ON SEISMIC RISK ANALYSIS OF NUCLEAR PLANTS SAFETY SYSTEMS

D. L. ANDERSON

Acres Consulting Services Limited, 5259 Dorchester Road, Niagara Falls, Ontario, L2E 6Wl, on leave from Department of Civil Engineering, University of British Columbia, Vancouver, V6T 1W5.

R. G. CHARLWOOD

Acres Consulting Services Limited, 5259 Dorchester Road, Niagara Falls, Ontario, L2E 6Wl.

C. B. CHAPMAN

Acres Consulting Services Limited, 5259 Dorchester Road, Niagara Falls, Ontario, L2E 6Wl.

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ABSTRACT

The recent USAEC study (WASH-1400), using a simplified analysis, has estimated the probability of earthquake initiated accidents of nuclear plants to be in the range 10^{-5} to 10^{-8} per year, and has concluded that these are sufficiently small compared to other causes as to be safely ignored in assessing overall plant safety.

An event tree model is presented in this paper to assess the failure probabilities from seismic events of single and multiple component safety systems. The ground motion, response spectra and failure criteria are treated as probabilistic quantities. The model is applied to a Canadian site and failure probabilities are predicted for several levels of design. The sensitivity of uncertaintities in the input data on the failure probabilities is also investigated.

The results show the importance of considering common mode failures when determining seismically initiated system failure probabilities. Also, in assessing system failure probabilities of the order of 10^{-6} it is shown that the input with the highest sensitivity is the probability distribution of large magnitude earthquakes and the component failure probabilities.

It is recommended that the design basis earthquake be determined from the desired system failure probability level, and that more effort should be placed on estimating the probability of occurrence of large magnitude earthquakes and in determining component failure probabilities.

INTRODUCTION

In the design of nuclear power plants one of the main design decisions is the level of seismic activity that must be considered for the design of the safety systems. This is usually referred to as the DBE (design basis earthquake) or SSE (safe shutdown earthquake) and specified by a peak horizontal ground acceleration. The purpose of this paper is to present the results of a study of the implications and reliability of this design decision through a quantitative analysis of seismic risk. A three components-in-series system is considered to represent a main steam line, an emergency core cooling system and a containment structure, as a typical nuclear plant safety system.

The recent U.S. Reactor Safety Study (WASH-1400-D 1974) concluded that the probability of an earthquake initiated accident was 10^{-5} to 10^{-8} per year which was several orders of magnitude less than other causes and thus did not need to be considered in evaluating overall accident probabilities. The study reported here assesses the probability of an earthquake initiated common mode failure of a system at a representative Canadian site, to ascertain whether the probabilities are of the order of 10^{-5} or 10^{-6}, and identifies the relative importance of the factors governing these failure probabilities.

Current practice in earthquake analysis explicity recognises the stochastic nature of earthquake occurrence, but gives only implicit recognition to the stochastic nature of the response spectra and failure criteria, and ignores the relationship between seismic risk and other types of risk. The DBE may be established by multiplying the predicted 100 year return period acceleration by a factor, or by determining the site acceleration due to an earthquake of a magnitude larger than any recorded at the nearest center of recognizeable earthquake activity. The response spectra used is generally that specified by Newmark Blume and Kapur (1973) and corresponds to the smoothed 84 percentile response spectra curve. The structures and components are then designed to resist the acceleration forces elastically.

The present paper discusses an event tree model which has been used to assess the probability of an earthquake initiated system failure, and estimate the social and economic consequences of such failures. Sensitivity analyses were performed on the input data and the relative effects of various design levels were determined for a Canadian site for a nuclear plant. The analysis uses the EMR predicted distribution of ground acceleration, a probability distribution of response spectra, and a component failure probability based on the ratio of component acceleration to design acceleration which accounts for the reserve of strength of a structure beyond the yield level. The component failure probabilities are combined, recognising the common mode aspect of earthquake initiated accidents, for a three component in series system.

ANALYSIS MODEL

A flow chart of the analysis to assess the implications of seismic initiated failures of a nuclear plant safety system is given in Fig. 1 The analysis has been broken into two levels; level I is concerned with the prediction of site system failure probabilities caused by seismic events, and is treated in some detail in this paper; while level II which is concerned with the assessment of the social and economic consequences of a failure, is discussed in only general terms.

An event tree of the level I analysis is given in Fig. 2. The various stages of the tree are related to the boxes of Fig. 1 while the definition of the various probabilities and symbols are given in the following section on model application.

Box number 1 represents an analysis of earthquake occurrence in terms of a probability distribution of energy released at discrete epicenters, or alternatively, surface area or crustal volume (Dalal 1973, Esteva 1969). The energy is usually related to earthquake magnitude, however, a more useful relation would also include duration and frequency content.

Box number 2 represents the transmission of energy release to a plant site in probabilistic terms, and the integration of all energy releases to produce the predicted site seismic activity. The most common present approach is to use a deterministic distance attenuation relation to translate the magnitude of epicenter earthquakes into horizontal ground accelerations at the site.

The larger box of dotted lines encompassing boxes 1 and 2 and denoted by A, represents the estimate of seismic activity at the site in probailistic terms. In Canada the EMR earthquake prediction service (EMR Earthquake Probability Analysis) provides an estimate of seismic activity in terms of expected peak horizontal ground accelerations and associated probabilities of occurrence. The EMR method differs slightly from that described above as it determines the peak acceleration expected at the site from all known earthquakes, and then determines the probability of occurrence based on the time period over which records have been maintained. In the model application the EMR prediction method will be used.

Box number 3 represents the analysis that determines plant structural response conditional upon specific levels of site seismic activity, with the associated probabilities being considered later, and thus only a dotted line is used to connect boxes 2 and 3. This analysis could range from structural analysis of predicted site ground motions to static analysis using response spectra.

Box number 4 predicts the response of components or equipment attached to the structure.

The larger dotted box B encompassing boxes 3 and 4 represents the total analysis required to predict structure and equipment forces, velocities and other design variables conditional upon a specified value of site seismic activity. In the model application only components subjected to the site activity and represented by box 3 analysis will be considered. The analysis uses a probabilistic model of the response spectra and for a specified value of ground acceleration predicts the probability distribution of peak component acceleration.

Box number 5 predicts the probability of direct component failure, or the probability of various levels of damage, for different levels of seismic activity. Box number 6 represents other or indirect sources of seismically induced component failure, such as operator error during an earthquake that leads to failure of the component. The larger box C enclosing boxes 5 and 6 then represents the analysis required to predict the probability of component failures for the various levels of seismic response predicted by box B.

Box C contains the results of the design decisions. The predicted probability of component failure is dependent upon the level of the DBE, the care in design, the choice of materials, the operator requirements and many other factors which are designed into the system. In the model application only box 5 will be considered, and the component failure probability for a given design level will be assumed to depend upon the peak component acceleration.

The three large boxes A, B and C contain all the information needed to compute failure probabilities. The remaining boxes in level I are computational boxes using the above information. Box number 7 takes the component failure probabilities from box C for each level of seismic response from box B, and combines the probabilities to form the system failure probability. The system failure probabilities must be evaluated at this level in the model because the failures of the components making up the system have a common cause.

Box number 8 evaluates the system failure probability for each level of ground or site seismic activity by convoluting the probabilities of seismic response from box B with the failure probabilities of box 7. Box number 9 then evaluates the site system failure probability by combining the probabilities of site seismic activity from box A with the probabilities of system failure from box 8. At this stage the site system failure probability can be used as input to level II of the model or, as indicated by the dotted line from box 9 to box C, used to alter the design decisions that govern the component failure probabilities of box C.

The level II analysis while beyond the scope of this paper, is included to show how in general terms the social and/or economic consequences of a seismically induced system failure may be assessed. Similar models were applied to accidents initiated by non seismic events (WASH-1400-D, 1974), however, they did not include the effect of earthquakes on the models and would have to be modified to be applicable to assessing the consequences of earthquake initiated accidents. For example, a large earthquake would seriously hamper emergency services and could make communications and evacuation of an area much less effective. The evacuation model used (WASH-1400-D, 1974) assumes that for normal conditions half the people in an area can be evacuated in a two hour period (evacuation half time of two hours). Intuitively it is felt that a large earthquake would produce a significant increase in the evacuation half time.

The ultimate use of the level II analysis would be to use the predicted levels of social and economic cost to establish the DBE as indicated in Fig. 1 by the dotted line feeding back from the final box to box C.

APPLICATION OF MODEL

For demonstration purposes, the model is applied to a hypothetical site that is representative of Eastern Canadian sites. The EMR prediction of ground acceleration probability is used, along with the distribution of dynamic acceleration amplification factors of typical earthquakes (Newmark, Blume and Kapur 1973). The failure probability of a single component system with an assumed failure curve is calculated, and is then extended to the failure probability of a three component-in-series system.

Earthquake Prediction

The EMR earthquake prediction for the site is listed in Table 1. The equation of the best fit log normal probability distribution relation is given by

[1] $$\ln(A) = -2.26 - 0.83 \ln(-\ln P(A))$$

where A = peak horizontal ground acceleration in per cent of gravity

P(A) = probability that A will not be exceeded in one year.

With this relation the probability that an earthquake with peak acceleration falling in some interval can be determined. This is given in Table 1 also, where for

TABLE 1

ACCELERATION PROBABILITIES
HYPOTHETICAL SITE

A	/- P(A)	i	$\bar{A}_i$	$P(\bar{A}_i)$
% g			% g	
		0	0	.667
0	.333			
		1	0.5	.233
1	.100			
		2	1.5	.067
2	.033			
		3	2.5	.013
3	.020			
		4	4	.010
5	.010			
		5	7	.005
9	.005			
		6	10	.002
12	.003			
		7	20	.002
33	.001			
		8	42	.0005
57	.0005			
		9	122*	.0005
				1.0000

*Assuming A_{max} = 200 % g. For
A_{max} = 100 % g, $\bar{A}_9$ = 87 % g.
A_{max} = 50 % g, $\bar{A}_9$ = 50 % g.

example $P(\bar{A}_7)$ is the annual probability that an earthquake with a peak acceleration of $12 \le A \le 33\%g$ will occur. Thus

$$P(\bar{A}_7) = P(12) - P(33)$$
$$= .003 - .001 = .002$$

$\bar{A}_i$ is taken as the mean value of the peak acceleration in the i^{th} interval. In the last interval, taken here as i = 9, the average value is dependent on an assumed maximum cutoff acceleration A_{max}. The use of a cutoff acceleration affords a means of evaluating the importance of the tail of the acceleration distribution curve. It is known that the data base for the probability acceleration relation given by Eqn 1 is not adequate for extrapolation to values of P(A) approaching one, and by imposing a cutoff the distribution is effectively changed such that the probability of exceeding the A_{max} acceleration is so small as to be negligible.

Response Spectra

The use of a response spectra approach to determine structural and equipment response to earthquakes has been quite widely accepted. Since earthquakes have different frequency contents the response spectra differs widely for various earthquakes, and the deterministic approach has been to use a response spectra that is an envelope of near maximum responses of many different earthquakes. The approach taken here is that the response spectra is probabilistic and that a choice of a specific level of spectra is not required.

Newmark Blume and Kapur (1973) have reported on the results of an investigation where the response spectra was calculated for many different earthquakes (mainly California earthquakes) normalized to the same 1.0g peak acceleration. At any one frequency the response to the various earthquakes measured by the dynamic amplification factor (DAF), was observed to fit a normal or a log normal distribution. Thus for any single frequency, the dynamic amplification of the ground acceleration can be expressed in a probabilistic manner.

Fig. 3 shows the DAF as presented by Newmark Blume and Kapur (1973), for a structure with 2 per cent critical damping. The three smoothed curves represent the 50, 84.1, and 97.7 per cent probabilities,based on a log normal fit, that the DAF of any earthquake will not exceed that given by the curves.

For example considering a period of 0.5 seconds, the dynamic amplification factors are 1.9, 3.0 and 4.7 for the 50, 84.1 and 97.7 per cent probability levels respectively. Fig. 4 plots the DAF against these probabilities. From the curve it is possible to determine average DAF values in probability intervals in order that combined probabilities can be computed. The probability intervals used are shown in Fig. 4 along with the average value of the DAF in the interval, denoted by $\overline{DAF}_j$.

Table 2 lists the probability intervals and average values of the $\overline{DAF}_j$ in a format similar to the ground acceleration presented in Table 1.

Component Failure Probabilities

At this time very little information is available on the probabilities of structural or equipment failure, or damage, as a function of seismic response. However, to perform the analysis in this study, such distributions were essential and consequently a family of curves was devised which, it is believed, cover the range of likely distributions. In the following work the effect of different component failure probability curves on the accident probabilities are analyzed.

Fig. 5 shows what we have termed the "basic" component failure probability curve. The abscissa is the ratio of the peak acceleration of a component due to an earthquake, Ca, to the design peak acceleration of the component, Da. P(cf) is the probability that

TABLE 2

DAF PROBABILITIES
PERIOD = 0.5 SECONDS
DAMPING RATIO = 2 %

DAF	P($\overline{DAF}$)	j	$\overline{DAF}_j$	P($\overline{DAF}_j$)
0	0			
		1	1.5	.6
2.1	.6			
		2	2.4	.2
2.8	.8			
		3	3.1	.1
3.4	.9			
		4	3.7	.05
4.0	.95			
		5	5.0	.05
∞	1.00			

the component will fail.

The curve is based on the following assumptions and observations:

1. The component has been designed to nuclear standards and remains elastic at the design acceleration.

2. The results of an investigation into earthquake related failure probabilities (WASH-1400-D, 1974) estimated that there could be a 0.1 probability of failure for components subjected to the DBE, thus it is assumed that

$$P(cf) = 0.1 \text{ when } \frac{Ca}{Da} = 1$$

3. Most building code earthquake design requirements will allow a ductile structure to be very severely damaged if the earthquake induced stresses, calculated using an elastic analysis, are about four times the yield stress. If this design basis represents the performance of structures or components then it indicates that at accelerations of four times elastic yield design acceleration the probability of failure is high. Consequently in the analysis it was assumed that

$$P(cf) = 0.9 \text{ when } \frac{Ca}{Da} = 4$$

It is recognised that many structures or pieces of equipment may have a more severe failure probability curve, especially if they are non-ductile as was much of the electrical equipment that suffered damage and failure in the San Fernando 1971 earthquake. On the other hand many structures, with careful and proper attention paid to details to improve the ductility, may have less severe failure probability curves. It is felt that Fig. 5 represents a realistic basic failure probability curve for nuclear plant components. Variations on this curve are considered later.

Originally a smooth curve asymptotic to P = 0 and P = 1 was drawn through the two points described above. However, there was found to be very little difference in the failure probabilities due to earthquake between the smooth curve and the straight lines shown in Fig. 5, and so the latter was adopted for ease of calculation.

Single Component Seismic Failure Probability

For single component failure probabilities the response spectra probabilities as given in Table 2, and the component failure probabilities of Figure 5, are combined for the various ground accelerations to give the failure probability of a single component conditional upon the ground acceleration. The ground accelerations considered are those given in Table 1.

As an illustrative example consider a component design acceleration Da = .30g, which would be a low design level for the hypothetical site which has a 100 year acceleration of .05g.

Following the event tree calculation sequence shown in Fig. 2, Table 3 shows the calculations to determine the failure probability for the interval i = 7 of the earthquake ground acceleration ($\bar{A}_7$ = .20g). This calculation gives the result $P_i(cf) = P_7(cf) = .2115$. That is, if the component is subjected to a .20g ground acceleration, then there is a .2115 probability of failure.

Table 4 lists the values of all $P_i(cf)$ from calculations similar to those performed in Table 3. These results are then combined with the $P(\bar{A}_i)$ from Table 1 to give the probability of failure in each ground acceleration interval, and then summed to give the site failure probability.

TABLE 3

COMPONENT FAILURE PROBABILITY FOR A SPECIFIC LEVEL OF GROUND ACCELERATION

i	A_i %g	j	$\overline{DAF}_j$	$P(\overline{DAF}_j)$	Ca %g	$\frac{Ca}{Da}$	$P_{ij}(cf)$	$P(\overline{DAF}_j)P_{ij}(cf)$
7	20	1	1.5	.6	30	1.0	.10	.060
		2	2.4	.2	48	1.6	.26	.052
7	20	3	3.1	.1	62	2.1	.39	.039
		4	3.7	.05	74	2.8	.49	.0245
		5	5.0	.05	100	3.3	.72	.036
							$P_i(cf)$ =	.2115

TABLE 4

SITE COMPONENT
FAILURE PROBABILITY

i	$P(\overline{A}_i)$	$\overline{A}_i$ % g	$P_i(cf)$	$P(\overline{A}_i)P_i(cf)$
0	.667	0	0	0
1	.233	.5	0	0
2	.067	1.5	0	0
3	.013	2.5	0	0
4	.010	4	.0005	.000005
5	.005	7	.0135	.0000675
6	.002	10	.0430	.0000860
7	.002	20	.2115	.0004230
8	.0005	42	.579	.0002895
9	.0005	122	1.0	.0005
	1.000		P(cf) =	.001371
			R_c =	729 years

The results shown in Table 4 predict a site failure probability of 0.00137 per year, or a return period of 729 years. By contrast the return period for a peak ground acceleration of .10g is 244 years and for .16 g is 424 years. These latter two accelerations and return periods are given for comparison since for a design acceleration Da = .30g, .10g would represent the DBE if the 84 percentile DAF was used, and .16g would represent the DBE if the mean DAF was used. It is seen that for a reasonable component failure probability curve the failure return period is much longer than that of the DBE return period; and furthermore that the failure return period is not dependent upon some arbitrary choice of the DAF probability level curve.

System Seismic Failure Probabilities

In nuclear plants the safety system is designed so that more than one component must fail before an accident occurs, where accident refers to some relatively serious event that would cause shutdown for an extended period or a leak. In this section a three component system is considered and the resulting failure probabilities are calculated.

If each component had an identical failure probability, say $P(cf) = 1.37 \times 10^{-3}$ as given in Table 4, and if the causes of failure were independent, then the probability of failure of a three component system would be $P(sf) = \left[P(cf)\right]^3 = 2.58 \times 10^{-9}$. However, if the causes of failure are not independent but due to a common cause, such as an earthquake, the system failure probability must be computed by convoluting the component failure probability distributions. Table 5 gives the combined failure probabilities for three identical components, following the analysis shown by Fig. 2, for one interval of ground acceleration. This table is similar to Table 3 except that it now applies to three components rather than one. The three component failure probabilities are then combined with the ground acceleration probabilities to give the site failure probability for the system P(sf). This is done in Table 6 and the result is a system failure probability of 7.28×10^{-4}.

The large difference in the failure probabilities between common and independent causes, and the surprisingly small decrease in the system failure probability from the component failure probability occurs because, at the higher acceleration and DAF levels the component failure probability becomes large, and so the system probability remains large despite being the cube of the component probability.

The previous calculation for system probability assumed that the three components all had the

TABLE 5

SYSTEM FAILURE PROBABILITY FOR A SPECIFIC LEVEL OF GROUND ACCELERATION

i	$\bar{A}_i$ %g	j	$\overline{DAF}_j$	$P(\overline{DAF}_j)$	Ca %g	$\frac{Ca}{Da}$	$P_{ij}(cf)$	$P_{ij}(sf)$	$P(\overline{DAF}_j)P_{ij}(sf)$
7	20	1	1.5	.6	30	1	.10	.001	.0006
		2	2.4	.2	48	1.6	.26	.01758	.003515
		3	3.1	.1	62	2.1	.39	.06283	.005932
		4	3.7	.05	74	2.8	.49	.1176	.005882
		5	5.0	.05	100	3.3	.72	.3732	.018662

$P_i(sf) = 3.4592 \times 10^{-2}$

TABLE 6

SITE SYSTEM FAILURE PROBABILITY

i	$P(\bar{A}_i)$	$\bar{A}_i$ %g	$P_i(sf)$	$P(\bar{A}_i)P_i(sf)$
0	.667	0	0	0
1	.233	.5	0	0
2	.067	1.5	0	0
3	.013	2.5	0	0
4	.010	4	5×10^{-8}	0.000×10^{-4}
5	.005	7	1.8225×10^{-4}	0.009×10^{-4}
6	.002	10	1.4605×10^{-3}	0.029×10^{-4}
7	.002	20	3.4592×10^{-2}	0.692×10^{-4}
8	.0005	42	3.1042×10^{-1}	1.552×10^{-4}
9	.0005	122	1.00	5.000×10^{-4}
			$P(sf) =$	7.282×10^{-4}
			$R_s =$	1373

same natural frequency and therefore for a given earthquake they would all have the same DAF in the same probability interval. However, if the three components have different natural periods then they will have different DAF's in the same probability interval. This latter important variation occurs because the DAF curves of Fig. 1 represent the probabilities of many different earthquakes, and the chance that one particular earthquake would produce a high DAF over the whole range of periods is small. This idea is figuratively shown in Fig. 5 where the 97.7 percentile DAF probability curve is drawn along with typical DAF curves for three different earthquakes whose DAF's were the highest at the natural period of the three components. Thus if the three components were subjected to earthquake A, then component 1 would have a large DAF but components 2 and 3 would have small DAF's.

To demonstrate the importance of this effect the system failure probability is calculated assuming that the three components all have the same distribution and magnitude of DAF, but that they have different natural periods and so would be subjected to different DAF's in the same probability intervals. To do this the $P(\overline{DAF}_j)$ was broken into 20 equal segments and the $\overline{DAF}_j$ for each segment have been arranged so that one component has a high $\overline{DAF}_j$, one a medium and the third a low $\overline{DAF}_j$. This is shown in Table 7.

Table 8 combines the failure probabilities for a specific $\overline{A}_i$ and Table 9 gives the site failure probability for the system.

For this particular choice of components it is seen that $P_9(sf) = 1$ and that $P(\overline{A}_9)\ P_9(sf)$ is by far the largest contributor to P(sf) (Table 9). This means that even for the smallest $\overline{DAF}_j$ the i = 9 component acceleration is large enough to cause a component failure probability of one, thus no matter how the $\overline{DAF}_j$'s are combined the P(sf) will not change very much. If however,the design acceleration is high enough so that $\overline{A}_9$ will not cause failure for all the $\overline{DAF}_j$, the method of combining the component probabilities will make a much greater difference. This is shown in Table 10 where the system return period is calculated for four different design levels and for the two ways of combining component failures.

TABLE 7

ASSUMED $\overline{DAF}_j$ VALUES OF THREE COMPONENTS WITH SEPARATED PERIODS

Component		1	2	3
j	$P(\overline{DAF}_j)$	$\overline{DAF}_j^{(1)}$	$\overline{DAF}_j^{(2)}$	$\overline{DAF}_j^{(3)}$
1	.05	1.5	1.5	5.0
2	.05	1.5	1.5	3.7
3	.05	1.5	1.5	3.1
4	.05	1.5	1.5	3.1
5	.05	1.5	1.5	2.4
6	.05	1.5	1.5	2.4
7	.05	1.5	2.4	2.4
8	.05	1.5	2.4	2.4
9	.05	1.5	3.1	1.5
10	.05	1.5	3.7	1.5
11	.05	1.5	5.0	1.5
12	.05	1.5	3.1	1.5
13	.05	2.4	2.4	1.5
14	.05	2.4	2.4	1.5
15	.05	2.4	1.5	1.5
16	.05	2.4	1.5	1.5
17	.05	3.1	1.5	1.5
18	.05	3.1	1.5	1.5
19	.05	5.0	1.5	1.5
20	.05	5.0	1.5	1.5

TABLE 8

SYSTEM FAILURE PROBABILITIES $P_{ij}(sf)$
FOR A SPECIFIC LEVEL OF GROUND
ACCELERATION - SEPARATED PERIODS

i	$\bar{A}_i$	j	$\overline{DAF}_j^{(1)}$	$\frac{Ca}{Da}$	$P_{ij}^{(1)}(cf)$	$\overline{DAF}_j^{(2)}$	$P_{ij}^{(2)}(cf)$	$\overline{DAF}_j^{(3)}$	$P_{ij}^{(3)}(cf)$	$P_{ij}(sf)$
7	20	1	1.5	1.0	.10	1.5	.10	5.0	.72	.0072
		2	1.5	1.0	.10	1.5	.10	3.7	.49	.0049
		3	1.5	1.0	.10	1.5	.10	3.1	.39	.0039
		4	1.5	1.0	.10	1.5	.10	2.4	.39	.0039
		5	1.5	1.0	.10	1.5	.10	2.4	.76	.0026
		6	1.5	1.0	.10	1.5	.10	2.4	.26	.0026
		7	1.5	1.0	.10	2.4	.26	2.4	.26	.00676
		8	1.5	1.0	.10	2.4	.26	1.5	.26	.00676
		9	1.5	1.0	.10	3.1	.39	1.5	.10	.0039
		10	1.5	1.0	.10	3.7	.49	1.5	.10	.0049
		11	1.5	1.0	.10	5.0	.72	1.5	.10	.0072
		12	1.5	1.0	.10	3.1	.39	1.5	.10	.0039

Table 8 (cont'd)
System Failure Probabilities $P_{ij}(sf)$
For a Specific Level of Ground
Acceleration - Separated Periods

i	$\overline{A}_i$	j	$\overline{DAF}_j^{(1)}$	$\frac{Ca}{Da}$	$P_{ij}^{(1)}$ (cf)	$\overline{DAF}_j^{(2)}$	$P_{ij}^{(2)}$ (cf)	$\overline{DAF}_j^{(3)}$	$P_{ij}^{(3)}$ (cf)	$P_{ij}(sf)$
		13	2.4	1.60	.26	2.4	.26	1.5	.10	.00676
		14	2.4	1.60	.26	2.4	.26	1.5	.10	.00676
		15	2.4	1.60	.26	1.5	.10	1.5	.10	.0026
		16	2.4	1.60	.26	1.5	.10	1.5	.10	.0026
		17	3.1	2.07	.39	1.5	.10	1.5	.10	.0039
		18	3.1	2.07	.39	1.5	.10	1.5	.10	.0039
		19	3.7	2.47	.49	1.5	.10	1.5	.10	.0049
		20	5.0	3.33	.72	1.5	.10	1.5	.10	.0072
										.09714

$P(\overline{DAF}_j)$ = .05 for each interval

$P_i(sf) = P(\overline{DAF}_j)\ \Sigma_j\ P_{ij}(sf) = 0.05\ (.09714) = .004857$

TABLE 9

SITE SYSTEM FAILURE PROBABILITY
SEPARATED PERIODS

i	$P(\overline{A}_i)$	$\overline{A}_i$ %g	$P_i(sf)$	$P(\overline{A}_i)P_i(sf)$
0	.667	0	0	0
1	.233	.5	0	0
2	.067	1.5	0	0
3	.013	2.5	0	0
4	.010	4	0	0
5	.005	7	0	0
6	.002	10	0	0
7	.002	20	.004857	0.097×10^{-4}
8	.0005	42	.1535765	0.773×10^{-4}
9	.0005	122	1.00	5.000×10^{-4}

$P(sf) = 5.870 \times 10^{-4}$

R_s = 1704 years

TABLE 10

RETURN PERIODS

Design Acceleration	.30 g	.45 g	.60 g	.75 g
Single Component	729	1171	1667	2221
Three-Component System (same periods)	1373	2020	3261	4771
Three-Component System (separated periods)	1704	2284	4217	7650

SENSITIVITY ANALYSIS

In the previous sections, failure probabilities were computed assuming that the probabilities of ground acceleration, response spectra values and component failures were certainties. It has been suggested that the EMR earthquake probability estimates are subject to an uncertainty factor of two, i.e. the ground acceleration estimates could be double or half those quoted. Similarly, the response spectra values are dependent on local site conditions and structural damping and could conceivably also have an uncertainty factor of two. Finally, the component failure probability curve is open for wide argument as to its definition, shape, and magnitude.

This section looks at the changes in the computed site failure probabilities due to the changes in the various inputs in order to assess their relative importance. The results indicate the most sensitive areas and identifies the effects of various design decisions.

To compare the influence of the uncertainties in the input, the site failure return period is calculated for a single component and for a three-component system with different natural frequencies, designed for two levels of design acceleration, Da = .30g and .75g.

These return periods are compared to the return periods calculated for the input described and used previously with $A_{max} = 2g$, which will be referred to as the basic input.

Earthquake Motion

Two parameters are considered in describing the uncertainty of the predicted peak horizontal ground acceleration. Firstly, A_{max}, the cutoff acceleration, is given values of 2g, 1g and 0.5g. Secondly, the acceleration in the acceleration probability relation is doubled while holding A_{max} constant.

Table 11 gives the return periods for the various ground accelerations. For designs with relatively low return periods (Da = .30g) the change in A_{max} from 2g to 1g has very little influence. This occurs because even at the cutoff level of 1g the probability of component failure is one for nearly all increments of the $\overline{DAF}_j$, and thus increasing A_{max} to 2g can not increase the failure probability significantly. Reducing A_{max} to .50g has a much larger influence, especially on the system. For stronger designs (Da = .75g) the variation in A_{max} has considerable influence on system failure return periods where decreasing A_{max} from 2g to 1g causes a fourfold increase in R_s, and decreasing A_{max} from 1g to 0.5g causes an order of magnitude increase.

Increasing the basic ground acceleration by a factor of 2 lowers the return periods by a factor slightly more than 2 for the low return periods, and up to 3 for the higher return periods. Thus, this effect is not as sensitive as the variations in the magnitude of A_{max}.

Response Spectra

The response spectra probability curves were derived from the response spectra of many normalized earthquakes, mainly Californian. Since Californian earthquakes may not be representative of earthquakes in other areas, and since the normalization procedure used may not be the most suitable for all earthquakes, considerable uncertainties in the response spectra values may exist because of site conditions, as well as from structural damping uncertainties. This is especially true at the low frequency end of the spectra and an uncertainty factor of two is used to assess the sensitivity.

The failure return periods are calculated for $\overline{DAF}_j$ values twice the basic values given in Table 2. The results are given in Table 12 and, as expected, are not too different from those given in Table 11 for two times the basic ground acceleration.

TABLE 11

RETURN PERIODS FOR UNCERTAINTIES IN GROUND ACCELERATION

Ground Acceleration	A_{max}	Da = .30g		Da = .75g	
		Component (Rc)	System (Rs)	Component (Rc)	System (Rs)
Basic	.5g	826	4444	4505	318735
Basic	1g	741	1901	2985	32582
Basic	2g	729	1704	2221	7650
2x Basic	.5g	340	1808	1810	87931
2x Basic	1g	305	811	1186	9837
2x Basic	2g	305	811	1006	2924

A second variable describing the response spectra distribution at any frequency is the standard deviation of the DAF. For components with very high frequencies all the $\overline{DAF}_j = 1$, and therefore a common limiting case of the distribution has zero standard deviation. To assess the influence of the distribution of the DAF, return periods were calculated assuming that all the $\overline{DAF}_j$ values were equal to the mean value of DAF. The results are given in Table 12 in the row labelled "mean amplification". For single component failures, the mean amplification overestimates the return period. This was expected, since the higher component failure probabilities occur for the higher $\overline{DAF}_j$ values, and so the importance of the $\overline{DAF}_j$ values are weighted toward the higher end. However, the difference is not great and for single component failures the distribution of the DAF is deemed less important than other uncertainties considered here.

For system failures, the use of the "mean amplification" underestimates the return periods. This occurs since the use of a "mean amplification" implicitly assumes the $\overline{DAF}_j$ for each component are the same for each probability interval. This is equivalent to the assumption that each component has the same natural frequency which was shown previously to predict low return periods. Thus the system return period predicted by the "mean amplification" is low

TABLE 12

RETURN PERIODS FOR UNCERTAINTIES IN RESPONSE SPECTRA

Response Spectra Amplification	Da = .30g		Da = .75g	
	Component (Rc)	System (Rs)	Component (Rc)	System (Rs)
Basic	729	1704	2221	7650
2x Basic	305	811	938	1869
Mean Amplification	901	1688	2564	6915

despite the fact that the component return periods are high. However, they are not too different from the basic values and so again the distribution of the DAF is deemed relatively unimportant. The value of the mean DAF is much more important than the distribution of the DAF.

Component Failure Probabilities

In the previous section a basic component failure probability curve was presented. However, the basis for assigning component failure probabilities is open to argument and so uncertainties in such a curve are considered. Three additional component failure probability curves are used to determine the sensitivity of the system failure return periods to these uncertainties. The three curves are shown in Fig. 7. The "strong" and "weak" curves are defined to represent, in some manner, a component twice as strong and half as strong as the basic component. The brittle curve was chosen to show the importance of the reserve or plastic strength represented by the other curves.

Table 13 lists the return periods for the four failure curves. The effect of "halving" the strength from the basic curve is not as great as doubling the ground acceleration or response spectra amplification. The change caused by assuming the brittle curve instead

TABLE 13

RETURN PERIODS FOR DIFFERENT
CRITERIA COMPONENT FAILURE

Component Failure Probability Curve	A_{max}	Da = .30 g		Da = .75 g	
		Component (Rc)	System (Rs)	Component (Rc)	System (Rs)
Brittle	2g	274	333	1250	2000
Weak		503	1372	1468	2528
Basic		729	1704	2221	7650
Strong		1162	3826	3992	51821
Basic	1g	741	1901	2985	32582
Brittle	.5g	274	333	1538	6667
Weak		511	1556	2415	43163
Basic		826	4444	4504	318735
Strong		1511	25272	8658	2367700

of the basic curve, is comparable to doubling the acceleration or amplification. At low design levels doubling the strength has more influence than changes in A_{max}, approximately the same effect as halving the ground acceleration or DAF for single components. For systems the influence of doubling the strength is greater than halving the ground acceleration or DAF. At high design levels, with long return periods, doubling the strength approximately doubles the return period for single components, but has nearly an order of magnitude effect for the three-component system. This is a greater effect on the system return period than reducing A_{max} from 2g to 1g (factor of 4) but not as great as reducing A_{max} from 1g to 0.5g (factor of 10).

DISCUSSION AND CONCLUSIONS

The probability of earthquake initiated failure of a single component and a three component in series nuclear plant safety system has been computed for a hypothetical site by a method that explicitly utilizes the stochastic nature of the earthquake, response spectra and component failure. Sensitivity analyses were performed on the probalistic input data and on possible design decisions to establish a quantitative basis for comparison of their relative contributions, and the effect of uncertaintities, on the probability predictions.

The major conclusions from the results of the analyses are:

1. The 'common mode' or 'common cause" of failure from a seismic event must be accounted for in determing system failure probabilities. A typical single component failure probability is 4.5×10^{-4} per annum. If the events initiating failure are independent, a three component in series system has a failure probability of 9.5×10^{-11}. However, considering earthquakes as a common mode the probability is shown to be 1.3×10^{-4}, a small improvement over the single component.

2. The sensitivity of system failure probabilities to uncertaintities in the input data increases as the design level is increased. For systems with a low probability of failure, i.e. with a 'strong design', the failure probability is most strongly affected by the high acceleration tail of the earthquake probability curve. The significance of the tail is shown by considering various 'maximum possible' ground accelerations, above which it is assumed that the probability of occurrence is sufficiently small to be considered negligible. For a specific design example the probability of system failure for maximum possible ground accelerations of 0.5g and 2.0g are predicted to be 3.1×10^{-6} and 1.3×10^{-4} respectively, a change by a factor of 40.

The second most important uncertaintity for strong designs is the reserve strength represented by the different failure criteria curves. 'Doubling' the strength decreases the system failure probability by a factor of 7.

Doubling the earthquake acceleration values in the probability acceleration relation, while leaving the maximum possible unchanged, and doubling the response spectra values did not have as much effect upon failure probabilities as the other uncertaintities.

For low strength designs the rank of the different inputs, in terms of importance, was reversed from the high strength designs, but in general all of the sensitivities were reduced. Doubling the acceleration or response spectra roughly doubled the failure probability as compared to tripling or quadrupling for the high strength designs.

3. The recent USAEC WASH-1400 study suggested that seismic failure probabilities in the range of 10^{-5} to 10^{-6} per year were sufficient to be able to neglect seismic failure in the overall safety analysis. To predict this level of failure probability it is shown that it is necessary to establish the probability of occurrence of large magnitude earthquakes with a high degree of confidence. From this it is concluded that the EMR earthquake probability predictions are unsuitable for assessing nuclear plant safety system failure probabilities because of the recognised uncertaintities in extrapolation of the data to long return periods.

4. For systems with several components the failure probability can be appreciably lowered by designing the components to have widely separated natural periods so that the probability of receiving a high response spectra value in all components for the same earthquake is reduced.

5. The sensitivity of failure probability to changes in design level remained fairly constant at all design levels considered.

RECOMMENDATIONS

1. The selection of the DBE, which with an associated deterministic response spectra curve makes up the component design acceleration, should be based on the desired combined system failure probability. It has been suggested that an acceptable system failure probability is in the range 10^{-5} to 10^{-6} per year and it appears reasonable to expect to achieve this safety level.

2. Efforts are required to increase the reliability of large magnitude earthquake probability predictions or alternatively to develop a basis for estimating a maximum possible earthquake, as this is of prime importance in predicting the low failure probabilities of safety systems.

3. Increased efforts in predicting the failure probability of structures and components is required. At present there is practically no information available in this area on which system failure probabilities are highly sensitive.

4. In predicting the failure probabilities of multi-component systems from a common cause such as an earthquake, common mode failures <u>must</u> be taken into consideration.

5. System components should be designed to have widely separated natural periods as this can reduce the system failure probabilities.

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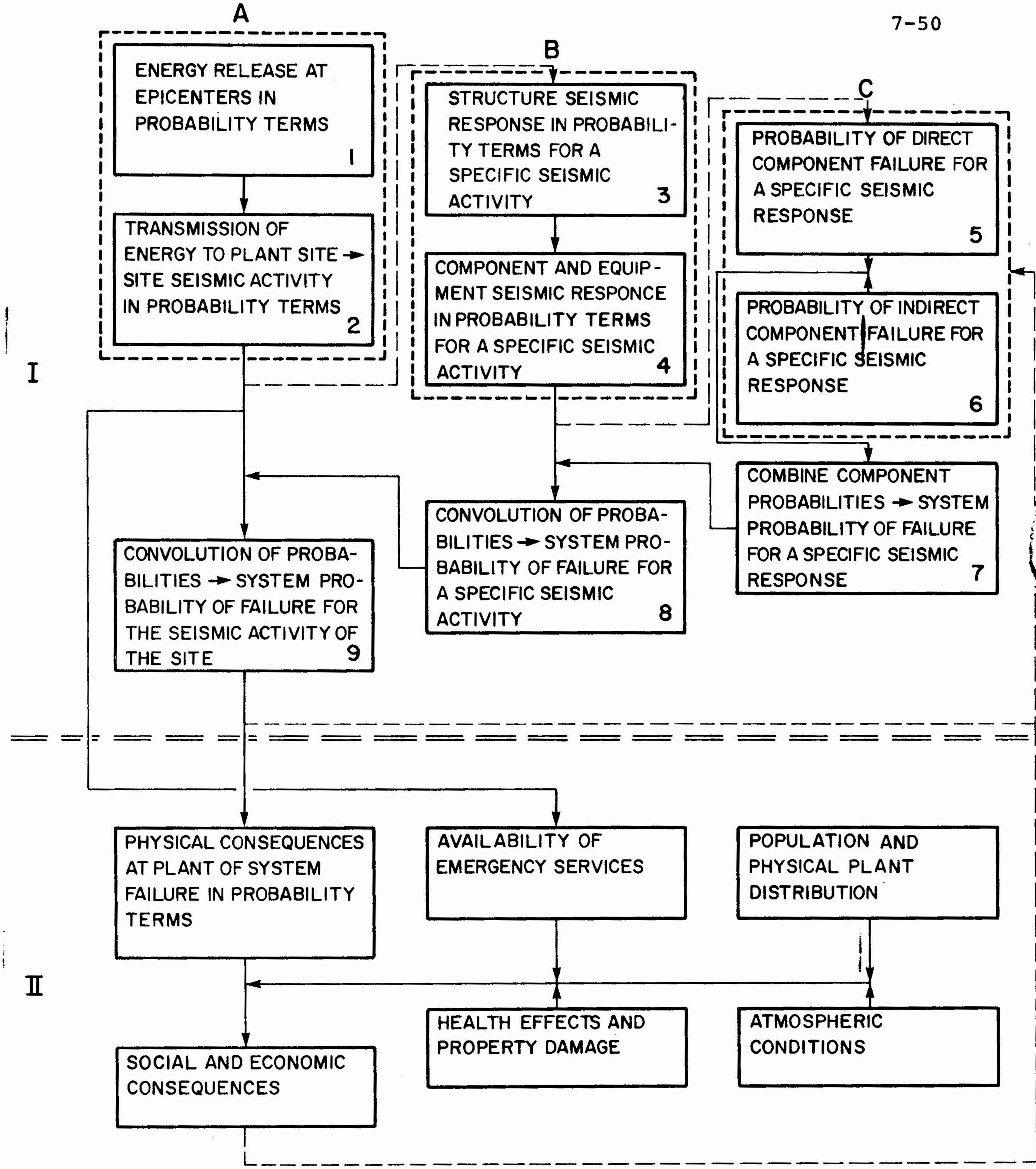
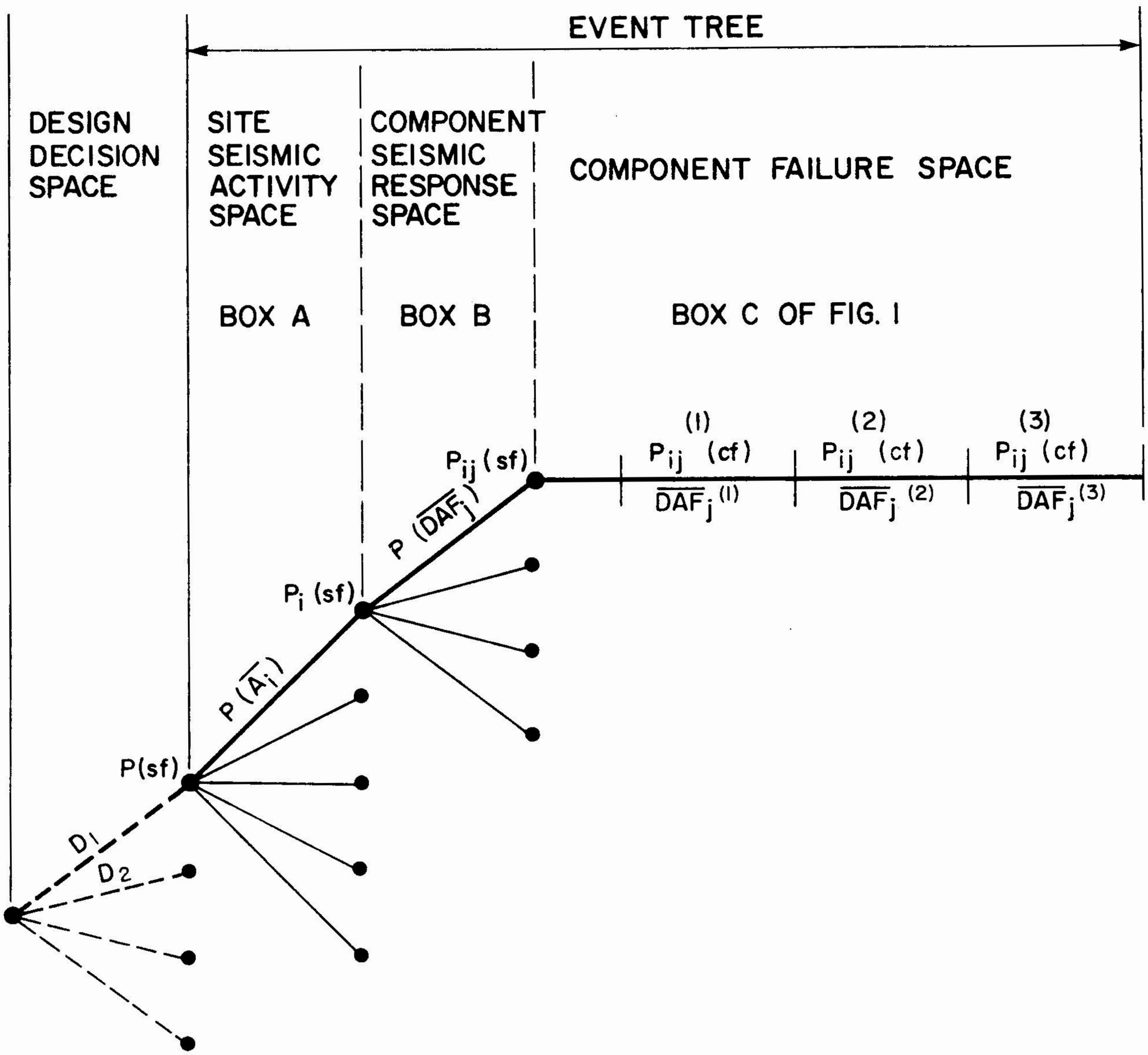


FIG. 1 FLOW CHART OF ANALYSIS MODEL



D_1, D_2 — DESIGN DECISIONS

$P(\overline{A}_i)$ — PROBABILITY INTERVAL i, $\overline{A}_i$ — MEAN VALUE OF ACCELERATION IN INTERVAL

$P(\overline{DAF}_j)$ — PROBABILITY INTERVAL j, $\overline{DAF}_j$, - REPRESENTS A SET OF DYNAMIC AMPLIFICATION FACTORS FOR THE DIFFERENT COMPONENTS

$\overline{DAF}_j^{(k)}$ — DAF FOR THE k^{th} COMPONENT IN THE j^{th} PROBABILITY INTERVAL

$P_{ij}^{(k)}(cf)$ — PROBABILITY OF FAILURE OF THE k^{th} COMPONENT FOR THE ACCELERATION $\overline{A}_i\ \overline{DAF}_j^{(k)}$

$P_{ij}(sf)$ — $P_{ij}^{(1)}(cf)\ P_{ij}^{(2)}(cf) \cdot\cdot P_{ij}^{(N)}(cf)$ WHERE N = NUMBER OF COMPONENTS-BOX NUMBER 7 OF FIG.I.

$P_i(sf)$ — $\sum_j P(\overline{DAF}_j)\ P_{ij}(sf)$ = PROBILITY OF SYSTEM FAILURE CONDITIONAL UPON SITE ACCELERATION A_i - BOX NUMBER 8 OF FIG. I

$P(sf)$ — $\sum_i P(\overline{A}_i)\ P_i(sf)$ = SITE SYSTEM FAILURE PROBABILITY — BOX 9 OF FIG.I

FIG. 2 EVENT TREE

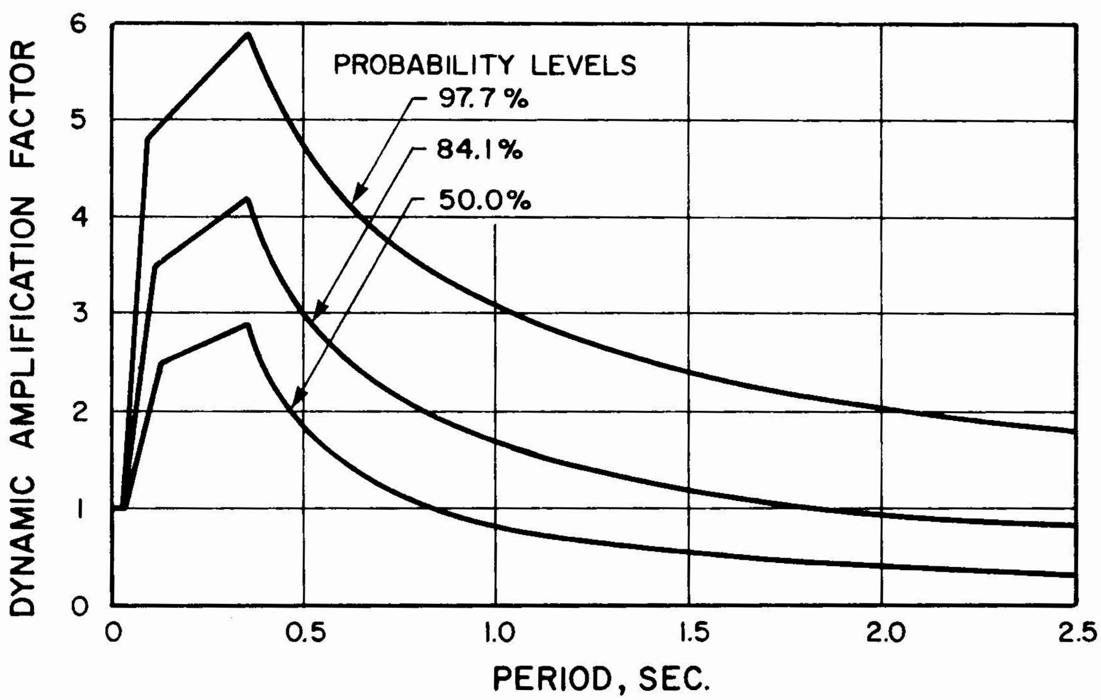


FIG. 3 SMOOTHED DYNAMIC AMPLIFICATION FACTOR PROBABILITY CURVES. DAMPING RATIO = 2%

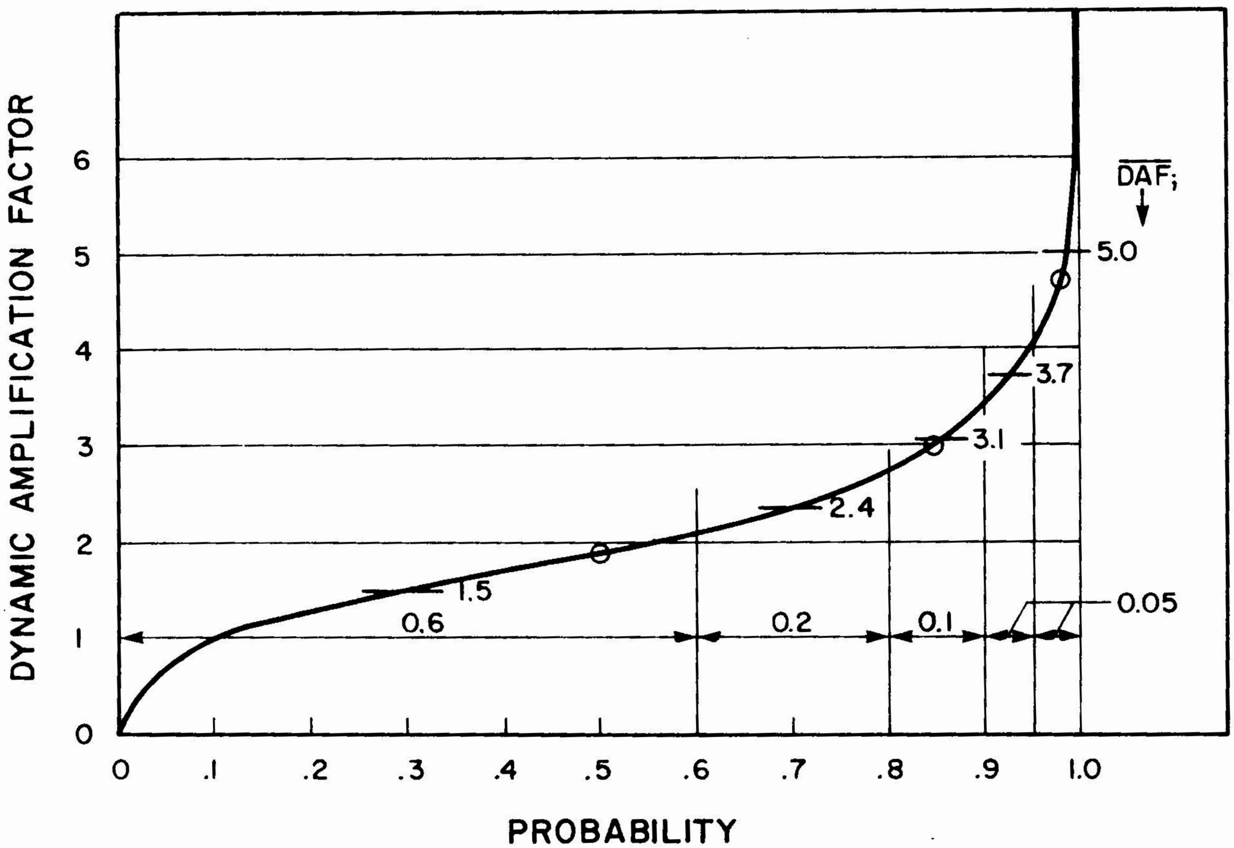


FIG. 4 PROBABILITY DISTRIBUTION OF DYNAMIC AMPLIFICATION FACTOR. PERIOD = 0.5 SEC., DAMPING RATIO = 2%

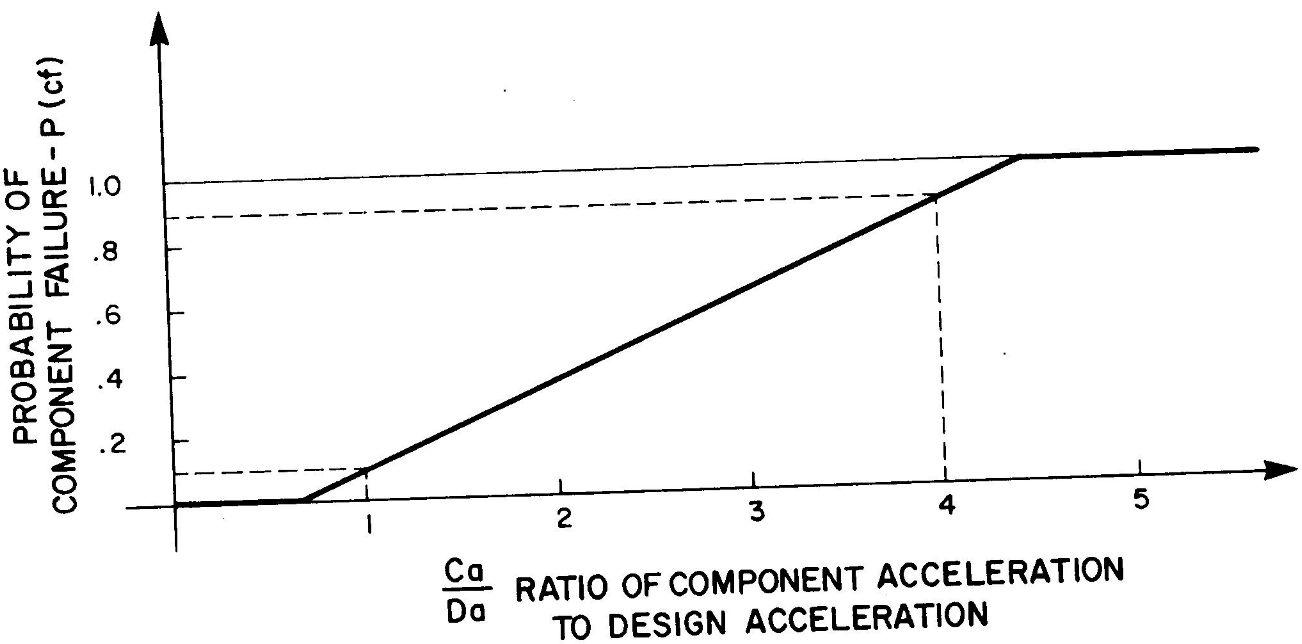


FIG. 5 COMPONENT FAILURE PROBABILITIES

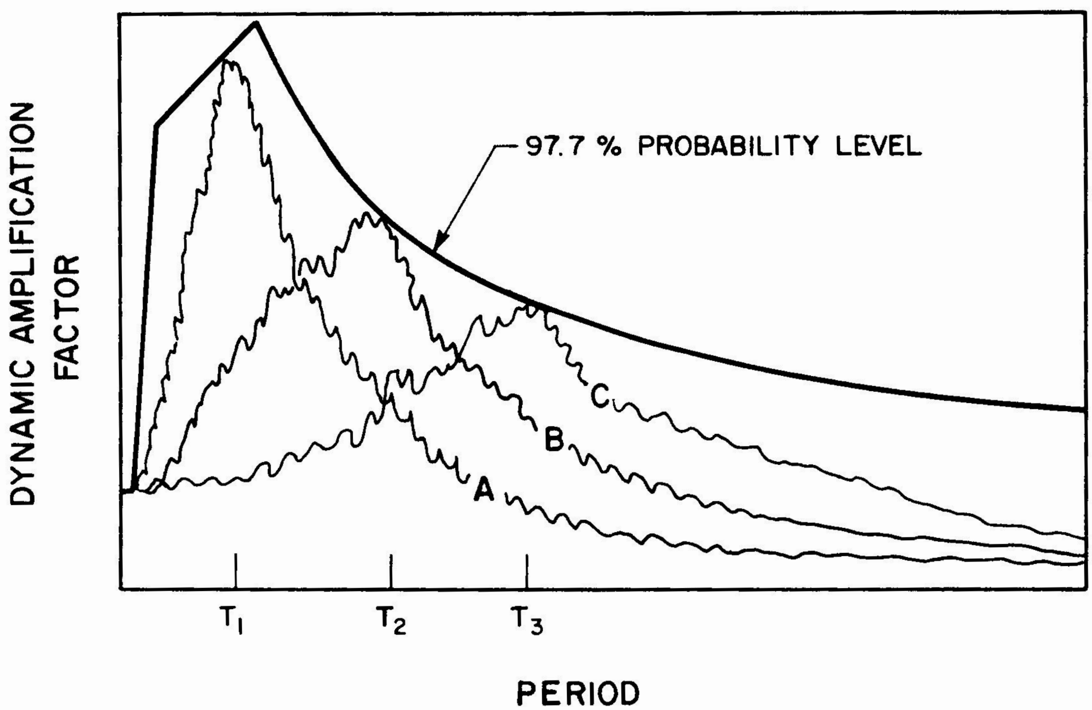


FIG. 6 DYNAMIC AMPLIFICATION FACTOR OF DIFFERENT EARTHQUAKES

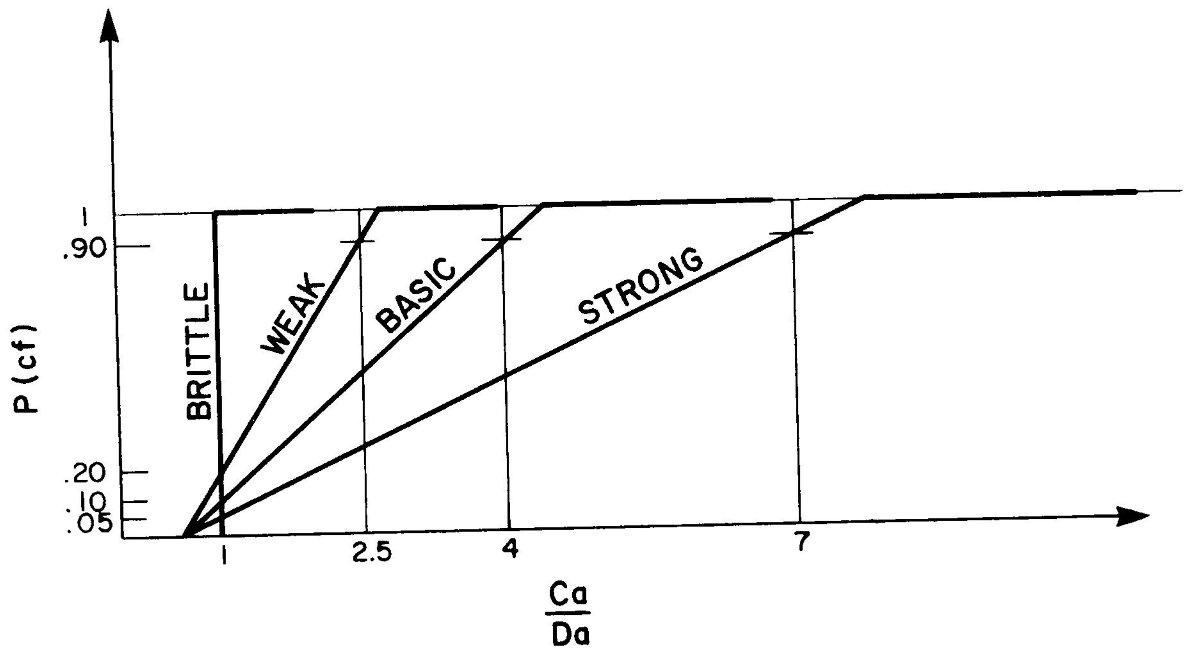


FIG. 7 VARIATIONS OF THE COMPONENT FAILURE PROBABILITIES