C-1	Artificial	12 (2, 2, 8)	8	(0.068 g) 2.201 (2.05 sec)	.228 (2.975 sec)	.052 (4.425 sec)		0.552
D-1	Artificial	10 (2, .5, 7.5)	7	(0.485 g) 15.617 (2.675 sec)	.898 (2.325 sec)	.159 (3.100 sec)		

NOTES: (1) The three figures within parentheses refer to the duration of time in seconds of the initial buildup, constant, and decay phases of the envelop function used in the generation of design earthquakes.

(2) The last figures shown within the parentheses indicate the time of occurrence of maximum acceleration, velocity, and displacement.

(3) Spectral intensity is computed for 20 percent damping. Pseudovelocity spectra was used for design earthquakes while velocity spectra was used for the El Centro earthquake.

Table II

Comparison of Maximum Responses of Models A and B

Response	EARTHQUAKE: El Centro, Model A	El Centro, Model B	C-1, Model A	C-1, Model B
Displacement, ft	.097	.112	.0218	.0217
Acceleration, ft/sec^2	52.1	63.8	11.2	10.4
Shear, kips	2.17×10^5	2.36×10^5	4.97×10^4	3.81×10^4
Moment, ft - kips	2.06×10^7	2.27×10^7	4.56×10^6	3.53×10^6

Table III

Values of Absolute Maximum Shear Forces (Kips) for
Scaled 0.2 g Earthquakes

β \ E	2,300 ksi	3,500 ksi	4,800 ksi
El Centro Earthquake, 1940 N-S			
2%	46,142	69,232	68,882
5%	34,972	53,066	49,104
10%	33,608	38,782	38,902
Design Earthquake, B-1			
2%	62,598	68,264	55,048
5%	44,242	50,914	42,638
10%	32,392	38,112	35,152
Design Earthquake, C-1			
2%	56,606	47,004	58,250
5%	44,812	45,194	49,710
10%	39,522	39,756	43,018
Design Earthquake, D-1			
2%	39,152	55,974	50,894
5%	36,564	49,528	43,978
10%	33.096	40,476	39,884

Table IV

Values of Absolute Maximum Internal Moments (Foot Kips)
for Scaled 0.2 g Earthquakes

β \ E	2,300 ksi	3,500 ksi	4,800 ksi
El Centro Earthquake, 1940 N-S			
2%	4,655,200	7,027,200	6,910,600
5%	3,460,400	5,248,400	4,694,200
10%	3,003,600	3,780,800	3,510,800
Design Earthquake, B-1			
2%	5,692,000	6,372,000	5,092,200
5%	4,010,400	4,770,000	3,832,600
10%	3,096,200	3,555,400	3,131,000
Design Earthquake, C-1			
2%	5,335,000	4,294,200	5,905,800
5%	4,232,800	4,165,400	4,644,200
10%	3,603,400	3,635,800	3,952,800
Design Earthquake, D-1			
2%	3,308,800	5,383,400	4,779,200
5%	3,311,800	4,510,800	4,234,200
10%	3,097,000	3,711,200	3,772,800

Table V

Shear Stress (psi) in A Critical Component for 0.2 g Earthquakes

β % \ E ksi	2,300	2,900	3,500	4,200	4,800
El Centro Earthquake, 1940 N-S					
2	186		278		276
3.5		192		248	
5	140		214		196
7.5		164		187	
10	134		156		156
Design Earthquake, B-1					
2	250		274		220
3.5		206		186	
5	178		204		170
7.5		166		156	
10	130		152		140
Design Earthquake, C-1					
2	230		188		236
3.5		238		174	
5	180		182		200
7.5		174		170	
10	158		178		174
Design Earthquake, D-1					
2	156		224		204
3.5		212		224	
5	146		198		176
7.5		160		182	
10	132		162		160

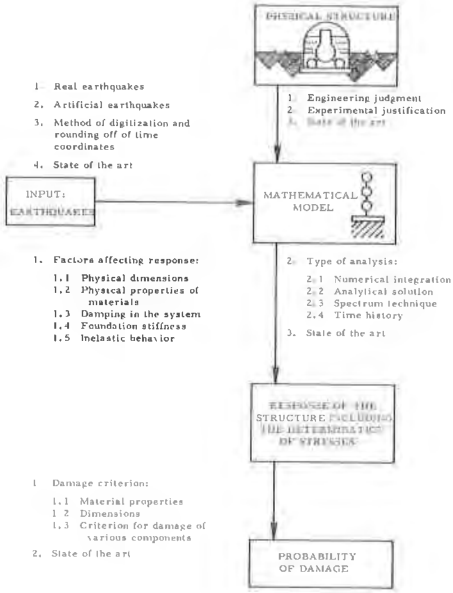


Figure 1. Factors affecting the probability of damage of reactor containment structures.

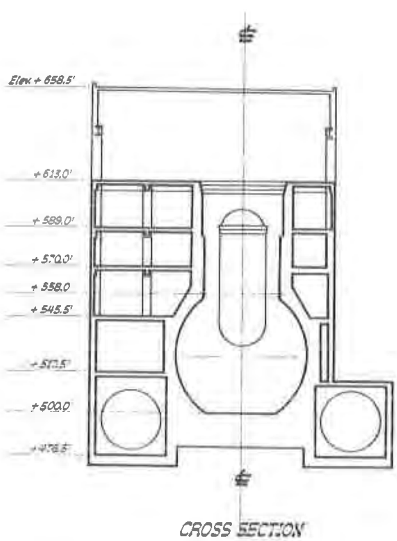


Figure 2. Sectional elevation of reactor building.

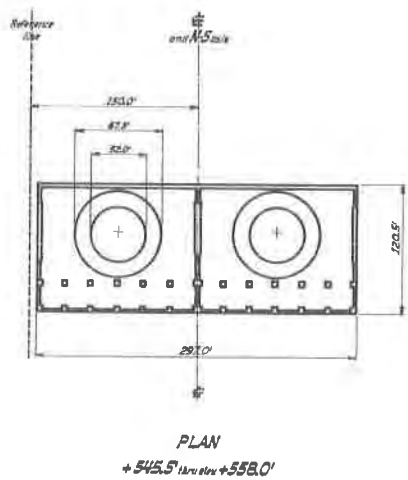


Figure 3. Sectional plans of reactor building properties about [illegible]

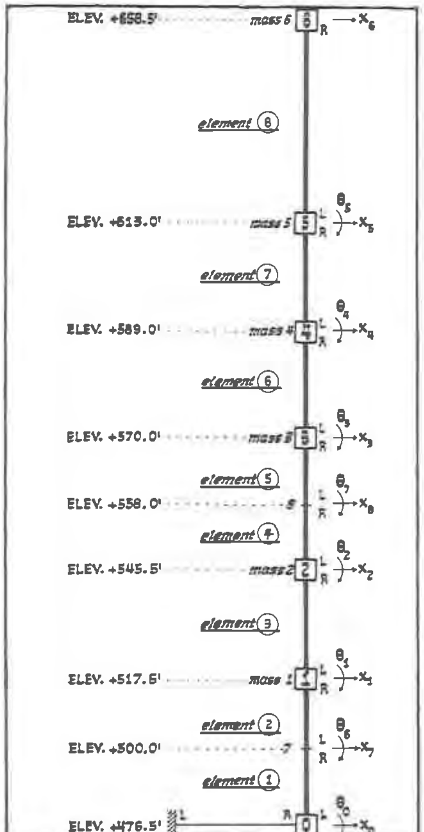
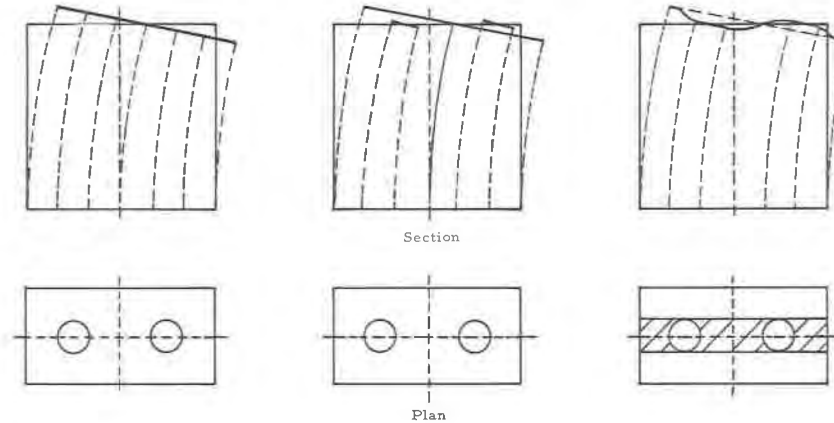


Figure 4. Mathematical model for reactor building.



Model A showing composite action of shield and walls.

Model B showing independent action of shield and walls.

Model C showing independent action of shield and wall including the participation of floors.

Figure 5. Variation of mathematical models.

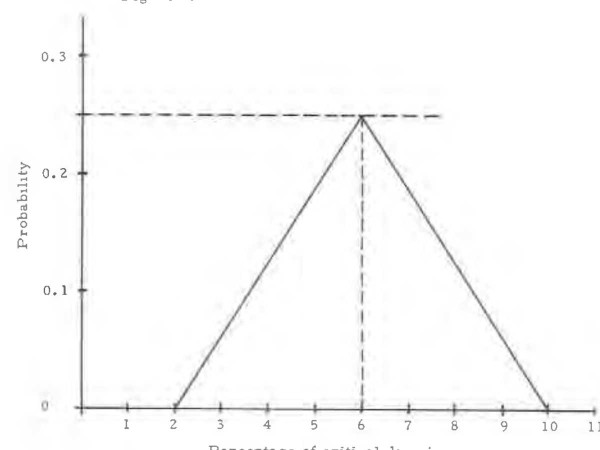
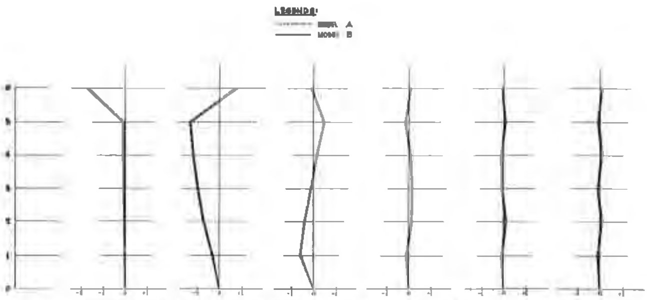


Figure 6. Probability density function for damping.

K 1/3



MODE	1		2		3		4		5		6	
E (10^6 psi)	3.00		3.00		3.00		3.00		3.00		3.00	
MODEL	A	B	A	B	A	B	A	B	A	B	A	B
PERIOD (sec)	0.286	0.297	0.167	0.176	0.068	0.068	0.041	0.048	0.028	0.029	0.020	0.020
FREQUENCY (cps)	3.576	3.366	5.984	5.681	15.780	15.380	24.377	23.755	35.271	34.925	50.086	49.566

Figure 7. Dynamic characteristics of mathematical Models A and B.

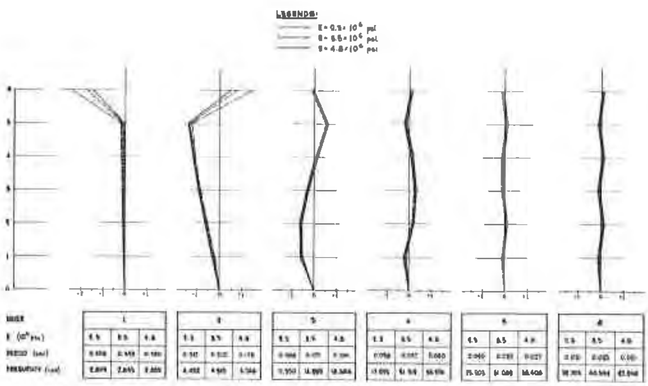


Figure 8. Dynamic characteristics of mathematical models for parametric study of response.

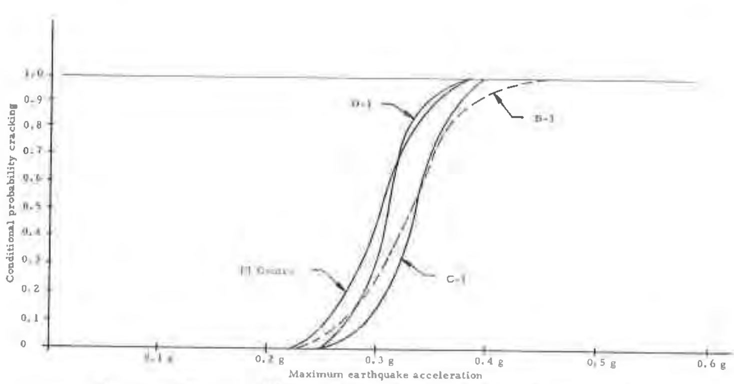


Figure 9 Conditional probability of cracking as a function of seismic intensity for four accelerograms.