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Docket Nos. 50-275 & 50-323

MEMORANDUM FOR: John F. Stolz, Chief, LWR Branch No. 1, DPM

FROM: Dennis P. Allison, Project Manager, LWR Branch No. 1, DPM

SUBJECT: DIABLO CANYON SEISMIC DESIGN - CONSULTANT REPORT

On November 9, 1977, we received the attached report, "A Probabilistic Seismic Safety Assessment of the Diablo Canyon Nuclear Power Plant," by Dr. Newmark and Dr. Ang.

The purpose of this memorandum is to transmit the report to the parties and the Public Document Rooms.

Original Signed By
Dennis P. Allison

Dennis P. Allison, Project Manager
LWR Branch No. 1
Division of Project Management

Attachment

M 4
GD

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DATE➤	11/ /77					

NOV 1 2 1977

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7 November 1977

Mr. Dennis Allison
Light Waters Reactor Branch
Division of Project Management
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Re: Transmittal of Report entitled
"A Probabilistic Seismic Safety Assessment
of the Diablo Canyon Nuclear Power Plant"

Dear Mr. Allison:

Enclosed is our report entitled as above, prepared by Dr. A. H-S. Ang, and approved by me. We are sending this to you Airmail-Special Delivery in the hope that you and others in your organization will have time to read it and discuss the report with us before I have to leave for San Francisco on 13 November.

In a short while I shall send you an addendum to this report with some additional considerations involved, and I trust that it will reach you before you have to submit your safety evaluation report.

Sincerely yours,

N. M. Newmark

N. M. Newmark

pg

Enclosure

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A PROBABILISTIC SEISMIC SAFETY ASSESSMENT OF THE DIABLO CANYON NUCLEAR POWER PLANT

by

A. H-S. Ang and N. M. Newmark

Report to the

U.S. Nuclear Regulatory Commission
Washington, D.C.

N. M. Newmark Consulting Engineering Services
Urbana, Illinois
November 1977

FOREWORD

This report represents the result of a study on the evaluation of the seismic safety of the Diablo Canyon nuclear power plant, performed by N. M. Newmark Consulting Engineering Services for the U.S. Nuclear Regulatory Commission.

References to some of the material and publications were provided by Dr. W. J. Hall during the course of the study. The analysis of the seismic hazard was performed with the assistance of Dr. A. Der-Kiureghian.

CONTENTS

A PROBABILISTIC SEISMIC SAFETY ASSESSMENT OF THE DIABLO CANYON NUCLEAR POWER PLANT

by

A. H-S. Ang and N. M. Newmark

1. Introduction

1.1 Background and Description of Problem

This study is a quantitative evaluation of the levels of safety of certain critical components or subsystems of the Diablo Canyon Nuclear Power Plant facility against seismic hazards. Specifically, safety is assessed in terms of the respective probability of damage. The probability evaluations were performed particularly in the light of the recently discovered Hosgri fault, as well as other known fault zones in the region of the plant. At its closest point, the Hosgri fault is approximately 6 kilometers offshore from the plant site.

The power plant is located in an active seismic region of Southern California. A number of fault zones have been identified in the general region (within 100 kilometers) of the plant, including the San Andreas, the San Simeon, the Rinconada and other faults. The distances of these various faults to the plant site vary from 6 kilometers for the Hosgri to 88 kilometers for the Santa Ynez fault as shown in Fig. 1.

The study is limited only to the consideration of the seismic hazard. Concern for the seismic safety of the plant arises from the fact that the plant was originally designed for a safe shutdown earthquake (SSE) of 0.4 g; presumably, at the time of the design, the existence of the Hosgri fault, as well as its proximity to the plant site, were not

confirmed and thus its implication on the plant safety was not fully recognized. The choice of an SSE of 0.4 g for the design of the plant subsequently proved to be insufficient in light of the evidence confirming the Hosgri fault as it is now recognized. Because of the location of the Hosgri fault relative to the plant site, the seismic hazard to the plant, therefore, is higher than that originally envisaged during the design of the plant. Consequently, the original level of safety perceived for the plant would be lower than it actually is.

The increased seismic hazard arising from the Hosgri fault can be quantitatively expressed only in terms of higher probabilities of the more severe ground motion intensities at the plant site. In other words, the seismic hazard can be predicted only in terms of probabilities. In this light, even though the safety level (in terms of probability of damage) of the existing plant over the original life of thirty years will be lower than that originally considered acceptable for the plant, the original level of safety may nevertheless be maintained for a life shorter than thirty years; for example, for a life of two or three years of operation, whereas if longer operational lives are desired, the plant must be retrofitted or upgraded for a higher SSE in order to achieve a safety level, at least, equal to that considered acceptable in the nuclear power industry.

The results developed in this study are intended to assist in resolving these questions. Furthermore, the study should serve also to assist in determining how much retrofitting would be necessary to achieve an acceptable level of safety for a full thirty-year operational life of the plant.

1.2 Premise and Scope of Study

The probabilities calculated herein pertain only to the safety of specific plant components against seismic hazard; in particular, the simultaneous occurrence of earthquakes and other environmental or man-made hazards is not considered. Moreover, probabilities of damage are calculated for specific critical components and/or sub-systems such as piping; consideration of the damage or failure probability of a complex system in a nuclear power plant will require fault tree and event-tree analyses, which in turn would require detailed knowledge of the system configuration and interrelationships among the components (NRC, WASH-1400), which the authors are not prepared to do as a part of this study.

The problems involve low probability and high consequence. Because extremely small probabilities are involved, the absolute values of the calculated probabilities may not, in themselves, be used to answer the question of whether or not the plant is safe enough. Nevertheless, the results provide a useful and consistent measure of safety. In particular, the calculated damage probabilities of the existing or retrofitted plant relative to the corresponding probabilities of the plant as originally designed (which postulated the absence of the Hosgri fault) may be used as a quantitative measure of relative safety.

For these purposes, damage probabilities were evaluated for the purpose of the following:

(i) To determine the damage probability of the existing plant for a life of 30 years, which was designed for an SSE OF 0.4 g with a safety factor of 4 to 6, assuming the absence of the Hosgri fault.

Presumably this probability corresponds to the safety level deemed to be adequate during the design of the plant (had there not been the Hosgri fault). Furthermore, if there were no Hosgri fault, the safety level of the existing plant may not have been seriously questioned. This calculation, therefore, serves to establish the level of damage probability that is implicitly considered to be acceptably low enough of current nuclear power plants.

(ii) To assess the damage probability of the existing plant for an interim life of 2 years under the "real" seismic hazard; i.e., including the presence of the Hosgri fault.

(iii) To evaluate the damage probability of the plant for a life of 30 years, if it is retrofitted to withstand an SSE of 0.75 g with a safety factor comparable to that of the original design (or slightly less) under the "real" seismic hazard including that from the Hosgri fault.

II. The Probabilistic Approach to Safety Assessment

2.1 Bases of Approach

Damage of a structural component, or safety equipment, would occur when the seismic ground motion exceeds the capability of the component to resist such motions. The maximum ground acceleration that could occur at the site of the power plant over a finite time period is clearly not predictable; similarly, the seismic capability of a major structural component or the fragility limit of a piece of equipment may only be described with a random variable. Consequently, the probability of the resistance being less than the conceivable levels of seismic motions at the site is the relevant measure of safety.

The calculated probabilities must necessarily depend on the bases under which the structural components were designed, and the various safety-related equipments were qualified, for seismic resistances. In this regard, it is assumed that the major structural components of the Diablo Canyon power plant were designed according to standard practice for the design of nuclear power plant structures, and the Class 1 safety equipments were qualified in accordance with recommended practices, e.g. IEEE (1974, 1975).

Because extremely small probabilities are involved, they cannot be estimated directly on the basis of statistical observations, nor verified on the same basis. However, the required probabilities may be deduced through a proper synthesis of available information and statistical data, using established physical relationships of the underlying problem. In this regard, it is important to recognize that the derived probabilities should be technologically credible, in the sense that it is based on the

logical synthesis of various pieces of information, each of which is based on objective data or can be individually judged to be technically reasonable or sound.

In performing the probability calculations, methodologies for determining the seismic hazards (e.g. the maximum ground motions) at the site as well as for determining the description of the resistances are required. The methods used for each of these purposes are described below.

2.2 Determination of Seismic Hazards

Realistically, over a finite time period, it is not possible to state with any precision the maximum ground acceleration that can be expected at the site of the Diablo Canyon power plant. The description of the ground motion intensity may be developed only in terms of probability -- specifically, for example, the probability of exceeding specified acceleration levels in a year, or the annual probability of exceedance.

The site of the power plant is subject to potential earthquake hazards from several well-defined fault systems, including the recently discovered Hosgri fault which is six kilometers offshore from the plant at its closest point. Although all the existing faults in the region are potential sources of earthquakes, the high intensity motions that may be expected at the site would largely be the result of events on the nearby faults, especially the Hosgri. In any case, the total seismic hazard at the site would be the cumulative contributions from all the sources in the region. The determination of these hazards can be performed systematically. In the present case, this determination is based on the recently developed fault-rupture model of Der-Kiureghian and Ang (1977). The details of this

model will not be described here; it will suffice only to point out the main features of this model.

The DerKiureghian-Ang model is physically consistent with the important characteristics of an earthquake event. It is based on the assumption that an earthquake originates as a fault break at its focus and propagates as an intermittent series of ruptures or slips in the earth's crust, and that the maximum intensity (e.g. acceleration) of ground shaking at the site is determined by the rupture that is closest to the site. Accordingly, the maximum ground acceleration at any point in the neighborhood of an earthquake attenuates with the shortest distance from the rupture. Finally, the occurrence of earthquakes with magnitudes $M \geq 4$ constitutes a Poisson process and, consistent with Richter's magnitude law, the magnitude of an earthquake is described with a truncated exponential distribution (Rosenblueth, 1973).

Schematically, the major fault systems in the region of the Diablo Canyon power plant is depicted in Fig. 1. For each of the faults indicated in Fig. 1, the frequencies of earthquakes of magnitude $M \geq 4$ are assumed to be the same as those observed historically in this region; furthermore, within a given fault, earthquakes are equally likely to occur anywhere along the fault. Also, during a given earthquake, the length of the fault rupture will be a function of the magnitude and appropriate relationship for this purpose must be specified.

The maximum intensity at the plant site, of course, will also depend on the distance to the closest rupture for an earthquake of given magnitude; as illustrated in Fig. 2a, if an earthquake occurs on one of the faults of such a magnitude as to produce a rupture length ℓ, then the

maximum acceleration at the site will be determined by the distance d as shown in Fig. 2a. Besides the hazards of earthquakes from the known faults, earthquakes may occur also anywhere spatially within a one hundred-kilometer radius; that is, the epicenter may not necessarily be on one of the known fault systems. Earthquakes from these latter sources are assumed to have fault ruptures that may propagate in any direction from its focus as illustrated in Fig. 2b; the maximum acceleration, however, is still determined by the shortest distance d to the causative fault. Appropriate relationship for the attenuation of the maximum acceleration with d, therefore, will be required.

In using the fault rupture model of Der-Kiureghian and Ang (1977), a number of physical relationships as well as parameter values, therefore, must be specified for the site in question. In this regard, the specific relationships and parameter values used for the Diablo Canyon plant site are based on information and data directly pertinent to the site.

With this model, the known physical relationships as well as all historical data of earthquakes and pertinent seismological information are synthesized systematically in a manner consistent with the major characteristics of earthquakes. Of course, the validity of the results will still depend on the reliability of the data and information used with the model to obtain the calculated results. In the present case, the bases for such information and data are described in Sect. 3.1.

On these bases, the total hazards of the Diablo Canyon power plant site is then obtained as the sum of all potential sources within a region of 100 kilometers. The result is the seismic hazard curve representing the annual probabilities of exceedance associated with specified maximum ground accelerations.

2.3 Determination of Seismic Resistances

The capabilities of specific structural components and equipments to resist specified levels of ground motions will obviously influence the respective probabilities of damage. However, for the purpose of calculating the underlying failure probabilities, it is not necessary to know the actual resistance capability or fragility limit of the structural component or piece of equipment; information on its seismic resistance capability relative to the maximum motion to which it is subjected should be sufficient. This information is contained in the factors of safety used in the design of the structural components; similarly, the margins or degrees of conservatism underlying the design of the structure or equipment would provide equivalent information. Following Newmark, the safety factors for the structure as well as the piping and equipment are defined as the ratio between the median value of the resistance to the safe shutdown earthquake prescribed for the design of the plant.

Another resistance parameter that has a bearing on the damage probability is the degree of uncertainty underlying the prediction of resistance, which may be expressed conveniently in terms of the coefficient-of-variation. This must include all the uncertainties underlying the predicted or estimated resistance. One of the main tasks, therefore, in the evaluation of the damage probability is the assessment of the various sources of uncertainty associated with the prediction of seismic resistance or fragility limit. Available observed data, including information on ranges of observed measurements, may be used in this determination. Invariably, however, the available data and information will be insufficient to provide completely objective bases for assessing the underlying degree

of uncertainty. In this light, it will be necessary to augment the available information with engineering judgment; the necessary judgments, however, may have to be expressed in or translated into probability terms in order to derive the appropriate coefficients-of-variation.

2.4 Calculation of Damage Probability

At a given level of ground acceleration, say a_o, the probability of damage of a component with resistance distribution function $F_R(a)$ would simply be

$$p_F = F_R(a_o) \tag{2.1}$$

which is the area in the probability density function of R below a_o as shown in Fig. 3. However, since there could conceivably be a wide range of possible ground accelerations, which may be described as a random variable with probability density function $f_A(a)$, the probability of damage must be integrated over all possible values of a. Hence,

$$p_F = \int_o^\infty F_R(a)\ f_A(a)\ da \tag{2.2}$$

An alternative but equivalent expression for the damage probability is

$$p_F = \int_o^\infty [1 - F_A(a)]\cdot f_R(a)\ da \tag{2.3}$$

in which $1 - F_A(a)$ is the exceedance probability of acceleration a, as given by the hazard curve of Fig. 4a. For the present study, Eq. 2.3 is preferred over Eq. 2.2 as the ordinate of the hazard curve gives directly $[1 - F_A(1)]$.

Numerically, Eq. 2 3 can be evaluated as

$$p_F = \sum_{\text{all } a\text{'s}} [1 - F_A(a)] \cdot \Delta F_R(a) \qquad (2.4)$$

where $\Delta F_R(a) = F_R(a + \frac{\Delta a}{2}) - F_R(a - \frac{\Delta a}{2})$ is the probability that the resistance will be in a small interval Δa, as represented by the incremental area shown in Fig. 4b.

Whereas the probability distribution of the seismic environment is described by the hazard curve obtained from a seismic risk analysis, the probability distribution for the resistance R may be prescribed to be lognormal (following Newmark, 1974). Thus,

$$F_R(a) = \phi\left(\frac{\ln a/\bar{r}}{\zeta_R}\right) \qquad (2.5)$$

in which $\phi(x)$ is the standard normal probability, and,

$$\zeta_R = \sqrt{\ln(1 + \Omega_R^2)} \qquad (2.6)$$

where $\bar{r}$ is the median of R; and Ω_R is the coefficient-of-variation representing the uncertainty in R.

Prescription of the lognormal distribution for R may be justified on the following ground. The correct resistance may be represented as the calculated resistance $\hat{R}$ multiplied by a series of correction factors; i.e.

$$R = C_1 C_2 \ldots C_k \hat{R} \qquad (2.7)$$

where C_i are random variables, representing the corrections for site conditions, dynamic amplification, effects of damping, dynamic analysis model, etc. Irrespective of the proper distribution of these individual variables, including that of $\hat{R}$, the product in Eq. 2.7 will (at least) be

approximately lognormal by virtue of the central-limit theorem.

The damage probability described above applies to individual components, where $\Delta F_R(a)$ is evaluated for suitable increments of a. However, in the case of a piping system, there may be several potential damage locations in the pipe; the damage of any of the critical sections will constitute failure of the entire system. In such a case, if there are n independent critical sections in a piping system, the resistance distribution of the system would be

$$F_R(a) = 1 - [1 - F_1(a)]^n \tag{2.8}$$

in which $F_1(a)$ is the resistance of one critical section.

The incremental probability $\Delta F_R(a)$ then becomes

$$\Delta F_R(a) = n[1 - F_1(a)]^{n-1} \cdot \Delta F_1(a) \tag{2.9}$$

Furthermore, it is generally recognized (e.g. NRC, WASH 1400) that failure of two or more systems would be necessary to cause release of radioactive material in the plant; for example, the joint occurrence of damage to a pipe section and malfunction of the associated safety equipment. In such a case, the cumulative distribution function of the resistance becomes

$$F_R(a) = F_1(a) \cdot F_E(a) \tag{2.10}$$

where $F_E(a)$ is the distribution function of the equipment fragility. From which the incremental resistance probability, therefore, is

$$\Delta F_R(a) = F_1(a) \cdot \Delta F_E(a) + F_E(a) \cdot \Delta F_1(a) \tag{2.11}$$

In Eqs. 2.8 and 2.10, the distribution functions of the respective seismic resistances are also assumed to be lognormal.

III. Pertinent Data and Information Base

As indicated earlier, specific physical relationships and parameter values are needed for the calculation of the required probabilities. The information base used for developing such relationships and for evaluating the specific parameter values are summarized below. In every case, information and data pertinent to the Diablo Canyon site are used for these purposes. However, where objective information is not available or sufficient, subjective judgments are introduced to augment the available information base; when necessary, such judgments would generally tend to be on the conservative side.

3.1 Data for Seismic Hazard Analysis

Obviously, the level of seismic hazard at the Diablo Canyon power plant site would depend on the seismicity of the major faults in the region surrounding the site. The locations of the major faults have been identified geologically; these are summarized in Fig. 1. At the closest point, the surface distances of these faults from the Diablo Canyon plant as well as the lengths of these faults are shown below in Table 3.1. Besides the known faults, it is assumed that earthquakes could originate also at any point within a 100-km radius from the plant; fault ruptures from such earthquakes could propagate in any direction. Potential sources within this 100-km radius are represented by the four annular areas indicated in Table 3.1.

Table 3.1 Seismic Source Definition

Seismic Source	Surface Distance from site, km	Fault Length, km
San Andreas	75	400
Santa Lucia Bank	48	55
Rinconada-Ozena	30	195
Santa Ynez	88	130
San Simeon	25	132
Hosgri	6	107
Annular areas:		
1	12	--
2	37	--
3	62	--
4	90	--

Seismicity of Each Fault -- The frequency of significant earthquakes, i.e. earthquakes with $M \geq 4$, are different among the various faults in this region. As expected, the average occurrence rate of such earthquakes is highest for the San Andreas fault. In any case, the occurrence rate of significant earthquakes for each of the faults are determined or evaluated on the basis of the epicenter map for this region, as shown in Fig. 5, which is reproduced from Volume II of the Final Safety Analysis Report (1974) for the Diablo Canyon power plant. Figure 5 represents the epicenter map for this region for the period 1934 through

1971. In the case of the Hosgri, the relocated epicenters reported by Gawthorp (1975) were also used. In evaluating the average occurrence rates for the respective faults, each epicenter shown in Fig. 5 is assigned to one of the faults within about 5 kilometers. In this way, an actual count of the epicenters for the respective faults is made, from which the average occurrence rate for each fault is then determined. On this basis, the annual occurrence rates of $M \geq 4$ for the various faults were obtained as shown in Table 3.2 below. For the areal sources, i.e. earthquakes not in any of the faults, the occurrence rates indicated below represent those for each of the annular areas; the number of epicenters in Fig. 5 that do not belong to one of the faults were divided in proportion to the annular areas to obtain the respective occurrence rates shown in Table 3.2 for these areas.

Table 3.2 Seismic Source Parameters

Seismic Source	Annual Mean Occurrence Rate of $M \geq 4$	Upper-Bound Magnitude, m_u
San Andreas	6.0	8.0
Santa Lucia Bank	0.92 (0.01-0.02)	7.5
Rinconada-Ozena	0.92 (0.18-0.36)	7.5
Santa Ynez	0.41	7.5
San Simeon	0.46	7.5
Hosgri	0.22 (0.02-0.30)	7.5
Annular areas:		
1	0.09	6.5
2	0.27	6.5
3	9.45	6.5
4	5.40	6.5

The annual mean occurrence rate of earthquakes in the different faults may also be assumed to be proportional to the fault movements over the last 10,000 or 20 million years. Based on the fault motion data provided by Hamilton (transmitted through NRC), reproduced in Table 3.3, the annual mean occurrence rates of $M \geq 4$ would be those shown in parentheses in Table 3.2 for the respective faults, relative to an occurrence rate of 6 per year on the San Andreas. Observe that the occurrence rate of 0.22 per year, used in the calculations for the Hosgri fault, is within the range (0.02-0.30).

Table 3.3 Total Dislocation (Tentative Values)

	In last 10,000 years (meters)	In last 20×10^6 years (km)
San Andreas	500	200
Hosgri	2	10
Rinconada	30	6
Nacimiento	4	0.8
West Huasna-Suey	2	0.4
La Panza-Ozena	2	0.4
Santa Lucia Bank	2	0.4

Data from Doug Hamilton, with some interpretation.

Note that the San Andreas average rate of slip has increased "recently" while the others have decreased.

Frequency Distribution of Magnitude -- The magnitude of significant earthquakes (i.e. with $M \geq 4$) can be described with the

shifted exponential distribution (Rosenblueth, 1973), as follows:

$$F_M(m) = k[1 - e^{-\beta(m-4)}]; \qquad 4 \leq m \leq m_u \tag{3.1}$$

where $k = 1 - e^{-\beta(m_u-4)}$, in which m_u is the upper-bound magnitude. Because the results could be sensitive to m_u (Der-Kiureghian and Ang, 1977), reasonable values of m_u must be specified for each of the potential earthquake sources; in the present case, the m_u assigned for the various sources in the region are as shown in Table 3.2; namely, $m_u = 8$ for the San Andreas, $m_u = 7.5$ for all the other faults, and $m_u = 6.5$ for earthquakes originating in any of the annular areas.

The parameter β in Eq. 3.1 influences the relative frequencies of different magnitudes, and therefore must also be carefully evaluated. This may be obtained as the slope to the magnitude recurrence curve for the region. The magnitude recurrence curve is a plot of the earthquake magnitude M versus the annual number of events with magnitudes greater than or equal to M. The frequency of different earthquake magnitudes in this region has been summarized by Anderson and Trifunac (1976); on the basis of these data, the recurrence line shown in Fig. 6 is developed. The recurrence line shown in Fig. 6 can be observed to be slightly conservative, as it is above almost all of the data points plotted on this figure. The slope of this recurrence line yields a value of $\beta = 2.01$.

The Attenuation Equation -- The attenuation of the maximum acceleration as a function of the shortest surface distance to the fault rupture (or causative fault) is needed with the seismic risk model of Der-Kiureghian and Ang. For the Southern California region, data of various earthquakes representing the maximum acceleration as a function of the

distance to the causative fault for various large-magnitude earthquakes have been reported by Page, et al (1972). These data are summarized in Fig. 7. Page, et al (1972) also observed that the maximum ground acceleration attenuates with the shortest distance d to the slipped fault at a rate of $d^{-1.5}$ to $d^{-2.0}$. On the basis of these data, the following attenuation for maximum acceleration is developed:

$$a = 1.35\, e^{0.67M}(d + 15)^{-1.75} \tag{3.2}$$

The maximum accelerations given by Eq. 3.2 may be compared with the observed data for this region in Fig. 7. It may be emphasized that in Eq. 3.2, d is the surface distance to the slipped fault.

Rupture Length Equation -- The rupture length is obviously a function of the earthquake magnitude. For this purpose, the rupture length equation proposed by Patwardhan, Tocher, and Savage (1975) is used for $M \leq 7$. This relationship can be represented by

$$\ell = 10^{(0.91M - 4.62)} \tag{3.3}$$

which gives rupture lengths in kilometers. Specific rupture lengths given by Eq. 3.3 for various magnitudes can be seen in the following table.

Magnitude	Rupture Length, km
4	0.1
5	1
6	7
6.5	20
7	56
7.5	107*
8	400**

*Assumed to be equal to the length of the Hosgri fault.

**Assumed to be equal to the length of the San Andreas fault.

3.2 The Seismic Hazard Curve

Using the data described above, and with the DerKiureghian-Ang seismic risk model, the annual probability of exceedance for all levels of ground acceleration (up to 1.5 g) were calculated. The results are the seismic hazard curves for the Diablo Canyon power plant site shown in Fig. 8.

Three specific cases were considered as follows:

(1) Considering all seismic sources in the region.

(2) Considering all sources except the Hosgri fault. Presumably, this is the seismic environment postulated during the design of the plant.

(3) Considering only the Hosgri fault. This case calculates the risk associated with earthquakes originating on the Hosgri fault only.

The resulting seismic hazard curves for all the three cases described above are summarized in Fig. 8. The annual exceedance probabilities for specific values of the maximum ground acceleration, ranging from 0 to 1.5 g, are also presented in Table 3.4.

The results presented in Fig. 8 also included the effects of the uncertainties underlying the seismic risk model. The most significant of these uncertainties is that associated with the attenuation equation (Der-Kiureghian and Ang, 1975). From the scatter of the observed accelerations shown in Fig. 7, the coefficient-of-variation representing the uncertainty underlying the above attenuation equation, Eq. 3.2, is estimated to be around 30%. In short, the hazard curves presented in Fig. 8 has been corrected for the uncertainties associated with the seismic risk model including, in particular, the major uncertainty underlying the attenuation equation.

According to the results of the three cases considered herein, the following information (see Fig. 8) may be inferred: At the higher

Table 3.4 Seismic Hazards at Diablo Canyon Power Plant Site

Annual Max. Ground Accel., g	Annual Probability of Exceedance		
	All Sources	All Sources Except Hosgri	Hosgri only
0.10	5.0×10^{-2}	3.4×10^{-2}	1.6×10^{-2}
0.20	7.0×10^{-3}	4.0×10^{-3}	3.0×10^{-3}
0.30	1.9×10^{-3}	8.8×10^{-4}	1.0×10^{-3}
0.40	7.4×10^{-4}	2.4×10^{-4}	5.0×10^{-4}
0.50	3.6×10^{-4}	6.6×10^{-5}	2.9×10^{-4}
0.60	2.0×10^{-4}	1.8×10^{-5}	1.8×10^{-4}
0.70	1.2×10^{-4}	4.8×10^{-6}	1.2×10^{-4}
0.80	7.8×10^{-5}	1.2×10^{-6}	7.7×10^{-5}
0.90	5.0×10^{-5}	2.4×10^{-7}	5.0×10^{-5}
1.00	3.1×10^{-5}	3.1×10^{-8}	3.1×10^{-5}
1.10	1.9×10^{-5}	8.1×10^{-10}	1.9×10^{-5}
1.20	1.2×10^{-5}	--	1.2×10^{-5}
1.30	7.2×10^{-6}	--	7.2×10^{-5}
1.40	4.3×10^{-6}	--	4.3×10^{-6}
1.50	2.5×10^{-6}	--	2.5×10^{-6}

acceleration levels, say $\geq$ 0.7 g, the seismic hazard at the Diablo Canyon power plant site is virtually entirely due to the Hosgri fault. At the lower levels of acceleration, the contributions from the other sources are more significant.

3.3 Response and Resistance Determinations

With regard to the evaluation of the structural components, it is the seismic capacity of a component relative to the applied maximum earthquake motion that is relevant. In describing the capacity of a structural component, it will suffice to know the factor of safety used in its design, as this then determines the design capacity relative to the specified safe shutdown earthquake (SSE). Structural components of nuclear power plant structures are believed to have an underlying factor of safety that is about three times that for ordinary structures (Newmark, 1975). Thus, the actual factor of safety for nuclear plant structures would range between 4 and 6; i.e. the median seismic resistances would be 4 to 6 times the specified SSE.

With regard to the seismic capability of a pipe section, it is believed that a pipe can undergo significant inelastic deformations prior to incurring damage; for this reason, it is reasonable to assume that the piping is designed to withstand the SSE with a safety factor slightly higher than that of the structure -- a value of 6 is considered realistic.

In the case of equipments, some reserve capacity is normally assured on the basis of qualification tests. According to the IEEE Standards (1974, 1975), light equipments are normally proof-tested for vibrations with maximum acceleration at the equipment mounting that is

10% above that expected during an SSE. The capacities of heavy equipments, however, are determined through calculations. In either case, the safety factors of Class 1 equipments are difficult to estimate; however, it is believed that safety factors ranging from 2.5 to 4 are probably reasonable. Light electrical equipments may generally tend to have lower safety factors than those of heavy mechanical equipments.

The degree of uncertainty in both the response prediction and capacity determination of the structure as well as the various equipments are also important in evaluating the damage probabilities. The various sources of uncertainty and their combined effects are assessed systematically below.

Uncertainty in Structural Response Prediction and Capacity -- In the case of the structural response prediction, the several sources of uncertainty are as follows:

(i) Effects of Site-Dependent Factors on the Definition of Ground Motions -- This would include the effect of local soil conditions and geology on the predicted ground motion at the site as determined by the seismic hazard curve described earlier. Information that can be used to assess the level of uncertainty associated with these factors is scarce. Based largely on subjective judgment, this uncertainty is estimated to be on the order of 20%, expressed in terms of coefficient-of-variation (c.o.v.).

(ii) Variability in Amplification Factors -- Mohraz, Hall, and Newmark (1972) in their study of a large number of earthquakes have reported the 50 and 90-percentile values of the amplification factors for displacement, velocity, and acceleration; from this study, the 50% and 90% values of the respective amplifications are as follows:

	Disp. Ampl.	Vel. Ampl.	Accel. Ampl.
50% value	1.40	1.66	2.11
90% value	2.21	2.51	2.82

Assuming that the amplification factors can be described as Gaussian random variables, the variabilities for the respective amplifications (expressed in terms of c.o.v.) are:

$$\Omega_d = 45\%$$

$$\Omega_v = 40\%$$

$$\Omega_a = 26\%$$

Therefore, a coefficient-of-variation of 30% to represent the uncertainty associated with the variability in the amplification factor for acceleration appears reasonable.

(iii) Uncertainty in the Specification of the Structural Damping -- The uncertainty associated with the specification of damping coefficients used in the response calculation of a structure may be of the order of 30%, as has been suggested by Newmark (1974).

(iv) Modeling of Structure for Dynamic Analysis -- Because of the complexity of a nuclear power plant structure, there is bound to be significant uncertainty in modeling the structure for dynamic response analysis. This should include the effect of structure-soil interaction when pertinent. It is conceivable that this uncertainty could be of the order of 30%, which would include the effects of soil-structure interaction. Since the Diablo Canyon power plant is built on rock, where the effects of soil-structure interaction would not be significant, the 30% assigned herein could be on the conservative side.

On the basis of the individual uncertainty levels assessed above, the total uncertainty associated with the prediction of structural response, based on first-order approximation (Ang, 1973; Ang and Cornell, 1974), therefore, is

$$\Omega_{res} = \sqrt{0.20^2 + 0.30^2 + 0.30^2 + 0.30^2}$$
$$= 0.56$$

With regard to the uncertainty underlying the estimation of the capacity of the structure or structural component, it is assumed that this uncertainty will be comparable to that associated with the prediction of the resistance to blast forces (Ang, Hall, and Hendron, 1974), which has been estimated to be 30%.

Therefore, the total uncertainty underlying both the response prediction and structural capacity becomes

$$\Omega_{structure} = \sqrt{0.56^2 + 0.3^2} = 0.64$$

From which the parameter ζ of Eq. 2.6 is

$$\zeta_{structure} = \sqrt{\ln(1 + 0.64^2)} = 0.59$$

<u>Uncertainty in the Response Prediction and Resistance of Piping Systems</u> -- The forcing function on a piping system would involve the floor response spectra of the structure. Accordingly, any uncertainty in the prediction of the response of a piping system would be above and beyond those estimated earlier for the structure. This additional uncertainty should include those underlying the in-structure motions such as the floor

response spectra that constitute the input to the response analysis of the piping system, as well as any imperfection associated with the modeling and response analysis of the piping system itself. Consider the complexity of pipings in a nuclear power plant, the uncertainty associated with these factors can be expected to be large; conceivably, this could range from 30% to 50%. For the purpose of this study, a coefficient-of-variation of 50% will be assumed to represent the level of uncertainty in the prediction of piping system response. Therefore, the total uncertainty associated with the piping system response, or the forces at a pipe section, would be

$$\Omega_{Res} = \sqrt{0.56^2 + 0.50^2} = 0.75$$

With regard to the resistance of a pipe section, it is reasonable to assume that pipes can take some inelastic action; for example, it is not unreasonable to expect a ductility factor of 1.5 for pipes. Accordingly, the uncertainty in the resistance of a piping system may be comparable to that of a structural component, and thus a coefficient-of-variation for the resistance of a pipe section of 30% may be assigned.

The total uncertainty associated with the response calculation and resistance of a pipe section, therefore, is

$$\Omega_{pipe} = \sqrt{0.75^2 + 0.30^2} = 0.81$$

Thus,

$$\zeta_{pipe} = \sqrt{\ln(1 + 0.81^2)} = 0.71$$

Uncertainty in the Response Prediction and Fragility of Equipment -- In the case of equipments, the additional uncertainty in the in-structure motions as well as in the modeling and response analysis of the equipment would be the same as those of the piping system; these uncertainties, of course, would again be above and beyond those underlying the response prediction of the structure. In other words, a coefficient-of-variation of 50% may also be used for the response prediction of equipment. Accordingly, the uncertainty associated with the response prediction of equipment would also be

$$\Omega_{res} = \sqrt{0.56^2 + 0.50^2} = 0.75$$

Information on the seismic capacity of equipments are generally proprietary (IEEE Standards, 1974). For this reason, specific information on the fragility limits of equipments used in nuclear power plants are quite rare. However, information on fragility limits of equipments subjected to blast-induced nuclear ground shocks is available (Newmark, et al, 1963); according to Ang, Hall, and Hendron (1974), the coefficients-of-variation associated with these data are 60% for heavy mechanical equipments, and 80% for light electrical and electronic equipments. On this basis, an average c.o.v. of 70%, therefore, may be used for the fragility of nuclear plant equipments in general. Hence, the total uncertainty underlying the response prediction and fragility of equipments is

$$\Omega_E = \sqrt{0.75^2 + 0.70^2} = 1.03$$

from which

$$\zeta_E = \sqrt{\ln(1 + 1.03^2)} = 0.85$$

IV. Calculated Damage Probabilities

With the formulations described in Chapter II and the specific relationships and parameter values given in Chapter III, the probabilities of specific adverse events involving the damage of major structural components, piping system, or safety-related equipment, or some combinations thereof are evaluated. In all cases, damage is defined to mean that the seismic capacity of the component or equipment is exceeded by the maximum seismic acceleration occurring at the site. Depending on the component concerned, damage is defined as in the following:

Component	Definition of Damage
Structural	Some definite degree of yielding at a critical section
Piping	Slight yielding at a critical location of a pipe
Equipment	Malfunction, or failure of equipment to perform required safety function, during an earthquake

In all cases, the relevant probabilities are calculated first in terms of the annual probability of damage, from which the damage probabilities for other durations of life are derived. These are obtained as the sum of the incremental probabilities, Δp_F, contributed by all levels of ground accelerations, as indicated in Eq. 2.4.

It may be emphasized that the calculated damage probabilities are based on certain tacit assumptions regarding the manner in which the various components are designed, as described in Chapter III. Such probabilities are, therefore, necessarily dependent on the criteria under which the components were designed; for example, on the factor of safety and the safe

shutdown earthquake specified for the design of these components. In this regard, the following are considered to be realistic conditions for nuclear power plant components.

Component	Safety Factor against SSE
Structural	4 to 6
Piping	6
Equipment	2.5 to 4

That is, a safety factor of between 4 and 6 against yielding under the SSE underlies the design of major structural components. In the case of the piping system, the safety factor is 6; whereas for equipment, the safety factor ranges from 2.5 to 4. In all cases, the SSE is 0.4 g for the existing plant, whereas an SSE of 0.75 g would be assumed for the retrofitted system.

Damage probabilities are calculated for two seismic environments: considering all the potential seismic sources in the region of the site, including the Hosgri fault; whereas, the other excludes the Hosgri fault in the region. The seismic hazard of the latter seismic environment (that is excluding the presence of the Hosgri fault) presumably is the environment believed to be appropriate at the time of the design of the original plant; damage probabilities calculated under this seismic environment, therefore, would correspond to the accepted safety level for the plant if the Hosgri fault did not exist.

Furthermore, aside from the damage probabilities for the existing plant, corresponding probabilities are also calculated for the plant retrofitted for an SSE of 0.75 g, assuming that approximately the same safety factors or range of safety factors can be maintained for the retrofitted system.

4.1 Damage of Structural Components

The probability of damage of specific structural components is evaluated using Eq. 2.4, in which $1-F_A(a)$ is the ordinate of the seismic hazard curve of Fig. 8, and $F_R(a)$ is assumed to be lognormal.

The results of the calculations are summarized in Table 4.1, which shows the incremental contributions to the damage probability from the different levels of ground accelerations. The sum of the incremental probabilities, Δp_F, is then the annual damage probability for a structural component.

The annual damage probabilities given in Table 4.1 pertain to the existing structure exposed to all the seismic hazard in the region including, in particular, that from the Hosgri fault. The damage probability for a period of two years under this "real" seismic environment is between 2.38×10^{-5} and 1.10×10^{-4}. In Table 4.1, the annual damage probabilities corresponding to the seismic environment originally assumed in the design of the plant (that is without the Hosgri fault) are also presented. On this latter basis, the thirty-year damage probability ranges between 6.12×10^{-5} and 4.65×10^{-4}; presumably this latter damage probability corresponds to the safety level accepted for the original design of the plant for a life of thirty years; implicitly, this damage probability has been considered acceptable for the design of nuclear power plant structural components.

In Table 4.2 are shown the corresponding damage probabilities of the structure if retrofitted for an SSE of 0.75 g. Assuming that approximately the same safety factors can be maintained, retrofitting would yield the thirty-year structural damage probability ranging from 3.42×10^{-5} to 1.42×10^{-4}.

Table 4.1 Probability of Damage of Structural Component of Existing Plant (SSE = 0.4 g)

	Incremental Probability, Δp_F			
	With Hosgri		Without Hosgri	
Accel., g	FS* = 4	FS = 6	FS = 4	FS = 6
0.20	1.60×10^{-5}	1.49×10^{-6}	9.08×10^{-6}	8.48×10^{-6}
0.40	1.63×10^{-5}	2.74×10^{-6}	5.28×10^{-6}	8.86×10^{-7}
0.60	1.14×10^{-5}	2.92×10^{-6}	1.03×10^{-6}	2.65×10^{-7}
0.80	6.62×10^{-6}	2.35×10^{-6}	1.00×10^{-7}	3.55×10^{-8}
1.00	3.07×10^{-6}	1.41×10^{-6}	3.01×10^{-9}	1.38×10^{-9}
1.20	1.20×10^{-6}	6.76×10^{-7}	--	--
1.40	4.07×10^{-7}	2.74×10^{-7}	--	--
Annual p_F:	5.50×10^{-5}	1.19×10^{-5}	1.55×10^{-5}	2.04×10^{-6}

Without Hosgri, 30-yr $p_F = 6.12 \times 10^{-5} - 4.65 \times 10^{-4}$

With Hosgri, 2-yr $p_F = 2.38 \times 10^{-5} - 1.10 \times 10^{-4}$

*FS = ratio of median resistance to SSE = 0.4 g

Table 4.2 Probability of Damage of Structural Component of Retrofitted Plant (SSE = 0.75 g)

	Incremental Probability, Δp_F	
	With Hosgri	
Accel., g	FS* = 4	FS = 5.5
0.20	3.34×10^{-7}	3.13×10^{-8}
0.40	8.50×10^{-7}	1.25×10^{-7}
0.60	1.13×10^{-6}	2.32×10^{-7}
0.80	1.09×10^{-6}	2.84×10^{-7}
1.00	7.49×10^{-7}	2.39×10^{-7}
1.20	4.05×10^{-7}	1.52×10^{-7}
1.40	1.80×10^{-7}	7.78×10^{-8}
Annual p_F:	4.74×10^{-6}	1.14×10^{-6}
With Hosgri, 30-yr	$p_F = 3.42 \times 10^{-5} - 1.42 \times 10^{-4}$	

*FS = ratio of median resistance to SSE = 0.75 g

It may be emphasized that the probabilities calculated above pertain to the damage (which means yielding) of individual structural components. In general, it does not necessarily mean the collapse of the component. Moreover, in the case of redundant structures, the damage of a single structural component may not be of serious consequence to the entire structural system. However, common design and construction procedures, plus a stringent inspection requirement to achieve uniform resistance quality, would tend to increase the statistical correlation between the resistances of various components. For this latter reason, damage to one structural component may likely cause similar damage to other components in the same structure.

4.2 Damage of Piping System

In the case of the piping system, the resistance along the length of the pipe will generally be correlated; however, because there are numerous sections and welded connections of a piping system in a nuclear power plant, many of which could be potential locations of weakness, it is likely that some of the pipe sections may be uncorrelated. In a given piping system, therefore, it is reasonable to assume that there could be several, say n, sections of potential damage locations in the system that are statistically independent. The occurrence of damage (e.g., yielding or some observable cracking at a welded joint) in one section would be an adverse event of the piping system; hence, damage in this case is determined by the weakest among the independent sections. Accordingly, for a piping system, the resistance distribution function would be defined by Eq. 2.8, in which $F_1(a)$ is the distribution function of the resistance of one pipe section. The entire piping system, of course, is subject to the same ground motion environment;

this common-mode loading is considered and implied in Eq. 2.4, which is the basis for calculating the damage probabilities for a piping system.

It is difficult, however, to determine how many of the potential damage locations in a piping system are uncorrelated or statistically independent; i.e., the appropriate value of n for a piping system may be difficult to evaluate. In the calculations summarized in Table 4.3, values for n between 1 and 10 were assumed. The incremental probabilities in each case are shown in Table 4.3 for accelerations ranging from 0 to 1.5 g in increments of 0.2 g. The respective annual damage probabilities are the sums of all the incremental probabilities tabulated in Table 4.3. On the basis of the results summarized in Table 4.3, the thirty-year damage probability of the piping system for the seismic environment used in the design of the existing plant (i.e. without the Hosgri fault) is between 3.05×10^{-4} and 2.96×10^{-3}; whereas the current two-year damage probability of the piping (with the Hosgri fault) is between 6.44×10^{-5} and 5.46×10^{-4}.

Table 4.4 shows the corresponding pipe damage probabilities for the plant if retrofitted for an SSE of 0.75 g. Assuming that the same safety factors can be maintained for the retrofitted plant, the thirty-year damage probability of the piping would range between 9.25×10^{-5} and 8.68×10^{-4}.

4.3 Damage (Malfunction) of Equipment

Safety-related equipment in nuclear power plants are believed to be designed with factors of safety ranging from 2.5 to 4. Again, these safety factors are defined to be against a specified SSE, which is 0.4 g for the existing plant and 0.75 g for the retrofitted plant.

Table 4.3 Damage Probability of Piping of Existing Plant
(FS = 6 against SSE = 0.4 g)

	Incremental Probability, Δp_F			
	With Hosgri		Without Hosgri	
Accel., g	n* = 1	n = 10	n = 1	n = 10
0.2	1.19×10^{-5}	1.19×10^{-4}	6.77×10^{-6}	6.75×10^{-5}
0.4	8.80×10^{-6}	8.35×10^{-5}	2.84×10^{-6}	2.70×10^{-5}
0.6	5.61×10^{-6}	4.45×10^{-5}	5.08×10^{-7}	4.03×10^{-6}
0.8	3.32×10^{-6}	1.89×10^{-5}	5.03×10^{-8}	2.86×10^{-7}
1.0	1.64×10^{-6}	5.82×10^{-6}	1.60×10^{-9}	5.69×10^{-9}
1.2	6.95×10^{-7}	1.38×10^{-6}	--	--
1.4	2.60×10^{-7}	2.66×10^{-7}	--	--
Annual p_F:	3.22×10^{-5}	2.73×10^{-4}	1.02×10^{-5}	9.88×10^{-5}

Without Hosgri, 30-yr $p_F = 3.05 \times 10^{-4} - 2.96 \times 10^{-3}$

With Hosgri, 2-yr $p_F = 6.44 \times 10^{-5} - 5.46 \times 10^{-4}$

*n = number of potential damage locations in the piping with independent resistances

Table 4.4 Damage Probability of Piping of Retrofitted Plant
(FS = 6 against SSE = 0.75 g)

	Incremental Probability, Δp_F	
	With Hosgri	
Accel., g	$n^* = 1$	$n = 10$
0.2	4.80×10^{-7}	4.80×10^{-6}
0.4	6.78×10^{-7}	6.76×10^{-6}
0.6	6.87×10^{-7}	6.73×10^{-6}
0.8	5.75×10^{-7}	5.38×10^{-6}
1.0	3.75×10^{-7}	3.21×10^{-6}
1.2	1.98×10^{-7}	1.49×10^{-6}
1.4	8.98×10^{-8}	5.66×10^{-7}
Annual p_F:	3.08×10^{-6}	2.89×10^{-5}
With Hosgri, 30-yr	$p_F = 9.25 \times 10^{-5}$ — 8.68×10^{-4}	

*n = number of potential damage locations in the piping with independent resistances

Probabilities of damage or malfunction of a safety-related equipment are shown in Table 4.5, calculated again for two seismic environments; namely, with and without the presence of the Hosgri fault, and for safety factors of 2.5 and 4 in each case. Table 4.5 shows the incremental probabilities contributed by different ground acceleration levels in increments of 0.2 g, yielding the annual damage probabilities as shown in Table 4.5.

Corresponding damage probabilities are also calculated and summarized in Table 4.6 for the retrofitted plant. Again, it is assumed that equipment are or can be upgraded to withstand an SSE of 0.75 g with the same range of safety factors as those of the existing plant.

On the basis of the results summarized in Tables 4.5 and 4.6, the following equipment damage probabilities are obtained for different operational lives.

In the absence of the Hosgri fault, the thirty-year damage probability underlying the original design ranges between 3.36×10^{-3} and 9.99×10^{-3}. In the presence of the Hosgri fault, the two-year damage probability of the existing plant would range between 4.8×10^{-4} and 1.32×10^{-3}; whereas, for the retrofitted plant, the 30-year damage probability of an equipment (in the presence of the Hosgri fault) would range between 1.32×10^{-3} and 2.21×10^{-3}.

4.4 Joint Damage of Piping and Equipment

An adverse event that could likely lead to release of radioactive material is the joint occurrence of damage somewhere along a piping system and the damage or malfunction of the safety-related equipment. The probability of this joint event can be calculated assuming that the resistances

Table 4.5 Damage Probability of Equipment in Existing Plant (SSE = 0.4 g)

	Incremental Probability, Δp_F			
	With Hosgri		Without Hosgri	
Accel., g	FS* = 2.5	FS = 4	FS = 2.5	FS = 4
0.2	5.27×10^{-4}	1.68×10^{-4}	2.99×10^{-4}	9.55×10^{-5}
0.4	9.54×10^{-5}	4.53×10^{-5}	3.08×10^{-5}	1.46×10^{-5}
0.6	2.62×10^{-5}	1.61×10^{-5}	2.38×10^{-6}	1.46×10^{-6}
0.8	8.92×10^{-6}	6.60×10^{-6}	1.35×10^{-7}	9.98×10^{-8}
1.0	2.95×10^{-6}	2.52×10^{-6}	2.88×10^{-9}	2.47×10^{-9}
1.2	9.20×10^{-7}	8.88×10^{-7}	--	--
1.4	2.68×10^{-7}	2.86×10^{-7}		
Annual p_F:	6.61×10^{-4}	2.40×10^{-4}	3.33×10^{-4}	1.12×10^{-4}

Without Hosgri, 30-yr $p_F = 3.36 \times 10^{-3}$ — 9.99×10^{-3}

With Hosgri, 2-yr $p_F = 4.80 \times 10^{-4}$ — 1.32×10^{-3}

*FS = ratio of median resistance to SSE = 0.4 g

Table 4.6 Damage Probability of Equipment in Retrofitted Plant (SSE = 0.75 g)

	Incremental Probability, Δp_F	
	With Hosgri	
Accel., g	FS* = 2.5	FS = 4.0
0.2	8.93×10^{-6}	2.35×10^{-5}
0.4	4.32×10^{-5}	1.05×10^{-5}
0.6	1.28×10^{-5}	5.24×10^{-6}
0.8	5.55×10^{-6}	2.75×10^{-6}
1.0	2.24×10^{-6}	1.28×10^{-6}
1.2	8.15×10^{-7}	5.23×10^{-7}
1.4	2.74×10^{-7}	1.93×10^{-7}
Annual p_F:	7.38×10^{-5}	4.39×10^{-5}
With Hosgri, 30-yr	$p_F = 1.32 \times 10^{-3}$ — 2.21×10^{-3}	

*FS = ratio of median resistance to SSE = 0.75 g

of the piping system and that of the equipment are statistically independent, but are subject to the common seismic environment of ground acceleration. In this case, the joint resistance distribution function is given by Eq. 2.10, from which the joint damage probability is obtained through Eq. 2.4

Joint damage probabilities of the existing plant, as well as of the retrofitted plant, are given in Tables 4.7 and 4.8, respectively. In the case of the existing plant, calculations were performed again assuming two seismic environments; namely, with and without the presence of the Hosgri fault in the region of the plant. For the existing plant, an SSE of 0.4 g is used, whereas for the retrofitted plant, both the piping and the equipment are assumed to be upgraded for an SSE of 0.7 g. In both cases, a safety factor of 6 is assumed for the piping, whereas safety factors ranging from 2.5 to 4 are assumed for the equipments.

Incremental damage probabilities are shown in Tables 4.7 and 4.8 in increments of 0.2 g accelerations, from which the respective sums give the annual joint damage probabilities. On the basis of these results, the joint damage probabilities for different operational lives were obtained as follows.

For the existing plant, the thirty-year joint damage probability assuming the absence of the Hosgri fault would be 1.4×10^{-5} to 3.4×10^{-5}; whereas in the presence of the Hosgri fault, the two-year damage probability of the existing plant would range between 9.2×10^{-6} and 1.69×10^{-5}. For the retrofitted plant, the thirty-year damage probability would be between 6.78×10^{-6} and 1.56×10^{-5}.

Table 4.7 Joint Damage of Piping and Equipment in Existing Plant
(SSE = 0.4 g; FS_{pipe} = 6.0)

	Incremental Probability, Δp_F			
	With Hosgri		Without Hosgri	
Accel., g	FS_E^* = 2.5	FS_E = 4.0	FS_E = 2.5	FS_E = 4.0
0.2	5.63×10^{-7}	1.64×10^{-7}	3.20×10^{-7}	9.30×10^{-8}
0.4	1.89×10^{-6}	8.07×10^{-7}	6.11×10^{-7}	2.60×10^{-7}
0.6	2.24×10^{-6}	1.17×10^{-6}	2.03×10^{-7}	1.06×10^{-7}
0.8	1.85×10^{-6}	1.12×10^{-6}	2.80×10^{-8}	1.70×10^{-8}
1.0	1.13×10^{-6}	7.56×10^{-7}	1.10×10^{-9}	7.40×10^{-10}
1.2	5.48×10^{-7}	3.99×10^{-7}	--	--
1.4	2.26×10^{-7}	1.76×10^{-7}	--	--
Annual p_F:	8.44×10^{-6}	4.60×10^{-6}	1.16×10^{-6}	4.78×10^{-7}

Without Hosgri, 30-yr $p_F = 1.40 \times 10^{-5}$ — 3.40×10^{-5}

With Hosgri, 2-yr $p_F = 9.2 \times 10^{-6}$ — 1.69×10^{-5}

*FS_E = safety factor of equipment; i.e. ratio of median resistance to SSE = 0.4 g

Table 4.8 Joint Damage of Piping and Equipment of Retrofitted Plant ($SSE = 0.75$ g; $FS_{pipe} = 6.0$)

	Δp_F	
	With Hosgri	
Accel., g	$FS_E^* = 2.5$	$FS_E = 4.0$
0.2	2.08×10^{-9}	4.82×10^{-10}
0.4	3.75×10^{-8}	9.44×10^{-9}
0.6	9.08×10^{-8}	3.19×10^{-8}
0.8	1.32×10^{-7}	5.50×10^{-8}
1.0	1.24×10^{-7}	5.86×10^{-8}
1.2	8.49×10^{-8}	4.42×10^{-8}
1.4	4.65×10^{-8}	2.62×10^{-8}
Annual p_F:	5.19×10^{-7}	2.26×10^{-7}
30-yr	$p_F = 6.78 \times 10^{-6}$ — 1.56×10^{-5}	

*FS_E = safety factor of equipment; i.e. ratio of median resistance to SSE = 0.75 g

V. Summary and Conclusions

Seismic safety of the Diablo Canyon Nuclear Power Plant has been systematically evaluated, in terms of the probabilities of adverse events involving the damage or malfunction of one or more components in the plant; although these events in themselves may not be serious and may not necessarily lead to serious adverse consequences, the occurrence of one or more of these adverse events during an earthquake could potentially be a prelude to the release of radioactive material. For this reason, the probabilities of these events represent some measure of the safety of the plant as well as those of its various components. The results of these evaluations are summarized as shown in Table 5.1.

Observations

From the results summarized in Table 5.1, the following may be observed:

1. The two-year damage probabilities of the existing plant, in the presence of the Hosgri fault, are consistently lower (by a factor of about 2 to 7) than the corresponding thirty-year damage probabilities of the plant if the Hosgri fault did not exist.

2. If the Plant were retrofitted for an SSE of 0.75 g, and assuming that the same safety factors can be approximately maintained for the upgraded plant, the thirty-year damage probabilities of the upgraded plant, in the presence of the Hosgri fault, are also consistently lower (by a factor of about 2 to 3) than the original thirty-year damage probabilities considered acceptable during the design of the plant.

Table 5.1 Summary of Damage Probabilities

STRUCTURE

Life	Seismic Environment	Existing Plant FS* = 4	Existing Plant FS = 6	Retrofitted Plant FS* = 4	Retrofitted Plant FS = 6
30-yr	Without Hosgri	4.65×10^{-4}	6.12×10^{-5}	--	--
30-yr	With Hosgri	--	--	1.42×10^{-4}	3.42×10^{-5}
2-yr	With Hosgri	1.10×10^{-4}	2.38×10^{-5}	--	--

PIPING (FS = 6)

Life	Seismic Environment	Existing Plant n** = 1	Existing Plant n = 10	Retrofitted Plant n = 1	Retrofitted Plant n = 10
30-yr	Without Hosgri	3.05×10^{-4}	2.96×10^{-3}	--	--
30-yr	With Hosgri	--	--	9.25×10^{-5}	8.68×10^{-4}
2-yr	With Hosgri	6.44×10^{-5}	5.46×10^{-4}	--	--

EQUIPMENT

Life	Seismic Environment	Existing Plant $FS_E^* = 2.5$	Existing Plant $FS_E = 4.0$	Retrofitted Plant $FS_E = 2.5$	Retrofitted Plant $FS_E = 4.0$
30-yr	Without Hosgri	9.99×10^{-3}	3.35×10^{-3}	--	--
30-yr	With Hosgri	--	--	2.21×10^{-3}	1.32×10^{-3}
2-yr	With Hosgri	1.32×10^{-3}	4.80×10^{-4}	--	--

PIPE DAMAGE AND EQUIPMENT MALFUNCTION

Life	Environment	Existing Plant $FS_E^* = 2.5$	Existing Plant $FS_E = 4.0$	Retrofitted Plant $FS_E = 2.5$	Retrofitted Plant $FS_E = 4.0$
30-yr	Without Hosgri	3.40×10^{-5}	1.40×10^{-5}	--	--
30-yr	With Hosgri	--	--	1.56×10^{-5}	6.78×10^{-6}
2-yr	With Hosgri	1.69×10^{-5}	9.2×10^{-6}	--	--

*FS = safety factor against SSE, defined as ratio of median resistance to SSE. For existing plant, SSE = 0.4 g; for retrofitted plant, SSE = 0.75 g.

** n = number of potential damage locations in a piping system with statistically independent resistances.

Conclusions

In light of the above observations, it can be concluded that for an interim period of two years, the safety of the plant would not be compromised relative to that originally envisaged and accepted for its design; that is, the risk associated with the operation of the existing plant for an interim period of two years will be lower than the risk initially accepted (based on the premise of no Hosgri fault) for a thirty-year life. Moreover, by retrofitting and upgrading the plant for an SSE of 0.75 g, the risk of the retrofitted plant over a period of thirty years will also be less than that originally accepted for its design; assuming, of course, that approximately the same safety factors can be maintained of the retrofitted plant.

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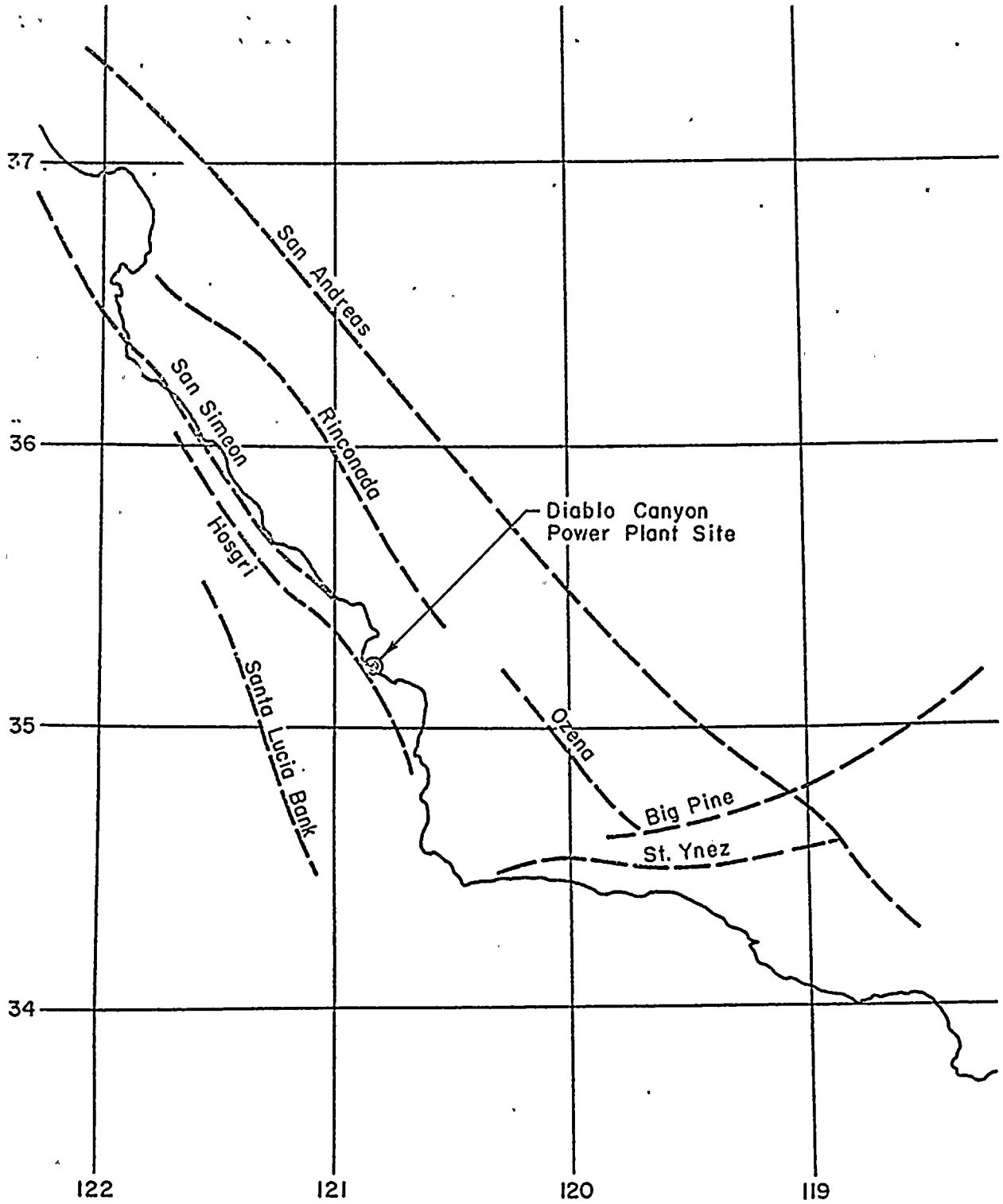

FIG. 1 MAJOR FAULTS IN REGION OF SITE

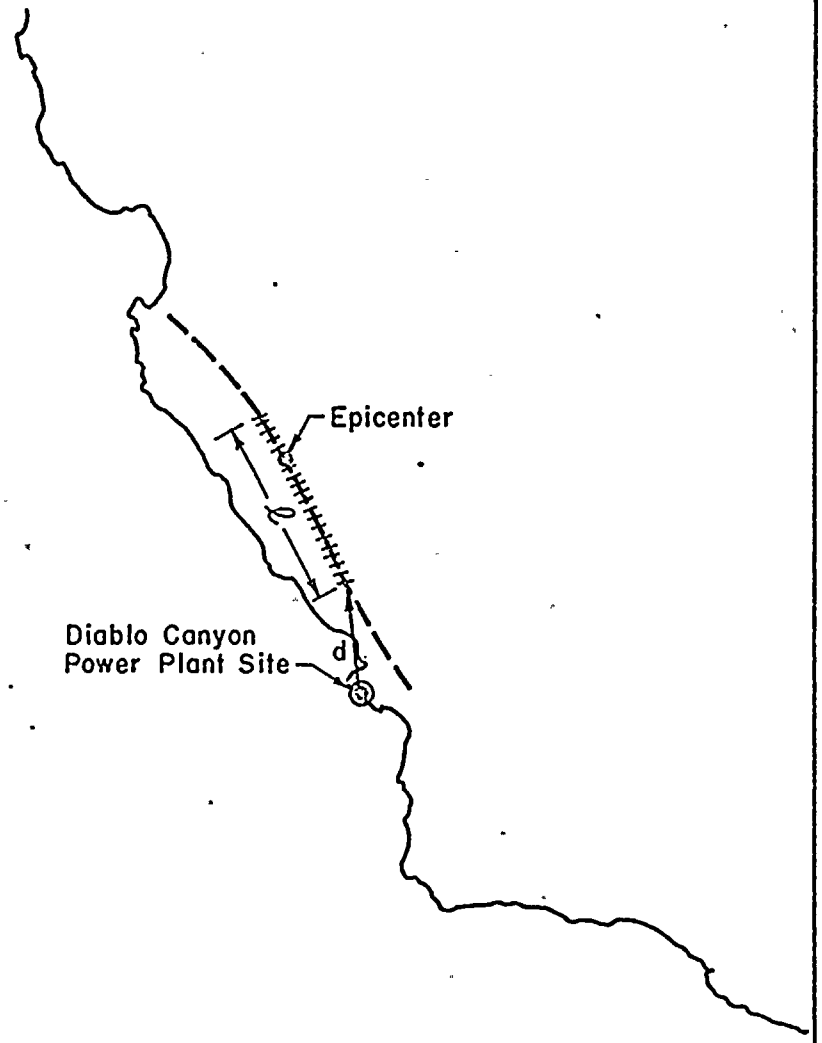

FIG. 2a EARTHQUAKE ON A KNOWN FAULT

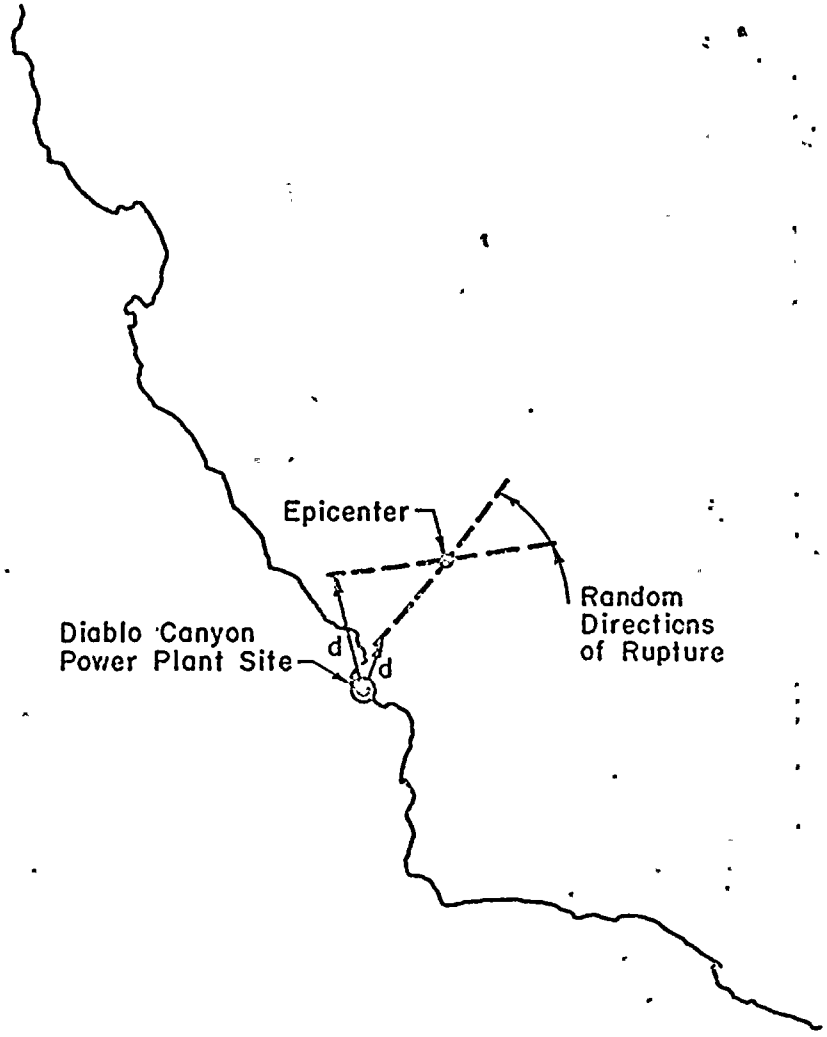

FIG. 2b EARTHQUAKE NOT ON A FAULT

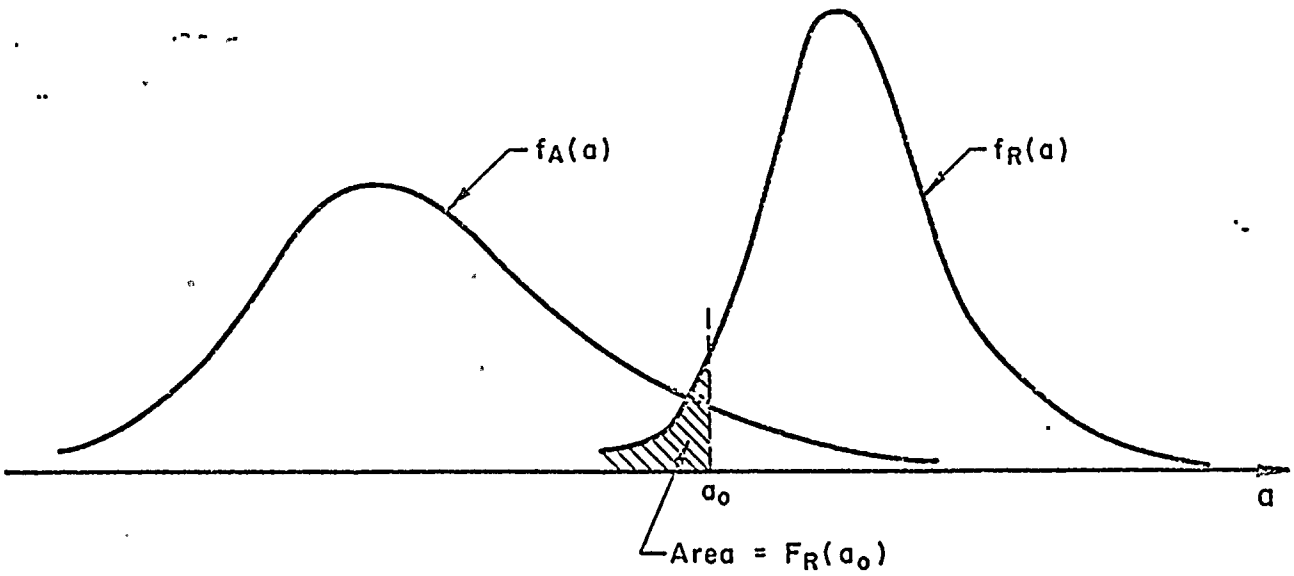

FIG. 3 PROBABILITY DENSITY FUNCTIONS

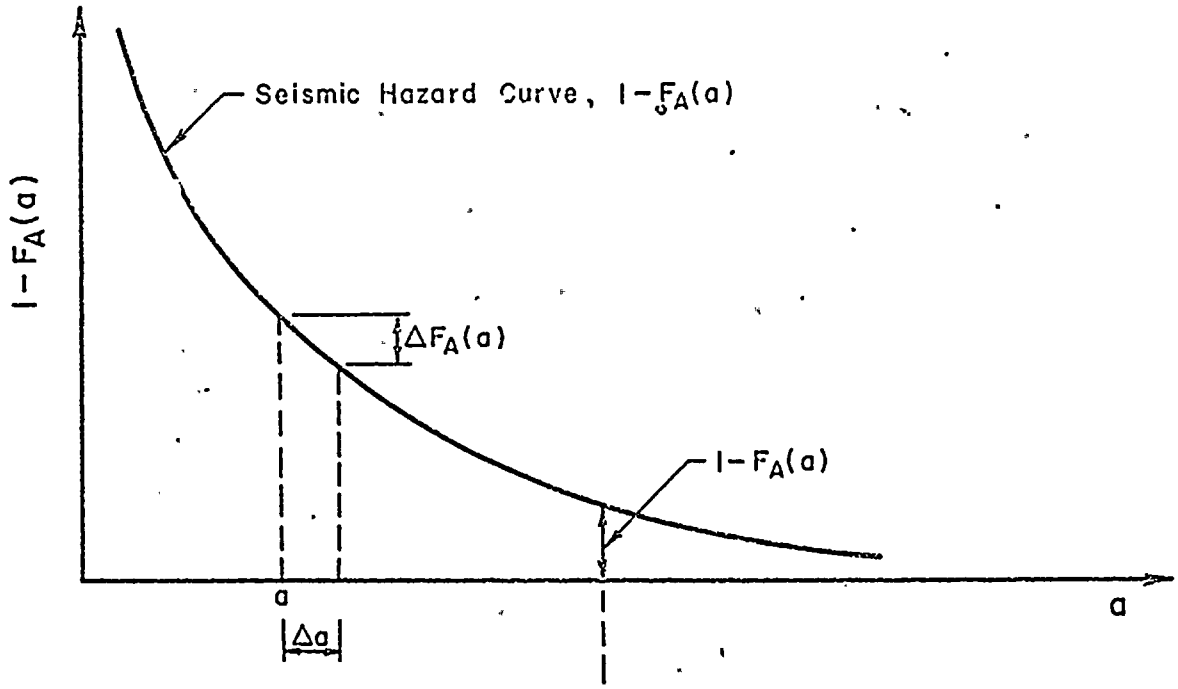

FIG. 4a SEISMIC HAZARD CURVE

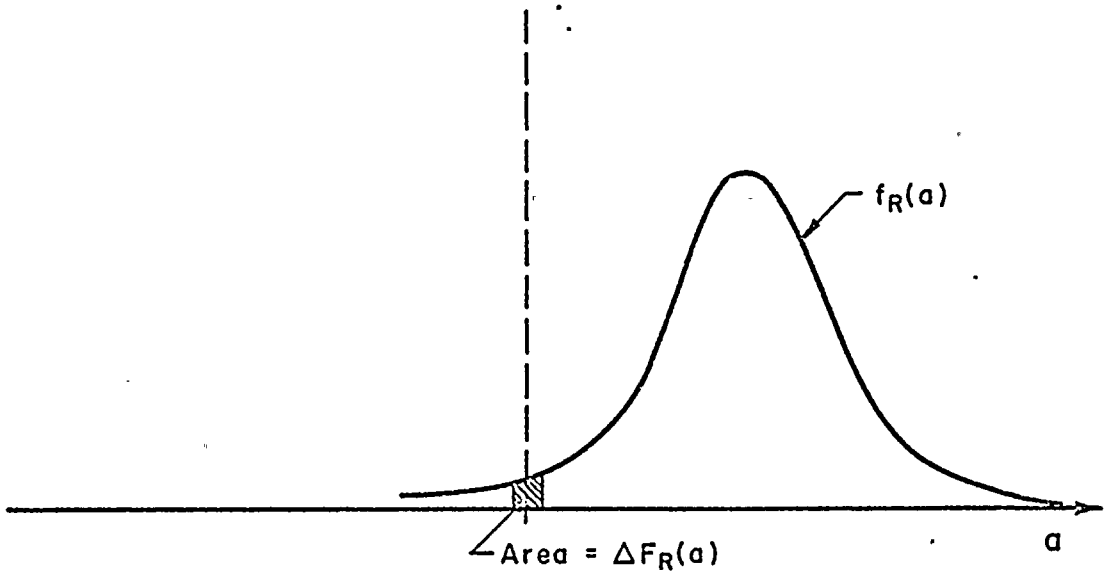

FIG. 4b RESISTANCE DENSITY FUNCTION

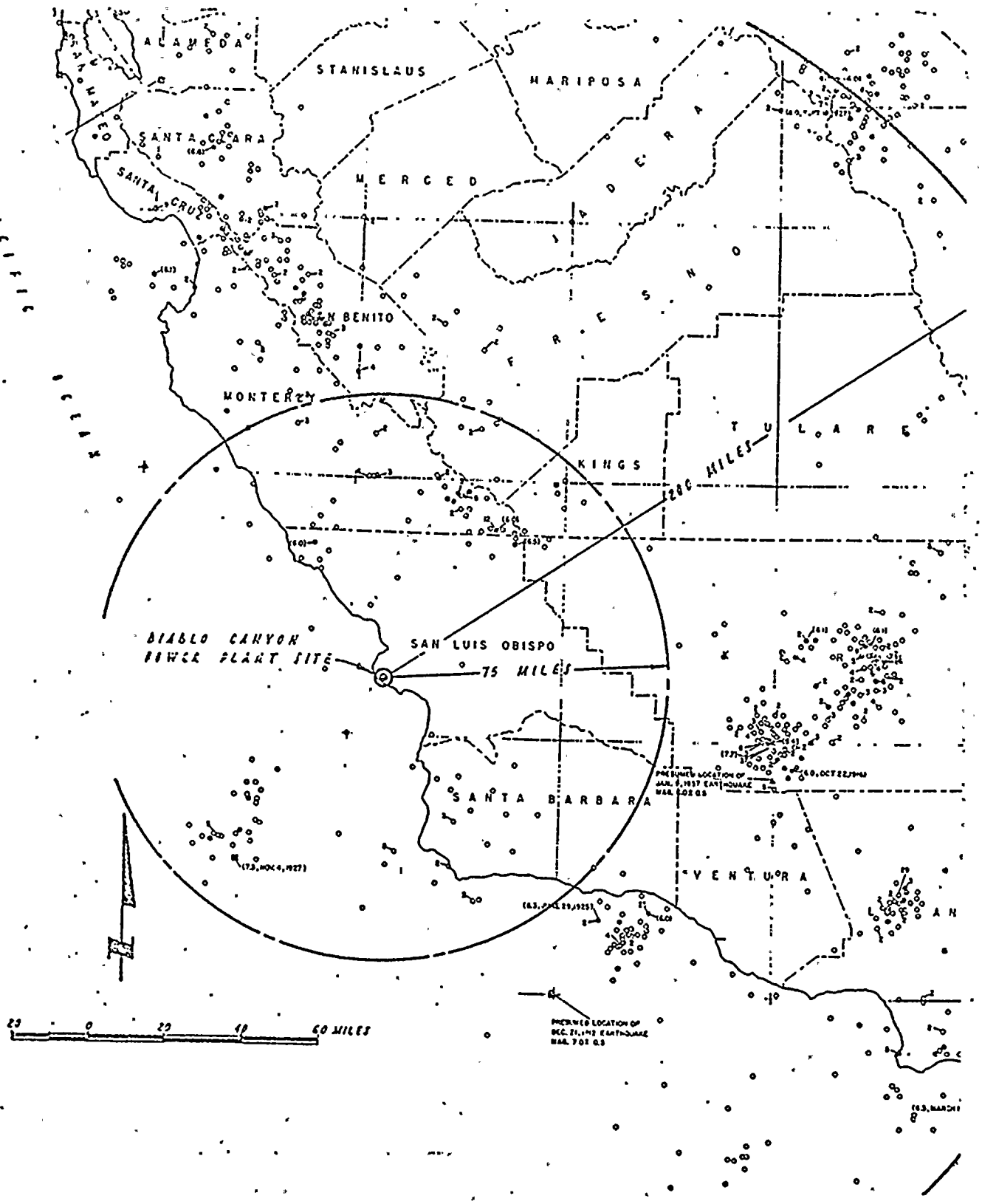

FIG. 5 EPICENTER MAP OF REGION

(Reproduced from Final Safety Analysis Report for Diablo Canyon Plant)

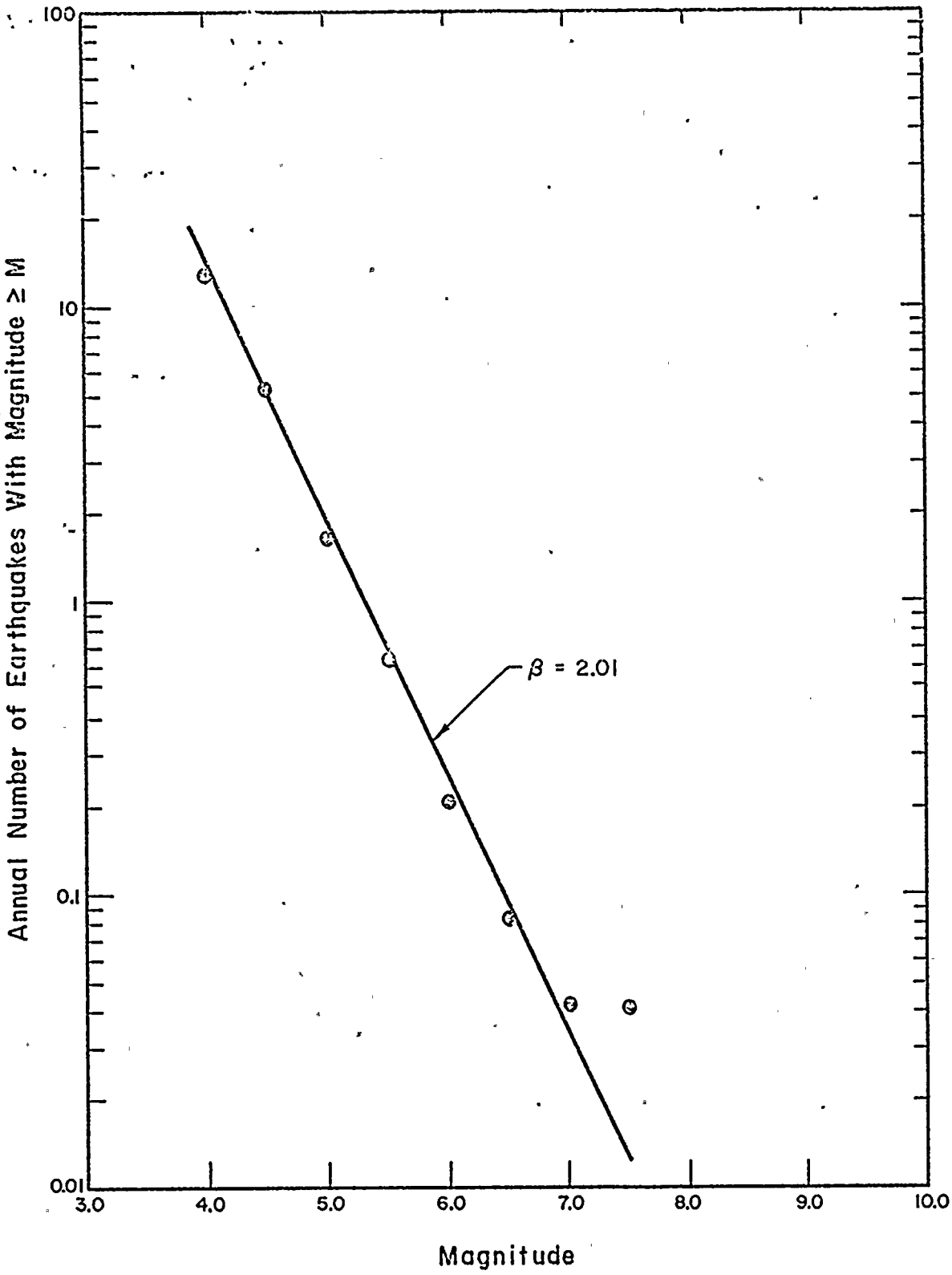

FIG. 6 MAGNITUDE RECURRENCE CURVE

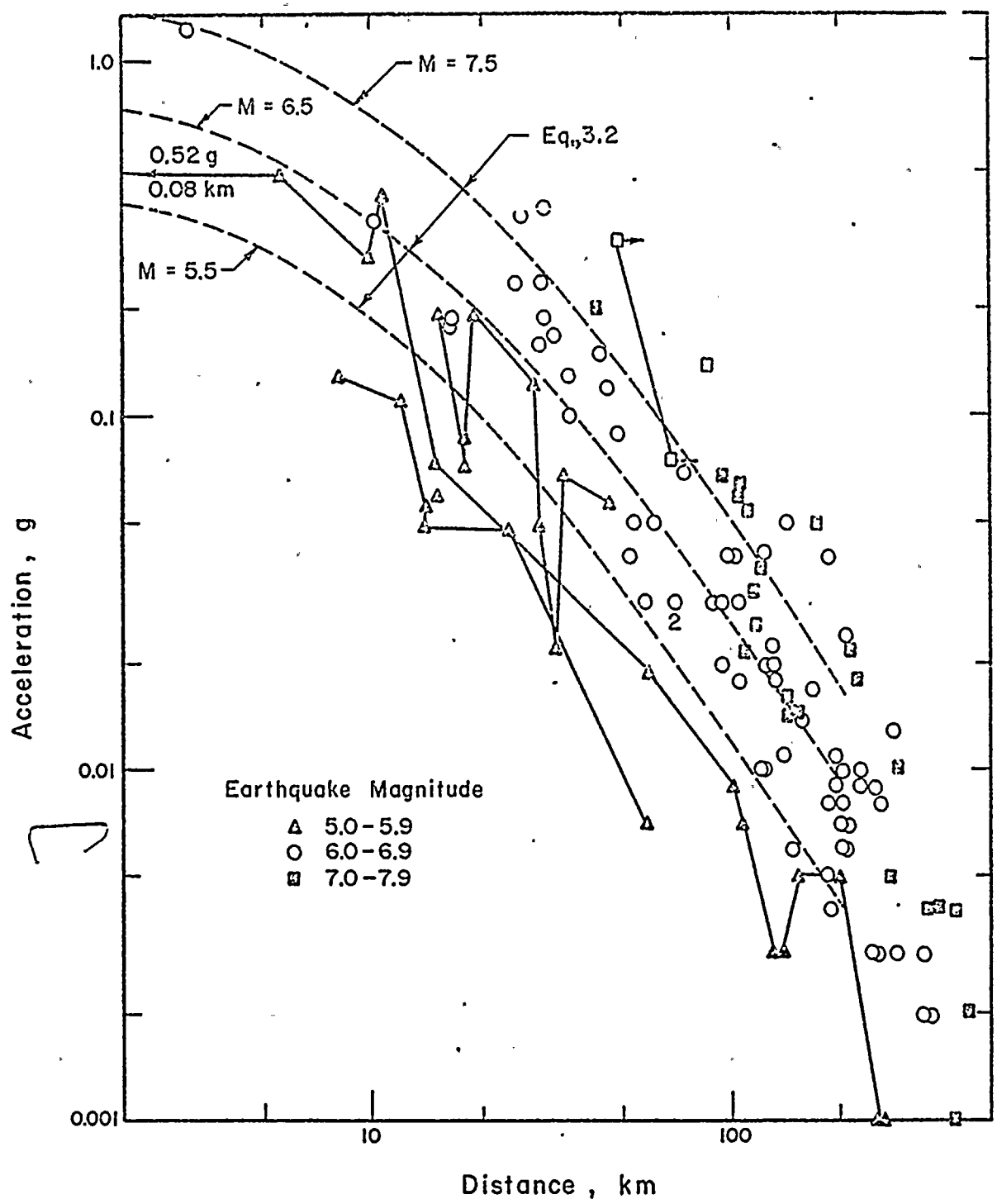

FIG. 7 ATTENUATION OF MAXIMUM ACCELERATION WITH DISTANCE TO SLIPPED FAULT (DATA REPRODUCED FROM PAGE, et al, 1972)

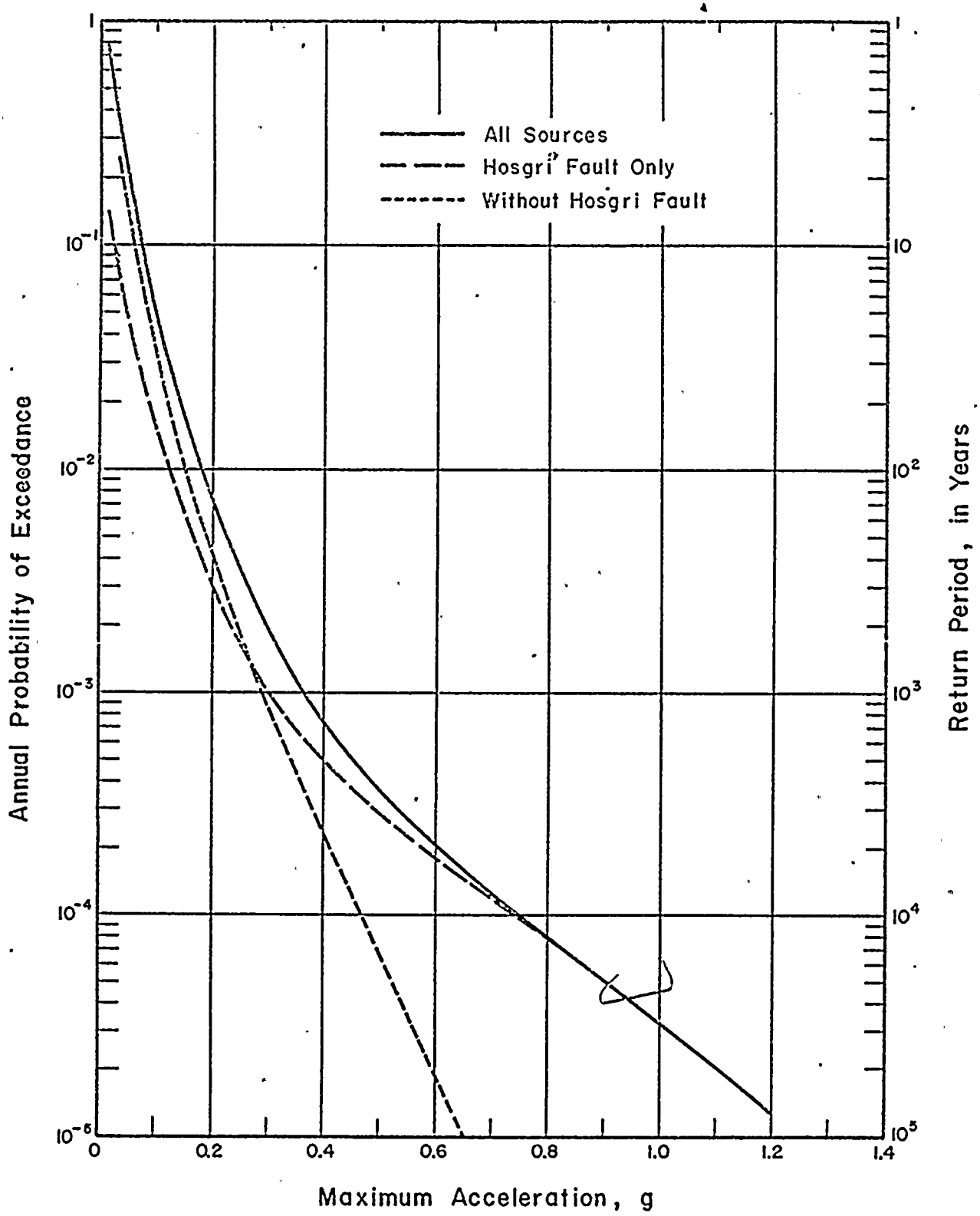

FIG. 8 SEISMIC HAZARD CURVES FOR DIABLO CANYON POWER PLANT SITE