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Probability of Pipe Failure in the Reactor Coolant Loops of Combustion Engineering PWR Plants

Vol. 3: Double-Ended Guillotine Break Indirectly Induced by Earthquakes

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ABSTRACT

The requirements to design nuclear power plants for the effects of an instantaneous double-ended guillotine break (DEGB) of reactor coolant loop (RCL) piping have led to excessive design costs, interference of normal plant operation and maintenance, and unnecessary radiation exposure of plant maintenance personnel. This report describes an aspect of the NRC/Lawrence Livermore National Laboratory sponsored research program aimed at investigating whether the probability of DEGB in RCL Piping of nuclear power plants is acceptably small and the requirements to design for the DEGB effects (e.g., provision of pipe whip restraints) may be removed. This study estimated the probability of indirect DEGB in RCL piping as a consequence of seismic-induced structural failures within the containment of Combustion Engineering supplied pressurized water reactor nuclear power plants in the United States. The median probability of indirect DEGB was estimated to be in the range of 10^{-6} per year for older plants, and less than 10^{-8} per year for modern plants; using very conservative assumptions, the 90% subjective probability value (confidence) of P_{DEGB} was found to be less than $5x10^{-5}$ per year for older plants and less than $3x10^{-7}$ per year for modern plants.

Key words: Design; Fragility; Guillotine Break; Pipes; Pipe Whip Restraints; Pressurized Water Reactor; Probabilistic Analysis; Reliability; Reactor Coolant Loop; Seismic Hazard; Seismic Response; CE Reactors.

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BACKGROUND

Currently, nuclear power plants are required to be designed for the effects of the unlikely event of double-ended guillotine break (DEGB) of the reactor coolant loop (RCL) piping, and the DEGB and the Safe Shutdown Earthquake (SSE) events are to be considered to occur simultaneously. This requirement has led to excessive design costs (i.e., provision of pipe whip restraints), interference of normal plant operation and unnecessary radiation exposure of plant maintenance personnel. The present work is part of an NRC directed research program, the Load Combination program, at the Lawrence Livermore National Laboratory (LLNL) to estimate the probability of a DEGB of RCL piping. The objective of the program was to recommend changes to the current regulatory requirements if the probability of DEGB is found to be extremely small.

Earthquakes are considered to be the only plausible cause for indirect DEGB of RCL piping. Two broad classes of DEGB induced by earthquakes have been identified. The directly-induced DEGB is the double-ended pipe break of RCL piping due to fatigue crack growth under the combined effects of thermal, pressure, seismic, and other cyclic loads. The indirectly-induced DEGB is the RCL pipe break due to causes other than direct such as support structural failures, missiles, and transient events caused by earthquakes. The indirectly-induced DEGB is the topic of this report.

TECHNICAL APPROACH

A methodology for estimating the probability of DEGB indirectly-induced by structural failures under earthquakes developed in a previous phase of the program was applied to the Combustion Engineering reactors. The key elements of the methodology are seismic hazard analysis, seismic response analysis, fragility evaluation for critical structural elements and analysis of reactor coolant loop integrity following structural failures. The uncertainties in seismic hazard, and in seismic responses and capacities are explicitly treated in this methodology to produce subjective probability bounds on the estimated probability of DEGB. By reviewing the plant arrangement and design bases for CE reactors, it was concluded that a failure of a primary equipment support (i.e., reactor pressure vessel, steam generator or reactor coolant pump) would lead to DEGB of RCL piping. Fragility descriptions of these supports were developed using information on plant design criteria and by appropriately extrapolating the responses calculated at the design analysis stage to the failure levels of structural elements of supports. Fragility was expressed in terms of a factor of safety over the SSE peak ground acceleration. The median factor of safety $\breve{F}$ and the variability estimates $\beta_{F,R}$ and $\beta_{F,U}$ were calculated.

The probability of indirectly-induced DEGB in RCL piping was estimated using the fragility descriptions and a set of seismic hazard curves appropriate to the particular site. Site-specific seismic hazard curves were used where available. For the CE reactor sites east of the Rocky Mountains, generic seismic hazard curves developed in a previous phase of this research program were utilized.

The median probability of indirect DEGB was estimated to be in the range of 10^{-6} per year for older plants, and less than 10^{-8} per year for modern plants; using very conservative assumptions, the 90% subjective probability (confidence) value of P_{DEGB} was found to be less than $5x10^{-5}$ per year for older plants and less than $3x10^{-7}$ per year for modern plants.

Based on the insights gained and the results of this study, the following conclusions are derived:

1. The probability of indirectly-induced DEGB in RCL piping due to earthquakes is very small for CE reactors.

2. Sensitivity studies have shown that only very unlikely design and construction errors of implausible magnitude may substantially change the probability of DEGB indirectly-induced by earthquakes calculated in this study.

ACKNOWLEDGEMENTS

The study reported herein was performed by Structural Mechanics Associates, Inc. under a subcontract from the Lawrence Livermore National Laboratory. The authors acknowledge the guidance and valuable comments of Drs. C. K. Chou, G. S. Holman and T. Y. Lo of the Lawarence Livermore National Laboratory, and Dr. J. J. Johnson of Structural Mechanics Associates, Inc.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The Code of Federal Regulations requires that structures, systems, and components important to the safety of nuclear power plants in the United States be designed to withstand appropriate combinations of effects of natural phenomena, normal situations, and accident conditions. One of the loading conditions that has been formulated on the basis of these federal regulations is the consideration of double-ended guillotine break (DEGB) of the reactor coolant loop (RCL) piping and the combination of its effects with those of the Safe Shutdown Earthquake (SSE). This requirement has led to excessive design costs (i.e., provision of pipe whip restraints), interference of normal plant operation and unnecessary radiation exposure of the plant maintenance personnel. Since some of the operating plants have not been designed for this loading condition, extensive plant modifications may be necessary to meet this design requirement. In order to judge the need for DEGB requirements, the NRC directed a research program, the Load Combination Program, at the Lawrence Livermore National Laboratory (LLNL), to estimate the probability of a DEGB of RCL piping. The first phase of the program addressed the issue for Westinghouse (W) PWR plants. The present phase of the program is concentrating on the PWRs supplied by Combustion Engineering (CE). The objective of the program is to recommend changes to the current regulatory requirements if the probability of DEGB is found to be acceptably small. If the probability of DEGB is acceptable, it may no longer be necessary to consider 1) asymmetric blowdown loading, 2) combination of SSE and DEGB loads and 3) installing and maintaining pipe whip restraints for the RCL piping.

Earthquakes are considered to be the only plausible cause for indirect DEGB of RCL piping. Two broad classes of DEGB induced by earthquakes have been identified. The directly-induced DEGB is the break of RCL piping due to fatigue crack growth under the combined effects of thermal, pressure, seismic, and other cyclic loads. The indirectly-induced DEGB is the break of RCL piping due to causes such as structural failures, missiles, electrical failures and transient events caused by earthquakes. Of these, the only credible source of indirectly-induced DEGB would be structural failures within the containment. This report discusses the indirectly-induced DEGB of RCL piping only.

1.1.1 Study on Westinghouse Reactors

In the first phase of the Load Combination Program, the probability of indirectly-induced DEGB in RCL piping of Westinghouse reactors was evaluated (Ravindra, et al, 1983). A methodology for calculating this probability, P_{DEGB}, was developed using Zion Nuclear Generating Station as a pilot plant. It was concluded that failure of the supports of the reactor pressure vessel, reactor coolant pump, or steam generator may potentially cause a DEGB of the reactor coolant loop piping. In the pilot study on the Zion Nuclear Generating Station, the median capacities and responses of these supports were calculated by conducting detailed seismic response analysis and failure mode evaluation. The variabilities representing inherent randomness and uncertainty were estimated. Using the site-specific seismic hazard curves, the probability of indirect DEGB was evaluated. The median probability of indirect DEGB was obtained as 1.3×10^{-8} per year; the 10 and 90 percent subjective probability bounds on this probability were estimated as 4.1×10^{-10} and 3.5×10^{-7} per year, respectively.

A generic study on 46 Westinghouse supplied PWRs was performed to extend the results of the Zion pilot study. A set of generic seismic hazard curves deemed to be applicable for sites located east of the Rocky Mountains was developed using published site-specific seismic hazard studies. Westinghouse provided data on the seismic design parameters and SSE design margins for the reactor coolant loop design of each reactor unit. Since these units were designed for a variety of response spectra and zero period peak ground acceleration using different methods of analysis and damping values, the design margins were reassessed to put them on a consistent basis. The total population of Westinghouse reactor units were classified into two groups:

- Units with primary equipment supports designed by W.
- Units with primary equipment supports designed by the architect-engineer.

In each group plants, the plant with lowest margin was selected for further study. Detailed information on design of the plant and inherent safety margins in the ASME code were used in estimating the factors of safety available against SSE for equipment supports in these selected plants. Using the generic seismic hazard curves and the factors of safety for equipment supports, the median annual probability of indirect DEGB was estimated as 3.3×10^{-6} per year and 2.4×10^{-6} per year for the two selected plants. The 10% to 90% subjective probability bounds on this DEGB probability was approximately 2.0×10^{-7} to 2.0×10^{-5} per year.

From the plants located in the Western U.S., Diablo Canyon and San Onofre Unit 1 were selected for estimation of the indirect DEGB probability. Site-specific hazard curves and seismic margins calculated in the reevaluations of these plants were used for this purpose. The median probability of indirect DEGB was estimated to be about 3×10^{-6} per year. The 10% to 90% subjective probability range of this probability was estimated as approximately 2×10^{-7} per year to 6×10^{-5} per year.

This study on Westinghouse reactors showed that the probability of indirect DEGB in RCL piping due to earthquakes is very small and that the failure of some major equipment supports has a high likelihood of rupturing the RCL piping inside the reactor cavity (i.e., between the shield wall and RPV).

1.1.2 Reactor Coolant Loop Arrangement in CE Reactors

The reactor coolant system in a CE reactor typically consists of two loops and includes the reactor vessel, two steam generators, four reactor coolant pumps and the pressurizer. With the exception of one plant (Ft. Calhoun), all RCL piping is fabricated from carbon steel. The reactor vessel is supported on three or four nozzles of the cold legs. The support system of the RPV consists of a nozzle pad, usually supported by means of columns extending to the base mat. The steam generator has a skirt support with a sliding base; its upper support consists of a key and a snubber assembly. The reactor coolant pump is supported at the top by horizontal snubbers and struts. At the pump skirt level, the vertical support is either spring hangers, snubbers or vertical columns; the horizontal support is given by means of horizontal struts or snubbers.

Figures 1-1 through 1-6 give details of the RCL arrangement at the reference plant for our study, the Palo Verde Nuclear Generating Station (PVNGS) operated by the Arizona Public Service Company.

The design criteria for the CE reactor units have evolved over the years; the very early plant(s) was designed using static analysis whereas the more modern plants have been analyzed using coupled time history analysis with three-directional seismic input. Although all plants have been designed for guillotine and slot breaks in the RCL piping, only recent plants have been designed for the full effects of DEGB (i.e., asymmetric blowdown, and SSE and DEGB load combination). Because of these large differences in the plant design criteria, the indirect DEGB study presented in this report treated each plant separately.

1.2 GENERAL APPROACH

1.2.1 Objective and Scope

The objective of the present study was to evaluate the probability of seismically-induced indirect DEGB in the reactor coolant loop piping of CE reactors. The study consisted of the following major tasks:

1. Review the seismic hazard curves for the plants located in the western United States.

2. Perform a walk-through inspection of the reference plant (i.e., Palo Verde) with the objectives of becoming familiar with the equipment support arrangement in a CE plant and identifying components within the containment whose failure under earthquakes may induce a DEGB of RCL piping.

3. Using the information provided by CE on seismic margins and plant design criteria, estimate the seismic capacities and realistic responses taking into account the differences between the current state-of-the-art and the methods of analysis used in design.

4. Calculate the probability of indirect DEGB using the relevant seismic hazard curves and the information generated in Step 3.

1.2.2 Plants Studied

CE classified the reactors based on the loop arrangement and design criteria into four groups (Maine Yankee is group D, but is excluded from this report for lack of information):

Group A	Group B	Group C
Palisades	Ft. Calhoun	San Onofre 2 & 3
Calvert Cliffs		Waterford 3
Millstone 2		Palo Verde 1,2,&3
St. Lucie 1 & 2		WPPS 3

Group A consists of early plants with three nozzle supports for the RPV, and sliding base for steam generator. The RC pump supports varied with the Palisades pump supports not designed for seismic loading. There is a total of ten (10) snubbers provided for equipment attached to RCL piping. The Group B plant - Ft. Calhoun - has the RCL piping made of stainless steel. Group C consists of modern plants with four nozzle supports on the RPV.

The steam generator has a sliding base with a snubber-lever-assembly at the top. The reactor coolant pump is designed for seismic and pipe rupture loads. The plants in this group are designed for all the effects of DEGB (i.e., asymmetric blowdown and SSE+DEGB load combination). In this group of plants, there are about 4-6 snubbers for each plant.

1.3 OUTLINE OF THE REPORT

The technical approach developed in this study is described in Chapter 2. A general methodology for estimating the probability of seismically-induced indirect DEGB in RCL piping is outlined. The major elements of this methodology are seismic hazard analysis, seismic fragility evaluation and assessment of consequences of structural failures within the containment on the RCL piping. The plant design information provided by CE is discussed. The generic seismic hazard curves and the site-specific hazard studies are briefly discussed. As an illustration of the methodology, the calculations performed for evaluating the probability of indirect DEGB in Palo Verde RCL piping are described.

The results of this study are provided in Chapter 3. A comparison with previous phases of this program is given. Sensitivity of the results to seismic hazard assumptions and potential design and construction errors is discussed. The chapter ends with a summary of the study and significant conclusions.

Appendix A gives an example of the plant design information obtained from CE. The data on Palo Verde Nuclear Generating Station is included. Appendix B describes the quality assurance procedures used by CE in the design, construction and inspection of RCL piping and equipment supports.

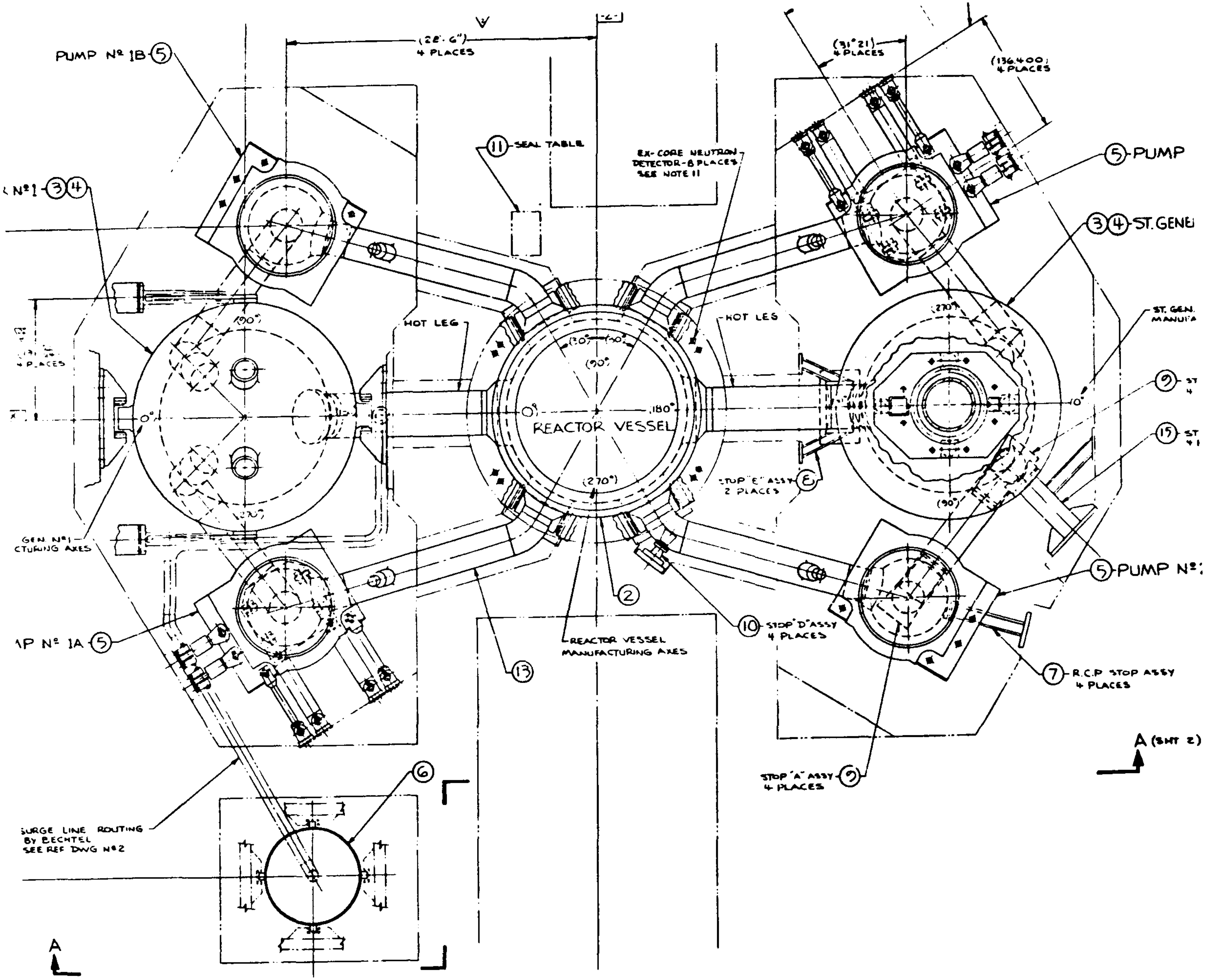

FIGURE 1-1. PALO VERDE REACTOR COOLANT LOOP ARRANGEMENT - PLAN

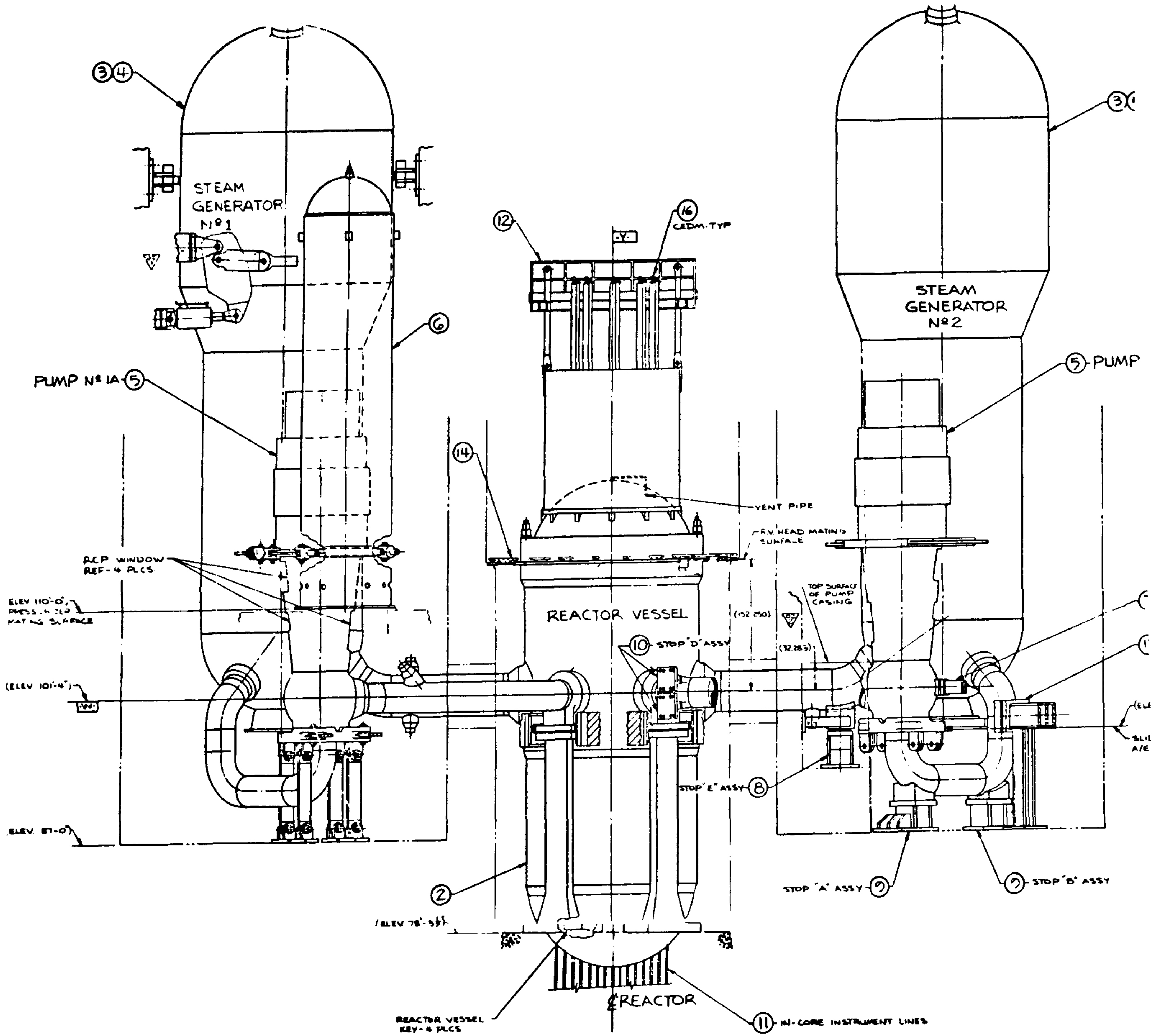

FIGURE 1-2. PALO VERDE REACTOR COOLANT LOOP ARRANGEMENT - ELEVATION

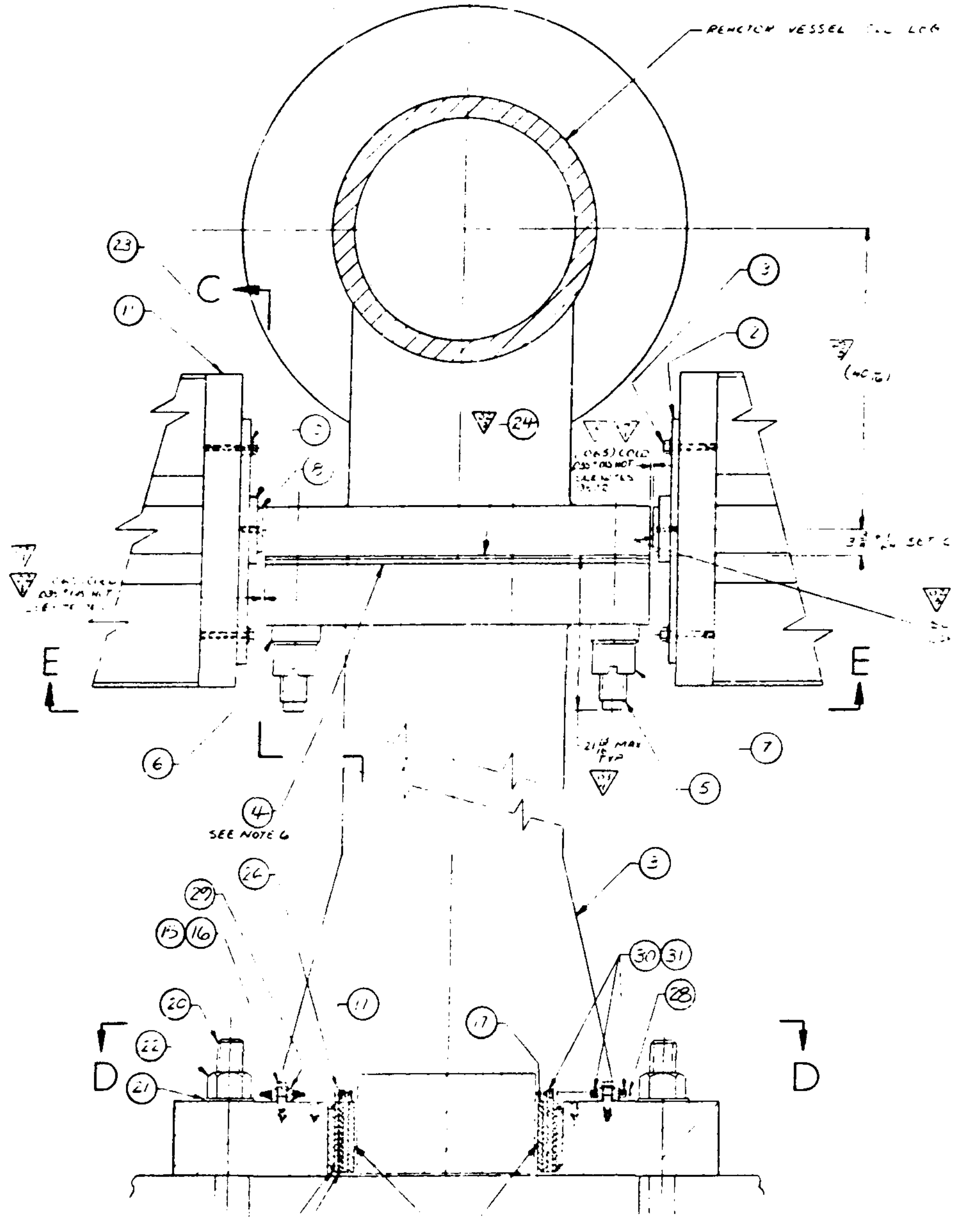

FIGURE 1-3. REACTOR PRESSURE VESSEL SUPPORT

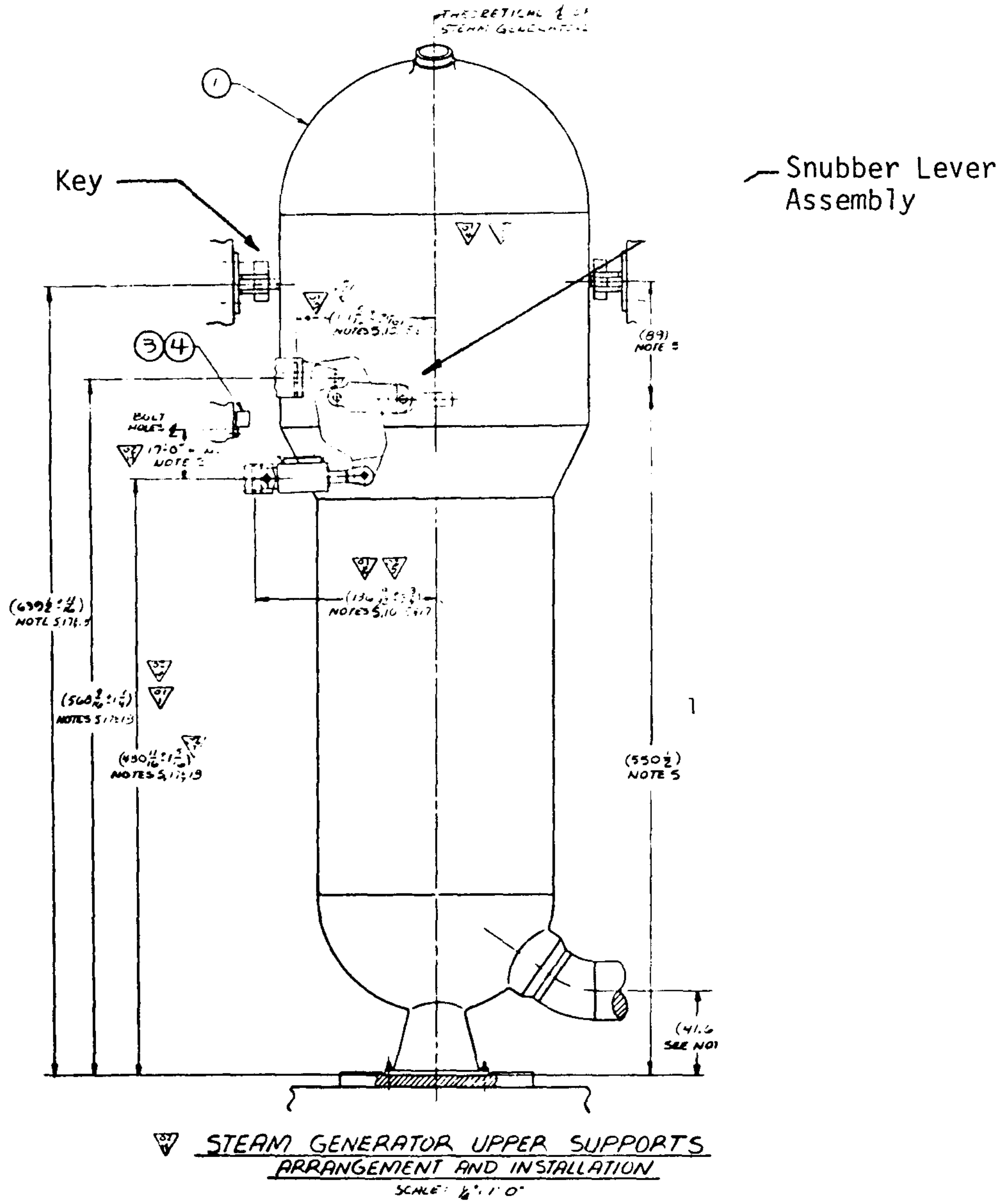

FIGURE 1-4. STEAM GENERATOR UPPER SUPPORTS

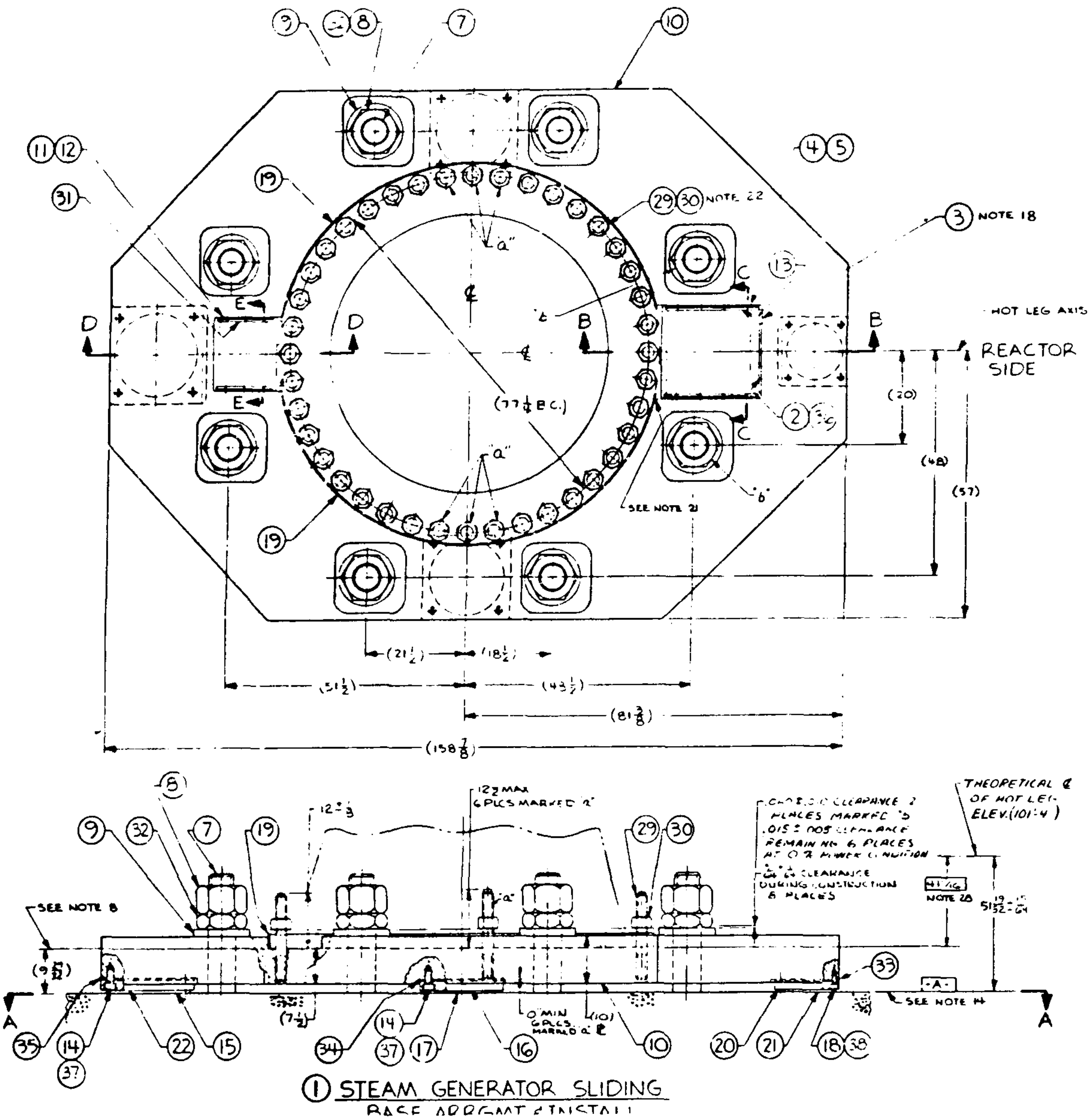

FIGURE 1-5. STEAM GENERATOR SLIDING BASE

CHAPTER 2

TECHNICAL APPROACH

In this chapter, we describe the analytical approach pursued in the calculation of the probability of indirect DEGB induced by structural failures under earthquakes. A general methodology is first presented. The key elements of the methodology are seismic hazard analysis and evaluation of the fragility of equipment supports whose failure might lead to a DEGB of RCL piping.

2.1 METHODOLOGY

The objective of the present study is to calculate the probability of DEGB as a result of structural failures which are induced by an earthquake. This probability, P_{DEGB}, can be mathematically expressed as:

$$P_{DEGB} = \int_0^{\infty} P\left[\bigcup_{i=1}^{n} (C_i < R_i) \middle| A = a\right] f_A(a)\, da \qquad (2\text{-}1)$$

where

C_i = capacity of a structural element i (e.g., reactor pressure vessel column, steam generator support snubber, and reactor coolant pump horizontal strut support); i=1, 2, ..., n; a random variable.

R_i = seismic response of element i due to an earthquake of peak ground acceleration a; a random variable.

$\bigcup_{i=1}^{n}$ = "Union" symbol.

$f_A(a)$ = frequency of occurrence of peak ground acceleration at the site between a and a+da.

Equation 2-1 is written assuming that there is perfect knowledge about the values of the parameters that define the probability terms. Since there is uncertainty in these parameter values, a subjective probability distribution of the probability of indirectly-induced DEGB will be obtained by appropriately varying the parameter values as will be subsequently described.

The first term within the integral of Equation 2-1 is the conditional probability of occurrence of DEGB due to structural failures for a given peak ground acceleration, a. It is defined as the probability of failure of at least one of the structural elements which can lead to DEGB of RCL piping. Therefore, the focus in this study is only on those structural elements within the containment whose failure can result in DEGB. Among these, some elements may have large margins of safety against seismic failure and thus may not contribute significantly to the probability of DEGB. Therefore, critical elements are defined as those whose failure could contribute significantly to the probability of indirectly-induced DEGB. These are identified as the steam generator supports, the reactor coolant pump supports, and the reactor pressure vessel supports.

The conditional probability of DEGB is evaluated by treating the failure events of individual structural elements as statistically independent and it is derived from the conditional probabilities of failure of these structural elements. This gives a conservative upper bound on the probability of DEGB. Also, if one of the structural elements has a very high conditional probability of failure compared to other elements, the upper bound is a good approximation to the actual P_{DEGB}.

2.1.1 Seismic Fragility

The conditional probability of failure of a structural element for a given peak ground acceleration is called the seismic fragility of the element (Figure 2-1). The fragility evaluation is accomplished in this study using information on plant design bases and by appropriately extrapolating the responses calculated at the design analysis stage to the failure levels of the structural elements.

Evaluation of the fragility is simplified by defining a random variable called the ground acceleration capacity. The ground acceleration capacity, A_c, is expressed as:

$$A_c = F \cdot A_{SSE} \tag{2-2}$$

where F is the factor of safety on the design basis earthquake (e.g., safe shutdown earthquake) and A_{SSE} is the peak ground acceleration specified for SSE. The factor of safety is defined as a ratio of the seismic capacity of the structural element, C_i, to the response, R_i, due to SSE. Since C_i and R_i are random variables, the factor of safety, F, is also a random variable.

The factor of safety, F, is modeled as a lognormally distributed random variable with the parameters, median $\breve{F}$ and logarithmic standard deviation, β_F. Two basic types of variability are identified (Kennedy, et al, 1980) in describing the factor of safety; one that represents the inherent randomness and the other which represents the uncertainty in the parameter value, e.g., the median. These variabilities are quantified by the logarithmic standard deviations, $\beta_{F,R}$ and $\beta_{F,U}$, respectively. Essentially, $\beta_{F,R}$ represents the variability due to randomness of earthquake characteristics for the same peak ground acceleration and to the randomness of the structural response parameters which relate to these characteristics. The dispersion represented by $\beta_{F,U}$ is due to such factors as:

1. Our lack of understanding of structural material properties such as strength, inelastic energy absorption capacity and damping, and

2. Errors in calculated response due to use of approximate modeling of the structure and equipment and inaccuracies in mass and stiffness representations.

For equipment supports, the factor of safety can be modeled as the product of the three random variables (Kennedy and Ravindra, 1983):

$$F = F_C \, F_{RS} \, F_{RE} \tag{2-3}$$

The capacity factor, F_C, for the equipment support is a product of a strength factor, F_S, and an inelastic energy absorption factor, F_μ.

The strength factor, F_S, represents the ratio of ultimate strength to the stress calculated for A_{SSE}. In calculating the value of F , the non-seismic portion of the total load acting on the support is subtracted from the strength as follows:

$$F_S = \frac{S-P_N}{P_T-P_N} \tag{2-4}$$

where S is the ultimate structural strength for the specific failure mode, P_N is the normal operating load (i.e., dead load, operating temperature load, etc.), and P_T is the total load on the support (i.e., sum of the seismic load for A_{SSE} and the normal operating load). For higher levels of earthquake, other transients (e.g., turbine trip) may have a high probability of occurring simultaneously with the earthquake; the definition of P_N in such cases should be extended to include the loads from these transients.

The strength, S, is a function of the failure mode i.e., brittle or ductile modes. Brittle failures are defined as those failure modes which have little or no system inelastic energy absorption capability. Examples are:

1. Anchor bolt failures
2. Support weld failures
3. Shear pin failures

Each of these failure modes has the ability to absorb some inelastic energy on the component level, but the plastic zone is very localized, and the system ductility for an anchor bolt or a support weld is very small. The strength of the component failing in a brittle mode is therefore calculated using the ultimate strength of the material.

Ductile failure modes are those in which the structural system can absorb a significant amount of energy through inelastic deformation. Examples include:

1. Pressure boundary failure of piping
2. Primary equipment supports failing in tension

The strength of the element failing in a ductile mode is calculated using the yield strength of the material for tensile loading. For flexural loading, the strength is defined as the limit load or load to develop a plastic hinge.

The inelastic energy absorption factor, F_μ , for an equipment support is a function of the ductility ratio, μ and damping, δ. The median value $\check{F}_\mu$ is considered to be close to 1.0 for brittle and functional failure modes. For ductile failure modes of equipment support that respond in the amplified acceleration region of the design spectrum (i.e., 2 to 8 Hz) the inelastic energy absorption factor is calculated using the procedure given in Riddell and Newmark (1979).

The median $\check{F}_C$ and the variability estimates, $\beta_{C,R}$ and $\beta_{C,U}$ of the capacity factor are obtained as follows:

$$\check{F}_C = \check{F}_S \check{F}_\mu \tag{2-5}$$

$$\beta_{C,R} = (\beta^2_{S,R} + \beta^2_{\mu,R})^{1/2} \tag{2-6}$$

$$\beta_{C,U} = (\beta^2_{S,U} + \beta^2_{\mu,U})^{1/2} \tag{2-7}$$

where

$\check{F}_S$ = median strength factor

$\check{F}_\mu$ = median inelastic energy absorption factor

$\beta_{S,R}$ = logarithmic standard deviation of the randomness in the strength factor.

$\beta_{S,U}$ = logarithmic standard deviation of the uncertainty in the median value of strength factor.

$\beta_{\mu,R}$ = logarithmic standard deviation of the randomness in the inelastic energy absorption factor.

$\beta_{\mu,U}$ = logarithmic standard deviation of the uncertainty in the median value of the inelastic energy absorption factor.

The structure response factor, F_{RS}, recognizes that in the design analyses, the structural response was computed using specific (often conservative) deterministic response parameters for the structure. Because many of these parameters are random (often with a wide variability), the actual response may differ substantially from the design analyses calculated response for a given peak ground acceleration level.

The structural response factor, F_{RS}, is modeled as a product of factors influencing the response of variability.

$$F_{RS} = F_{SA} F_{\delta} F_M F_{SD} F_{SS} \tag{2-8}$$

where

F_{SA} = spectral shape factor representing the variability in ground motion and the associated ground response spectra.

F_{δ} = damping factor representing the variability in response due to difference in actual damping and design damping.

F_M = modeling factor accounting for the uncertainty in response due to modeling assumptions.

F_{SD} = factor to reflect the reduction of seismic input with depth.

F_{SS} = factor to account for the effect of soil-structure interaction.

The median $\check{F}_{RS}$ and the variability estimates $\beta_{RS,R}$ and $\beta_{RS,U}$ are calculated using Equation 2-5 and the properties of lognormal probability law:

$$\check{F}_{RS} = \check{F}_{SA} \check{F}_{\delta} \check{F}_M \check{F}_{SD} \check{F}_{SS} \tag{2-9}$$

$$\beta_{RS,R} = (\beta^2_{SA,R} + \beta^2_{\delta,R} + \beta^2_{M,R} + \beta^2_{SD,R} + \beta^2_{SS,R})^{\frac{1}{2}} \tag{2-10}$$

A similar expression exists for $\beta_{RS,U}$.

The equipment response factor, F_{RE}, is the ratio of equipment response calculated in the design to the realistic equipment response; both the responses are calculated for the design floor spectra. F_{RE} is the factor of safety inherent in the computation of equipment response. It depends upon the response characteristics of the equipment and is influenced by the variables listed below. F_{RE} is modeled as:

$$F_{RE} = F_{SA} F_{\delta} F_M F_{MC} F_{EC} \tag{2-11}$$

F_{SA} = Spectral shape factor - including the effects of peak broadening and smoothing, and artificial time history generation.

F_{δ} = Damping factor.

F_M = Modeling factor (affects mode shape and frequency results).

F_{MC} = Factor to account for margins in combination of modal responses.

F_{EC} = Factor to account for margins in combination of earthquake components.

The median $\check{F}$ and the variability estimates, β_R and β_U of the equipment response factor are obtained using Equation 2-11 and the properties of the lognormal probability law as described above.

With the overall factor of safety F estimated as described above, the ground acceleration capacity of the structural element is calculated using Equation 2-2.

$$\check{A}_C = \check{F} A_{SSE} \tag{2-12}$$

$$\check{F} = \check{F}_C \check{F}_{RS} \check{F}_{RE} \tag{2-13}$$

$$\beta_{A,R} = \beta_{F,R} = (\beta^2_{C,R} + \beta^2_{RS,R} + \beta^2_{RE,R})^{1/2} \tag{2-14}$$

$$\beta_{A,U} = \beta_{F,U} = (\beta^2_{C,U} + \beta^2_{RS,U} + \beta^2_{RE,U})^{1/2} \tag{2-15}$$

The overall factor of safety is thus decomposed into factors that we can model and for which we have data and information. In some instances, evaluating β values exactly would require detailed analysis and/or more extensive data than is available. For these cases, it is sometimes necessary to use subjective evaluations and engineering judgment to evaluate the β values. As an example, consider the case for which the median value of the factor is known and a lower bound value, below which it is fairly unlikely that the factor will fall, is also known. Given that the factor is lognormally distributed, the β value may be evaluated by assuming the lower bound to be, say, a 5 percentile value. Although this procedure is subjective, it is generally observed that changes in the β value for the particular factor have a small effect on the final probabilities calculated (Ravindra, et al, 1984). This results from the fact that the β's of the overall safety factor are the SRSS of many β's (Equations 2-14, 2-15) of similar magnitude and therefore, insensitive to minor variations in the individual β's. Also, the seismic hazard uncertainty tends to dominate the final analysis variability, making the calculated probabilities relatively insensitive to minor changes in the β values estimated.

The ground acceleration capacity of each equipment support was modeled in this study as the lowest capacity in all credible failure modes. This is a realistic assumption since the failure modes are highly correlated due to common structural material and method of fabrication. Again, if the structural element is one of the failure modes has a very low capacity compared to other modes, this assumption leads to a good approximation of the probability distribution of the capacity.

2.1.2 Seismic Hazard

The last term within the integral of Equation 2-1, $f_A(a)da$, is the probability that the peak ground acceleration at the site in a year is between a and a+da. This is usually described by a set of seismic hazard curves (Figure 2-2) where each curve is a plot of the annual exceedence probability versus peak ground acceleration. The uncertainty in hazard curves is presented by developing a family of curves and assigning a subjective weighting factor (or probability) to each curve.

2.1.3 Calculation of DEGB Probability

Equation 2-1 was evaluated in this study using the SMA computer program SEISRISK. The program first combines the individual component fragilities into a plant level fragility (i.e., union operation in this case) and then convolves the plant level fragility with the family of seismic hazard curves to obtain the subjective probability distribution of the probability of DEGB indirectly-induced by earthquakes (Figure 2-3).

2.2 DESIGN INFORMATION PROVIDED BY COMBUSTION ENGINEERING

For this study, Combustion Engineering (CE) provided information on the design bases and features of the reactor coolant loop and primary equipment supports at all the nuclear power plants with CE reactors. As explained in Section 1.3, the reactors were grouped into four categories and the design information was obtained on Groups A, C and D.

2.2.1 Information on Seismic Hazard

Site-specific seismic hazard studies were performed for CE on the Palo Verde Nuclear Station (by Ertec, 1982) and the San Onofre Nuclear Generating Station (New Mexico Engineering Consultants, 1983). CE provided the results of these studies in the form of seismic hazard curves to assist in the DEGB probability evaluation.

2.2.2 RCL Equipment Support Details

CE furnished the engineering drawings of the reactor coolant loop in each plant showing details of primary equipment supports. These drawings were reviewed in this project to identify critical elements in equipment supports and to assess the effects of their seismic failures on the RCL piping.

2.2.3 Seismic Margins

CE provided the seismic margins against code allowables for each critical element in the primary equipment supports of different plants. The seismic margins were calculated as follows:

$$\text{Seismic Margin} = \left[\frac{\text{Faulted Allowable Stress} - \text{Normal Operating Stress}}{\text{Stress due to SSE}}\right] \quad (2\text{-}16)$$

In addition, information on the support material types, faulted allowable stresses and failure modes was provided. Table 2-1 is a sample of the information provided by CE on seismic margins.

2.2.4 Information on Seismic Response

The following information was provided by CE for each plant to assist in the evaluation of the response factors:

Structural Response

- Ground spectrum used for design
- Structural damping
- Site characteristics (shear wave velocity, thicknesses of different strata).
- Fundamental frequency of internal structure if uncoupled analysis was conducted.
- Interface spectra for NSSS points of connection to structure if uncoupled analysis was conducted.
- Input ground spectra resulting from synthetic time history applied to structural model.

NSSS Response

- Method of analysis (time history, response spectrum, etc.)
- Modeling of NSSS and structure (coupled or uncoupled).
- NSSS system damping.
- NSSS system fundamental frequency or frequency range.
- If uncoupled analysis was done, were envelope or multisupport spectra used?

Appendix A shows an example of the information provided by CE for the Palo Verde Nuclear Generating Station.

2.3 GENERIC SEISMIC HAZARD CURVES AND SITE-SPECIFIC SEISMIC HAZARD STUDIES

The CE reactor sites are dispersed throughout the United States. Ideally, the site-specific seismic hazard curves are needed for a realistic estimation of DEGB probability for each plant. Since such site-specific seismic hazard curves are not available for all the plants, generic seismic hazard curves were utilized where deemed appropriate.

2.3.1 Generic Seismic Hazard Curves

The generic seismic hazard curves developed for our study of Westinghouse plants located east of the Rocky Mountains (Ravindra, et al, 1983) were utilized in this study. For the sake of completeness, a brief background information on these hazard curves is given.

A total of six sites dispersed over the eastern and midwestern states were chosen. These are the sites for which formal seismic hazard analyses have been performed (Figure 2-4). Some of these analyses have been published (e.g., Zion and Indian Point Seismic Hazard Analyses). Others are part of PRA studies yet to be published. In order to preserve the anonymity of these seismic hazard studies, the plants with unpublished reports on seismic hazard studies have been labeled as A, B, C and D.

All of these seismic hazard studies have been conducted by Dr. Robin McGuire of Dames and Moore. The salient assumptions and data (i.e., seismogenic regions, attenuation functions, activity rates, and upper bound magnitudes of earthquakes) used in generating these seismic hazard curves have been reviewed thoroughly and accepted by the NRC and the peer reviewers during the Zion and Indian Point PRA studies. This methodology also explicitly treats the uncertainties in seismic hazard modeling and in the parameter values. Therefore, a family of seismic hazard curves is obtained for each site: a subjective probability value is assigned to each hazard curve to reflect the confidence in the hypothesis used to generate that curve.

Figure 2-5 shows the mean seismic hazard curves for the selected six sites. It may be observed that the mean hazard curves vary widely for different locations. It would not be appropriate to select an envelope of these mean hazard curves as the mean generic hazard curve because it would be too conservative for plants located in most parts of the eastern and midwestern United States. Also, the Safe Shutdown Earthquake (SSE) levels of these plants vary from 0.10g to 0.25g peak ground acceleration. Hence, the seismic hazard curves have to be normalized such that the peculiar features of seismicity of the region and the differences in SSE levels are not given undue importance. In this study, the hazard curves were normalized by dividing the peak ground acceleration by the larger of SSE or 0.15g. The use of 0.15g is justified because this is thought to be the currently acceptable minimum SSE in most parts of the eastern and midwestern United States. If this limit of 0.15g was not introduced, the seismic hazard at some sites would have been disproportionately amplified in the sample of the six sites studied. Figure 2-6 shows the normalized mean seismic hazard curves at the chosen six sites.

The set of generic seismic hazard curves was developed using the following procedure.

The normalized seismic hazard curves for each of the six sites were pooled together as one population consisting of 40 seismic hazard curves. The subjective probability assigned to each curve in the original set (i.e., specific to the site) was divided by six, the number of sites included in this development of generic hazard curves. This means that each site was assigned equal weight. For the ease of further computation, the total set of 40 normalized hazard curves was condensed into five generic hazard curves with subjective probabilities of 0.1, 0.2, 0.4, 0.2 and 0.1, respectively. This was done by developing a subjective probability distribution of the probability of exceedence at each specified value of X: i.e., A/(larger of SSE and 0.15g). This subjective probability distribution was discretized into five regions with probabilities of 0.1, 0.2, 0.4, 0.2, and 0.1, respectively, and the centroid (giving the annual probability of exceedence of X) of each region was determined. By repeating this procedure for each X and joining the corresponding centroids, the set of five generic seismic hazard curves was obtained.

Figure 2-7 shows the generic seismic hazard curves that were used in the present study. For display purposes, Figure 2-8 shows the median generic hazard curve and the curves corresponding to 90% and 10% exceedence subjective probabilities. At a value of X=1, (i.e., at peak ground acceleration equal to SSE or 0.15g), the median annual frequency of exceedence is 1.6×10^{-4}; the 90% to 10% exceedence subjective probability bounds on the annual probability of exceedence are 3.7×10^{-5} to 5.5×10^{-4}. These exceedence probabilities generally represent the bounds that most seismologists and hazard analysts believe are appropriate for eastern and midwestern U.S. sites. At higher values of X, these bounds become larger reflecting the greater degree of uncertainty.

Figure 2-4 shows the regions of the U.S. where the generic seismic hazard curves are deemed applicable.

2.3.2 Seismic Hazard Analysis of the Palo Verde Nuclear Generating Station

A seismic hazard analysis of the Palo Verde Nuclear Generating Station (PVNGS) was performed by Ertec Inc. (1982). Most of the data used to perform the seismic hazard analysis were obtained from information within the PVNGS PSAR and FSAR documents, and recent seismicity data compiled by the California Institute of Technology and the National Oceanic and Atmospheric Administration. The seismic hazard model used in this investigation was based on the work of Cornell (1968).

The probabilities of exceeding various levels of peak horizontal ground acceleration were calculated using the following steps:

1. Identify all faults and zones of seismicity capable of producing strong ground motion at the site (Figure 2-9).

2. Estimate the seismic activity of each of the faults and zones of seismicity within the site region based on the recorded seismicity and geologic history. The seismic activity of these seismic source zones was characterized by recurrence curves, which represent the average number of earthquakes of different magnitudes per year per unit area. A maximum magnitude of earthquake that the source (i.e., fault or seismic zone) is capable of generating was estimated for each source (Table 2-2).

3. An attenuation relationship between the peak ground acceleration at the site, earthquake magnitude, and the site to source distance was established (Joyner and Boore, 1981):

$$\log a = -1.080 + 0.249\ M - \log r - 0.00255\ r \qquad (2\text{-}17)$$

where:

a = mean peak horizontal ground acceleration in g.

M = moment magnitude

$r = (d^2 + 7.3^2)^{1/2}$

d = closest distance to the surface projection of the fault rupture in km.

The uncertainty associated with this relationship is expressed in terms of the standard deviation of the residuals, i.e., $\sigma_{\log a}$, equal to 0.26.

The results of the seismic hazard analysis for the PVNGS site are presented as three hazard curves: lower bound, best estimate and upper bound. The lower and upper bounds represent approximately 10 percent and 90 percent non-exceedence subjective probability (confidence) limits. It may be noted that the best estimate and upper bound hazard curves are terminated at 0.50g peak ground acceleration in the original seismic hazard analysis. For the purposes of calculating the probability of indirect DEGB, we generated a set of five hazard curves by interpolating within these bounds (Figure 2-10). Also, we extended these curves to cover peak ground acceleration values up to 2g.

2.3.3 Seismic Hazard Analysis of the San Onofre Nuclear Generating Station (SONGS)

Two independent studies on seismic hazard at SONGS were performed by TERA Corporation and Woodward-Clyde consultants. These studies clearly showed that the nearby Offshore Zone of Deformation postulated to lie 8 km from the site at the closest point dominates the seismic hazard at SONGS. A reconcilation of these two studies based on a critical examination of the bases and results by New Mexico Engineering Consultants (1983) led to three seismic hazard curves (at exceedence probabilities of 90%, 50% and 10%). Two are shown in Figure 2-11. Note that these curves asympotically approach 0.67g, 0.93g and 1.05g, respectively. Since these curves are reasonably close together, it was decided to use only the upper and lower bound curves and assign them equal subjective probabilities (0.5 each) in calculating the DEGB probability. This set of hazard curves is denoted SONGS Set 1 (Figure 2-11).

The asymptotic behavior of the hazard curves at 1.05g (about 1.5 times the SSE acceleration) is not universally accepted. Also, a comparison with published seismic hazard studies done for eastern and midwestern U.S. sites indicated that the above seismic hazard curves (SONGS Set 1) may be optimistic. Therefore, a second set of seismic hazard curves was developed based on the information available in the literature. Algermissen, et al (1982) have published the seismic hazard maps for the Continental United States. Using their maps, the peak ground acceleration values corresponding to annual probabilities of exceedence of 10^{-2}, 2×10^{-3} and 4×10^{-4} were obtained at the SONGS site. A hazard curve passing through these points and extrapolated loglinearly beyond 0.8g (i.e., annual exceedence probability of 4×10^{-4}) is denoted Curve #4. A seismic hazard curve developed by Ang and Newmark (1977) for the Diablo Canyon site is shown in Figure 2-11. This hazard curve is dominated by the Hosgri fault which is at 6 km from the site. The upper bound magnitude assigned to Hosgri fault was 7.5. These characteristics of the fault are similar to the Offshore Zone of Deformation postulated for SONGS site. Therefore, the hazard curve developed by Ang and Newmark (curve denoted #3) was considered applicable to the SONGS site. The three hazard curves #2, #3, and #4 were assigned equal subjective probability (0.33 each) and considered to form a seismic hazard set - SONGS Set 2.

2.3.4 WPPSS Seismic Hazard Curves

For the present study, no site-specific seismic hazard curves were available for the WPPSS site. Therefore, the seismic hazard maps published by Algermissen, et al (1982) were utilized to develop the seismic hazard curves for the WPPSS site. The middle curve in Figure 2-12 is based on the peak ground acceleration values reported by Algermissen, et al (1982) corresponding to annual probabilities of exceedence of 10^{-2}, 2×10^{-3} and 4×10^{-4}. This curve was considered to be the best estimate of the seismic hazard at the site. The work of Algermissen, et al (1982) did not consistently treat the uncertainty in attenuation relationship and did not consider the uncertainty in maximum magnitude and seismic source modeling. A review of available seismic hazard studies indicated that the uncertainty in the peak ground acceleration at a given annual probability of exceedance can be represented by the logarithmic standard deviation of $\sigma_{\ell na} = 0.45$. Using this value and the curve given by Algermissen, et al (1982) as the median curve, the seismic hazard at the WPPSS site was portrayed by five seismic hazard curves. The subjective probabilities (confidence) assigned to these curves were calculated using a lognormal distribution with the above median and $\sigma_{\ell na}$.

2.4 EXAMPLE OF INDIRECT DEGB PROBABILITY CALCULATIONS

In this section, the procedure of calculating the probability of indirect DEGB is illustrated using the Palo Verde reference plant as an example.

2.4.1 Support Arrangement and CE Seismic Margins

The Palo Verde RCL System consists of two loops which include the reactor vessel, two steam generators, four reactor coolant pumps and the pressurizer. The reactor vessel is supported on four nozzles on the cold legs. At each nozzle, the support system of the reactor vessel consists of a nozzle pad, and a column extending down to base mat. The steam generator has a skirt support with a sliding base; its upper support consists of a key and a snubber assembly. The reactor coolant pump is supported at the top by two horizontal snubbers and two horizontal struts. At the pump skirt level, the support system consists of two horizontal struts and four vertical columns (see Figures 1-1 through 1-6).

It was assumed that the seismic failure of any one of the supports of the reactor vessel, steam generator or reactor coolant pump would unconditionally result in DEGB of RCL piping. It was also assumed that the failure events of similar equipment supports in the RCL system are perfectly correlated. This is a realistic assumption because all the supports are essentially identical (e.g., the skirt supports on both steam generators are identical). For each support, all failure modes were identified and the mode (element) with the lowest seismic margin was considered in the fragility development.

A review of the support seismic margins calculated by CE for Palo Verde (See Appendix A) using Equation 2-4 showed the following critical items:

	Actual Seismic Margin
Reactor vessel columns	17.2
Steam generator snubber assembly	7.3
Reactor coolant pump snubber assembly	10.1

2.4.2 Capacity Factors

In the following, the procedure for evaluating the median and the variability estimates (β_R and β_U) for capacity of the above equipment support elements is described.

2.4.2.1 Reactor Vessel Support Column

The reactor vessel support column is made of ASTM SA-508 Class 2 material. The specified yield strength of this material at 100°F is 50 ksi. The faulted allowable stress in buckling, F_a was specified as 30.3 ksi using the formula given in ASME Boiler and Pressure Vessel Code,

$$F_a = \frac{2}{3}\left[1 - \frac{(k\ell/r)^2}{2C_c^2}\right] S_y \tag{2-18}$$

The slenderness parameter of the column $\lambda = (\sqrt{2}\, k\ell/r)/C_c$ was calculated as 0.60.

Hall (1981) has developed the following expression for the mean ultimate buckling strength of columns:

$$\bar{F}_{ult} = \bar{S}_y\,(1.3 - 0.57\lambda) \text{ for } \lambda \leq 1.53 \tag{2-19}$$

Therefore, mean ultimate strength of the column was estimated as

$$\bar{F}_{ult} = (1.25)(50)\,[1.3 - 0.57 \times 0.60] = 59.9 \text{ ksi}$$

where the mean yield strength was taken to be 1.25 times the specified yield strength (Rodabaugh and Desai, 1981). The uncertainty in the ultimate strength was obtained from:

$$\beta_{ult} = (\beta^2_{matl} + \beta^2_{fabr} + \beta^2_{\substack{equation\\error}})^{1/2} \tag{2-20}$$

where:

β_{matl} = logarithmic standard deviation of (the uncertainty in) material yield strength = 0.09 (Rodabaugh and Desai, 1981).

β_{fabr} = logarithmic standard deviation of the ultimate column strength due to uncertainties in fabrication = 0.05 estimated (Ravindra and Galambos, 1978).

$\beta_{\substack{equation\\error}}$ = logarithmic standard deviation reflecting the uncertainty in the strength predicted by Equation 2-19. This is the logarithmic standard deviation (approximately the coefficient of variation) of the ratio of the measured buckling stress to the predicted buckling stress. Hall (1981) gives this value as 0.0. This was considered too low. Consequently, a value of 0.15 was used.

Therefore, β_{ult} was calculated as 0.18. The median ultimate strength was calculated from:

$$\check{F}_{ult} = \bar{F}_{ult} \quad \exp\ (-1/2\ \beta^2_{ult}) \tag{2-21}$$

Hence, $\check{F}_{ult} = 58.9$ ksi.

The median strength factor $\check{F}_S$ was calculated from:

$$\check{F}_S = \frac{\check{F}_{ult}}{F_a} \text{ (seismic margin) } = (\frac{58.9}{30.3})\ (17.2) = 33.4$$

Since the failure mode is buckling, no credit for inelastic energy absorption was taken.

$$\check{F}_C = \check{F}_S = 33.4$$

$$\beta_{C,U} = \beta_{ult} = 0.18$$

2.4.2.2 <u>Steam Generator Snubber Assembly</u>

ASME Boiler & Pressure Vessel Code Division III Section NF gives the allowable load on the snubber under faulted conditions as 0.7 F_u where F_u is the specified ultimate capacity of snubber. The median ultimate capacity of snubber was estimated as 1.1 F_u.

The median strength factor was evaluated as

$$\check{F}_S = \left(\frac{1.1F_u}{0.7F_u}\right) \text{ (seismic margin) } = 11.4$$

Since snubber failure is a localized failure, the median inelastic energy absorption factor was taken to be 1.0. Therefore, $\check{F}_C = 11.4$. The uncertainty in the snubber capacity was estimated as

$$\beta_{C,U} = \left(\beta^2_{matl} + \beta^2_{\substack{failure\\point}}\right)^{½} \tag{2-22}$$

where

β_{matl} = logarithmic standard deviation of material strength = 0.06 estimated by considering the specified ultimate capacity to be a 5 percent non-exceedence value

$\beta_{\text{failure point}}$ = logarithmic standard deviation representing the uncertainty in the actual stress at which the snubber fails. Note that exact evaluation of this β would require testing of several of these snubbers. In the absence of such data, β = 0.15 is used based on engineering judgment and test results of systems of similar complexity. In any case, a minor error in this β value will not significantly affect the calculated probabilities as discussed in Section 2.1.1.

Therefore, $\beta_{C,U} = 0.16$

2.4.2.3 Reactor Coolant Pump Snubber Assembly

The CE calculated margin for this support element is 10.1. The median and uncertainty estimates were obtained using the above procedure (Section 2.4.2.2) as:

$$\check{F}_C = 15.9$$

$$\beta_{C,U} = 0.16$$

2.4.3 Structure Response Factor, F_{RS}

As noted before, the structure response factor F_{RS} is modeled as a product of several factors

$$F_{RS} = F_{SA}\, F_{\delta}\, F_M\, F_{SD}\, F_{SS} \tag{2-8}$$

2.4.3.1 Spectral Shape Factor F_{SA}

Palo Verde is founded on a multi-layer system of sand and clay. A review of the frequencies and composite modal damping values (Appendix A Pages 15-16) indicated that modes 1 and 2 with frequencies around 1.7 Hz for a composite modal damping value of 11.5% are dominant contributors to the structural response of internal structures. Therefore, the spectral shape factor was derived by comparing the design response spectrum with WASH 1255 median alluvium site spectrum at this frequency for 10% damping. The median spectrum for 11.5% damping was not available in WASH 1255. From the Palo Verde design response spectrum, spectral amplification factor at 1.7 Hz was found to be 0.41/0.25 = 1.64. From the WASH 1255 median alluvium spectrum, the corresponding spectral amplification factor was observed to be 1.50.

A time history was selected in the design analysis to match the Palo Verde design spectrum. Appendix A Page 8 shows the spectrum obtained from the time history and design spectrum at 10% damping. The ratio of the spectral accelerations from these two spectra varied from 1.0 (for a frequency of 1.7 Hz) to 1.27 (for a frequency of 7.8 hz).

Since the first mode contributed most of the response of the internal structure, the safety factor in the use of a synthetic time history was judged to be 1.1. Therefore, the median spectral shape factor F_{SA} was calculated as:

$$\check{F}_{SA} = \left(\frac{\begin{matrix}\text{Spectral Amplification}\\ \text{Factor From Design Spectrum}\end{matrix}}{\begin{matrix}\text{Spectral Amplification}\\ \text{Factor From Median}\\ \text{WASH 1255 Spectrum}\end{matrix}}\right)\left(\begin{matrix}\text{Safety Factor}\\ \text{in Time History}\\ \text{Used}\end{matrix}\right)$$

$$\check{F}_{SA} = \left(\frac{1.64}{1.50}\right)(1.1) = 1.19$$

Since the seismic response of internal structures was seen to be dominated by soil modes with high damping, the variability in the ground response spectrum was judged to induce minimal variability in the response. The value of $\beta_{SA,R}$ was estimated to be 0.10. In order to estimate the uncertainty associated with the applicability of the WASH 1255 median spectrum to the site, a comparison of site-specific spectra and the WASH 1255 spectrum for many sites, for which the WASH 1255 spectrum is considered applicable, would have to be made. In lieu of this, $\beta_{SA,U}$ = 0.10 was used based on engineering judgment. Any error in this estimate will have only a very minor effect on the calculated probabilities as discussed in Section 2.1.1.

2.4.3.2 Damping Factor, F_{δ}

The design damping for internal structures was 7%; the median damping of internal structures for seismic excitation at the failure level of equipment supports was estimated to be 7%. As mentioned before, the structural response is dominated by soil modes with high composite modal damping ratios. Therefore, the effect of any variation in the structural damping was judged to be a minimum. Therefore, $\check{F}_{\delta}$ = 1.0, $\beta_{\delta,R}$ = = 0.10 and $\beta_{\delta,U}$ = 0.10.

2.4.3.3 Modeling Factor, F_M

The modeling factor accounts for the variability in stiffnesses, masses, detail in modeling etc. The median modeling factor $\check{F}_M$ was judged to be 1.0 since the structural responses were obtained using a state-of-the-art coupled time history analysis. The variability in response due to modeling assumptions was estimated as $\beta_{M,U}$ = 0.15 (Hadjian, et al, 1977, Kennedy, et al, 1980).

2.4.3.4 Soil-Structure Interaction Factor, $F_{SSI} = F_{SD}F_{SS}$

The soil-structure interaction analysis was done during the design stage using state-of-the-art methods and according to current NRC criteria. The time history of the design earthquake was assumed to be at the base of the foundation for containment. The soil properties were varied over specified ranges and the structural responses enveloped. Based on previous studies (Johnson, 1983), the median embedment factor $\check{F}_{SD}$ was estimated as 1.25. The conservatism introduced by the soil property variation procedure and in the current SSI methods was represented by the median soil-structure factor of $\check{F}_{SS}$ = 1.15. Therefore, the median soil structure interaction factor was obtained as $\check{F}_{SSI}$ = (1.25)(1.15) = 1.44. The variability estimates for SSI factor were assessed as $\beta_{SSI,R}$= 0.20 and $\beta_{SSI,U}$ = 0.25.

The median structure response factor, $\check{F}_{RS}$, was calculated as 1.71 with $\beta_{RS,R} = 0.24$ and $\beta_{RS,U} = 0.32$ (see Table 2-3).

2.4.4 Equipment Response Factor, F_{RE}

The equipment response factor, F_{RE}, was modeled as

$$F_{RE} = F_{SA} F_{\delta} F_{M} F_{MC} F_{EC} \tag{2-11}$$

2.4.4.1 Spectral Shape Factor F_{SA}

The spectral shape factor represents the margins inherent in the selection of response spectra for the equipment response analysis. Since a coupled time history analysis was performed in the design, the value of $\check{F}_{SA}$ was taken to be 1.0 and the logarithmic standard deviation $\beta_{SA} = 0$.

2.4.4.2 Damping Factor F_{δ}

The design damping for the NSSS was 2%. The median damping at failure of equipment supports was estimated to be 7%. No floor response spectra were available. In soil sites, the floor spectra drop off very rapidly beyond 5 Hz. Since Palo Verde is on a medium stiff soil foundation, the spectral accelerations at the equipment frequencies (10 Hz) are expected to be slightly higher than the zero period acceleration. The response of equipment supports was judged to be not influenced by the difference in damping. Therefore, the damping was estimated as:

$$\check{F}_{\delta} = 1.10$$

$$\beta_{\delta,R} = 0.10$$

$$\beta_{\delta,U} = 0.10$$

2.4.4.3 Modeling Factor, F_M

The design analysis was done using a coupled time history three-dimensional analysis. It is assumed herein that the analyst had done a best job of modeling the NSSS equipment (i.e., modeling the supports, boundary conditions and representing material behavior), thus, $\check{F}_M = 1.0$. To evaluate $\beta_{M,U}$ for the NSSS, models would have to be developed and the analysis performed by different groups of analysts. This would give an indication of the response variability due to modeling assumptions. Obviously, this would be extremely expensive and is unwarranted in light of the relative insensitivity of the calculated probabilities to small variations in the β value (as discussed in Section 2.1.1). Based on the modeling uncertainty used for the structure (Section 2.4.3.3) and the fact that the NSSS is a complex 3-D system, $\beta_{M,U} = 0.20$ is judged appropriate.

2.4.4.4 Mode Combination Factor, F_{MC}

Since a time history analysis was performed at the design stage, the mode combination factor was taken to be 1.0 with $\beta_{MC} = 0$.

2.4.4.5 Earthquake Component Combination Factor, F_{EC}

Since a three-dimensional time history analysis was performed at the design stage, the response in equipment support so obtained was judged to be median-centered with no uncertainty, i.e., $\check{F}_{EC} = 1.0$ and $\beta_{EC,U} = 0$. Exact determination of the variability due to random phasing of earthquake time histories in the three directions would require numerous extensive three-dimensional time history analyses to be performed. Based on judgment, $\beta_{EC,R} = 0.10$ is felt to be a reasonable representation. As discussed in Section 2.1.1, minor variations in this value will not affect the final probabilities calculated.

The median equipment response factor, $\check{F}_{RE}$, was calculated as 1.10 with $\beta_{RE,R} = 0.14$ and $\beta_{RE,U} = 0.22$ (see Table 2-3).

Table 2-3 gives the overall response factor, F_R, as $\check{F}_R = 1.88$, $\beta_{R,R} = 0.28$ and $\beta_{R,U} = 0.39$. If a composite variability $\beta_{R,C} = (\beta^2_{R,R} + \beta^2_{R,U})^{1/2}$ is defined, the overall response factor derived herein indicates that the probability of the actual SSE response exceeding the calculated SSE response is 10%. For a modern plant using state-of-the-art method of analysis, this value of exceedence probability is expected.

2.4.5 Ground Acceleration Capacity, A_C

The median ground acceleration capacity of each equipment support was calculated using the formula:

$$\check{A}_C = A_{SSE} \quad \check{F}_C \, \check{F}_{RS} \, \check{F}_{RE} \tag{2-12}$$

and the variability estimates as

$$\beta_{A,R} = (\beta^2_{C,R} + \beta^2_{RS,R} + \beta^2_{RE,R})^{1/2} \tag{2-14}$$

$$\beta_{A,U} = (\beta^2_{C,U} + \beta^2_{RS,U} + \beta^2_{RE,U})^{1/2} \tag{2-15}$$

Table 2-4 presents the fragility parameters for Palo Verde RCL equipment supports.

2.4.6 Probability of Indirect DEGB

As stated before, it was assumed that the failure of any one of the equipment supports (Table 2-4) would result in a DEGB of RCL. By convolving the Palo Verde seismic hazard curves (Figure 2-10) with the fragility curves of the equipment supports generated using Table 2-4, the probability of indirect DEGB was calculated. The median probability of indirect DEGB was calculated as 3.8×10^{-16} per reactor-year and the 10 to 90% subjective probability (interval) on P_{DEGB} was obtained as 4×10^{-19} per reactor-year to 1×10^{-13} per reactor-year. These low probabilities are the result of rather high seismic margins in the equipment supports and the low seismic hazard predicted for the site. The sensitivity of the results to the seismic hazard prediction was examined by convolving the above fragility curves with the generic seismic hazard curves (Fig. 2-7) developed for the Eastern and midwestern United States. The median probability of DEGB was obtained as 5.4×10^{-10} per reactor-year and the 10 to 90% subjective probability interval on P_{DEGB} was found to be 2.4×10^{-12} per reactor-year to 2.6×10^{-8} per reactor-year. For the purposes of comparison, the median probability of indirect DEGB for the lowest capacity Westinghouse reactor estimated in Ravindra, et al (1983) was 3.3×10^{-6} per reactor-year with the 10 to 90% subjective probability interval as 2.3×10^{-7} to 2.3×10^{-5} per reactor-year.

TABLE 2-1

CE SEISMIC MARGINS FOR PALO VERDE

PLANT Composite Plant - Group C Primary Component Reactor Vessel

Item	Material	Dsgn/Fabr Code	Actual Seismic Margin	Allowable Stress	Failure Mode	Consequence of This Failure
Columns:	SA-508 class 2	ASME III NF	3.1	$F_a = \frac{2}{3}\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right]F_Y = 30.32^{KSI}$ $F_{ba} = 1.167 \times \frac{S_u}{S_Y}(.6S_Y) = 5045^{KSI}$ $F_{bc} = 1.167 \times \frac{S_u}{S_Y}(.75S_Y) = 63.1^{KSI}$	Buckling	Would result in equipment support failure.
Lower Key:	SA-533 GR. B Class 1	ASME III NB	3.6 "	$S_{all.} = 1.5(1.2S_Y) = 76.5^{KSI}$	Bearing	Would not result in equipment support failure.
Load Limiter Assembly:	SA-240 Type 321	ASME III NF	1.25	Allowable Load = 500^K (from allowable load graph.)	Crushing	" "
Nozzle pad:	SA-533 GR. B class 1	ASME III NB	4.25	$(S.I.)_{all.} = 1.5S_m = 76.4^{KSI}$	(P_m+P_b) stress	Would result in equipment support failure.

TABLE 2-1

CE SEISMIC MARGINS FOR PALO VERDE
(CONTINUED)

PLANT Composite Plant - Group C Primary Component Steam Generator

Item	Material	Dsgn/Fabr Code	Actual Seismic Margin	Allowable Stress	Failure Mode	Consequence of This Failure
Snubber Lever	SA-542 Class 1	ASME III NF	3.73	$S_{all.} = 1.167(\frac{S_u}{S_Y})(.4S_Y) = 45.9^{KSI}$	Shear	Would result in equipment support failure
Skirt Flange:	SA-533 GR. B Class 1	ASME III NB	4.6	$(S.I.)_{all.} = 1.2S_Y = 53^{KSI}$	P_m stress	" "
Weld-skirt to S.G.:	Linde-124 (20x150)	ASME III NB	4.4	$(S.I.)_{all.} = 1.5(1.2S_y) = 78^{KSI}$	$(P_m + P_b)$ stress	" "

TABLE 2-1

CE SEISMIC MARGINS FOR PALO VERDE
(CONTINUED)

PLANT Composite plant - Group C Primary Component Reactor Coolant Pump

Item	Material	Dsgn/Fabr Code	Actual Seismic Margin	Allowable Stress	Failure Mode	Consequence of This Failure
Snubber Bracket:	ASTM A-148-73 GR.120-95 (AISI 4130/4140 /4330/4340)	ASME III NF	2.3	$F_b = \frac{.75u}{F_t}(.75S_y)$ $= \frac{.75u}{.6S_y}(.75S_y) = 105^{KSI}$	Bending	Would result in equipment support failure
Column (end): (upper Horiz. Support)	SA-320 GR.L43	ASME III NF	5.0	$F_v = .42S_u = 49.5^{KSI}$	Shear	" "
Pin: (upper Horiz. Support)	SA-540-B23	ASME III NF	5.6	$F_b = 1.167(\frac{S_u}{S_y})(.75S_y)$ $= 132.6^{KSI}$	Bending	" "

TABLE 2-2

RECURRENCE CURVES FOR

DIFFERENT SEISMIC SOURCES -PVNGS

Seismic Source	Interval (Δm = .5) Recurrence Curve No./year/km^2	Best Estimate Maximum Magnitude
San Andreas Fault	log N(m) = - .496 - .825m	7.5
San Jacinto-Imperial Fault Zone	= .077 - .825m	7.25
Whittier-Elsinore Fault Zone	= .284 -1.02m	7.25
Cerro Prieto Fault Zone	= .077 - .825m	7.25
Sand Hills-Algodones Zone	= - .923 - .825m	6.5
Sierra Juarez Zone	= - .796 - .75m	7.25
Gulf of California Zone	= - .516 - .825m	7.25
Zone B	= -1.048 - .9m	5.0
Zone C	= -1.366 - .9m	6.5
Pitaycachi Fault	= - .204 - .9m	7.5
Verde Fault	= - .204 - .9m	7.5
Zone D	= -2.198 - .9m	4.5

TABLE 2-3

RESPONSE FACTORS FOR PALO VERDE EQUIPMENT SUPPORTS

Factor	Median Factor Safety	β_R	β_U
Structure Response			
o Spectral Shape	1.19	0.10	0.10
o Damping	1.00	0.10	0.10
o Modeling	1.00	0	0.15
o Soil Structure Interaction	1.44	0.20	0.25
F_{RS}	1.71	0.24	0.32
Equipment Response			
o Spectral Shape	1.00	0	0
o Damping	1.10	0.10	0.10
o Modeling	1.00	0	0.20
o Mode Combination	1.00	0	0
o Earthquake Component Combination	1.00	0.10	0
F_{RE}	1.10	0.14	0.22
Overall Response Factor	1.88	0.28	0.39

TABLE 2-4

PALO VERDE RCL EQUIPMENT SUPPORT FRAGILITY PARAMETERS

	Equipment Support	Failure Mode	Median Factor of Safety	Ground Acceleration Capacity		
				$\check{A}$ (g)	β_R	β_U
1.	Reactor Vessel	Column Buckling	62.8	15.7	0.28	0.43
2.	Steam Generator	Snubber Assembly Failure	21.4	5.3	0.28	0.42
3.	Reactor Coolant Pump	Snubber Assembly Failure	29.9	7.4	0.28	0.42

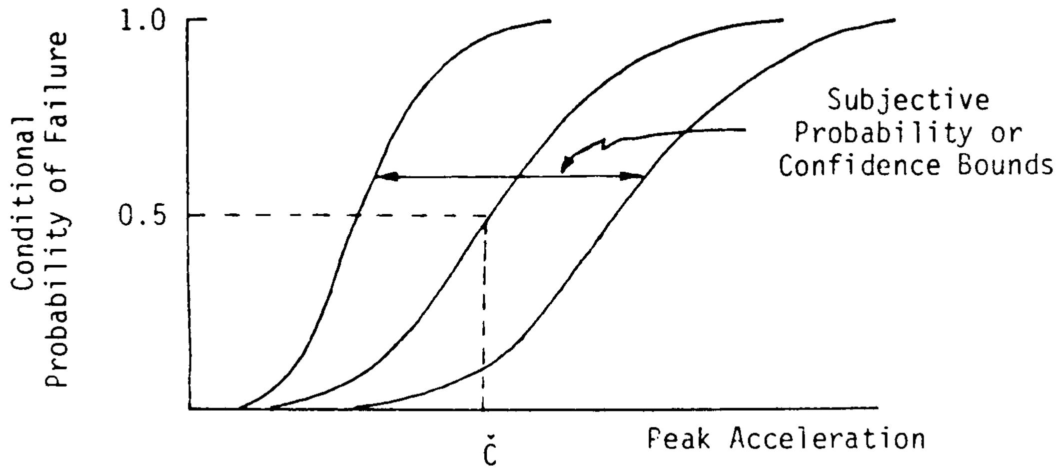

FIGURE 2-1. FRAGILITY OF STRUCTURE OR EQUIPMENT

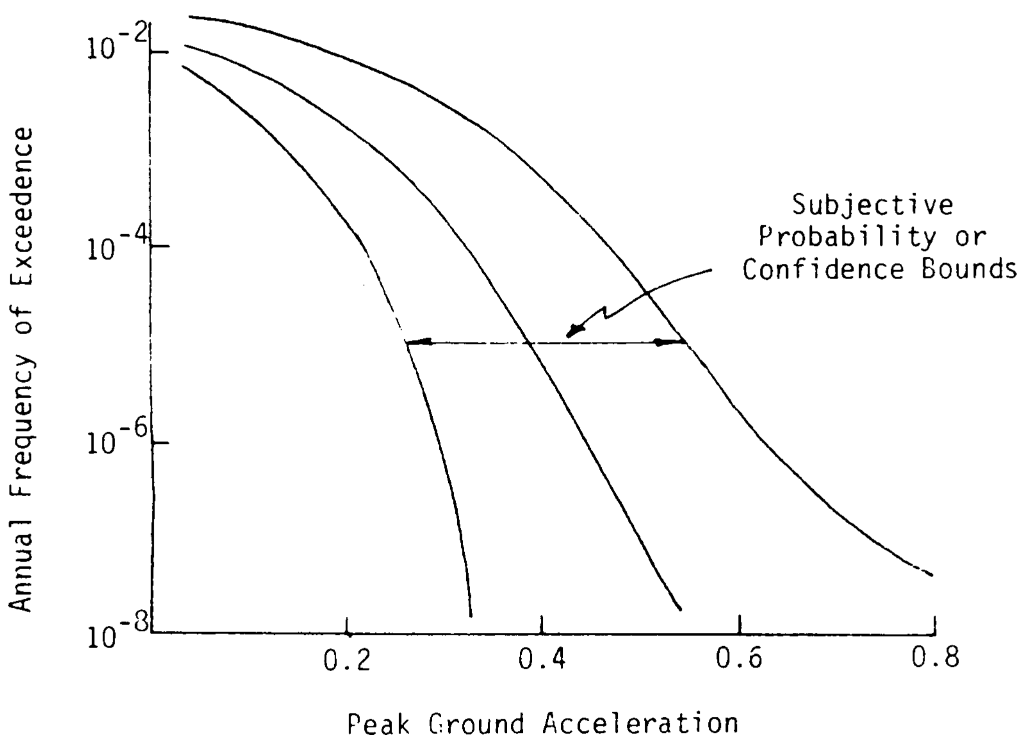

FIGURE 2-2. SEISMIC HAZARD CURVES

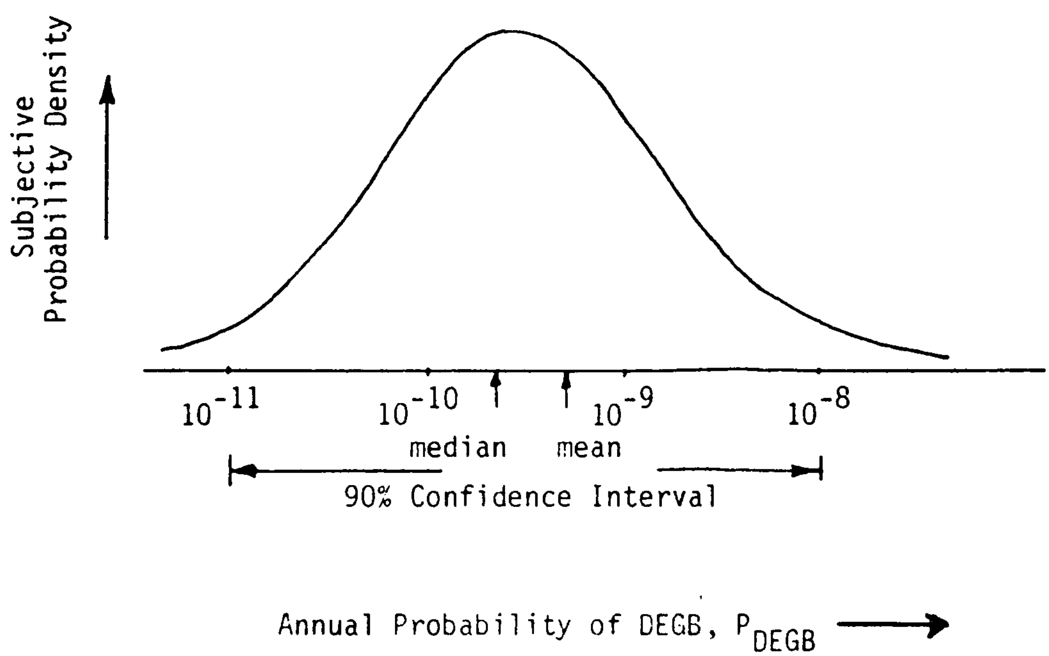

FIGURE 2-3. DISTRIBUTION OF THE PROBABILITY OF INDIRECTLY-INDUCED DEGB

FIGURE 2-4 REGION OF APPLICABILITY OF GENERIC SEISMIC HAZARD CURVES (RIGHT OF THE DASHED LINES)

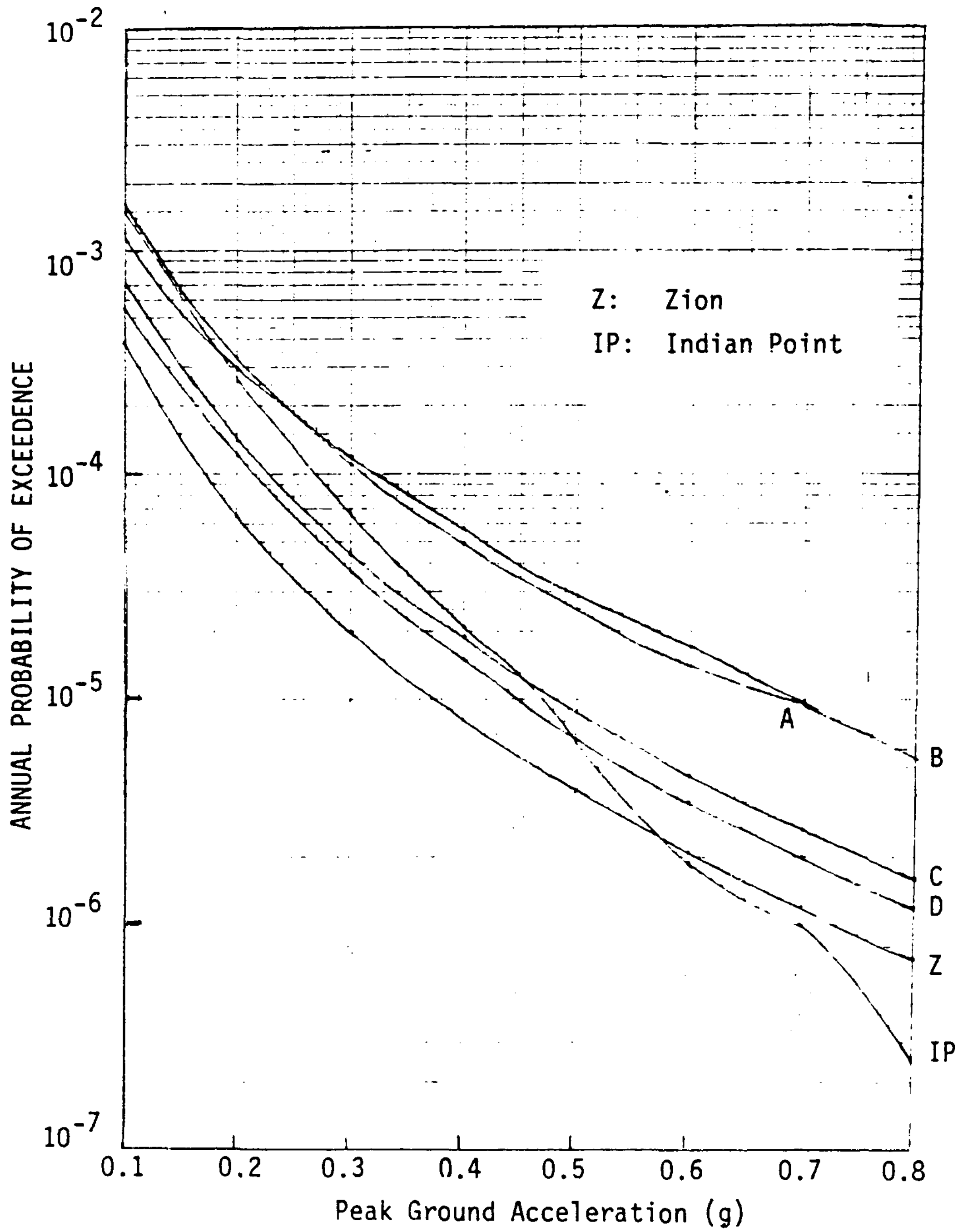

FIGURE 2-5 MEAN SEISMIC HAZARD CURVES

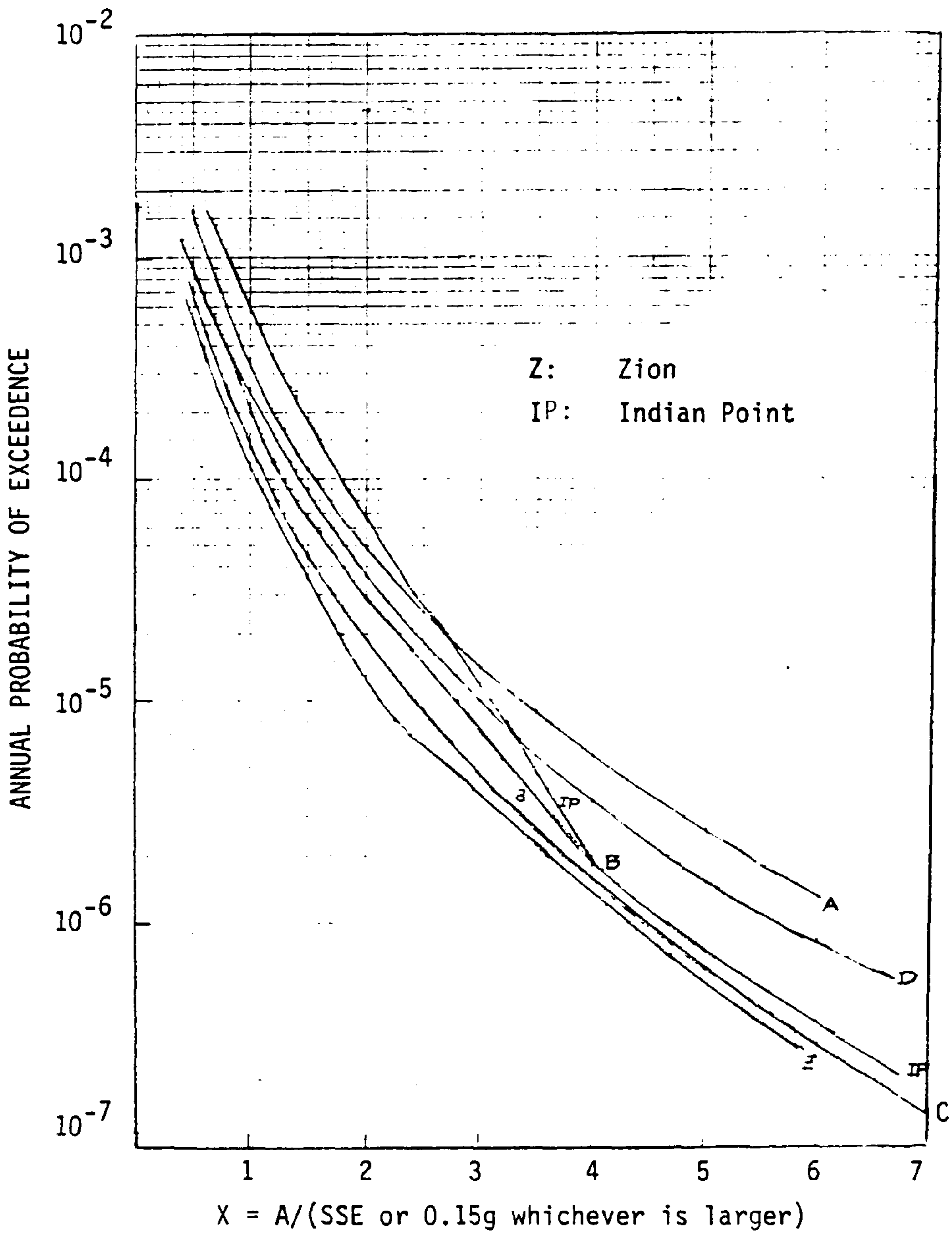

FIGURE 2-6 NORMALIZED MEAN HAZARD CURVES

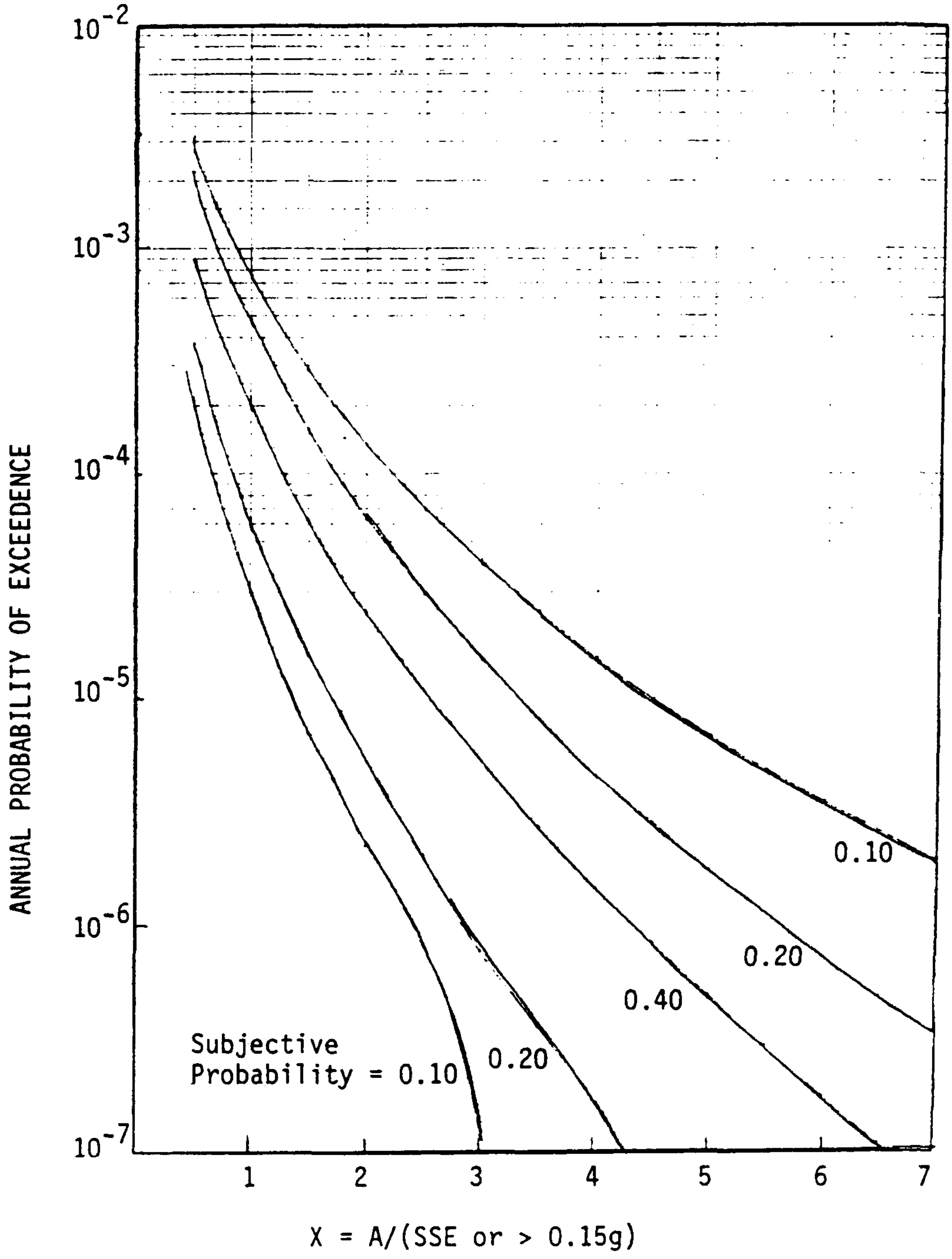

FIGURE 2-7 GENERIC SEISMIC HAZARD CURVES

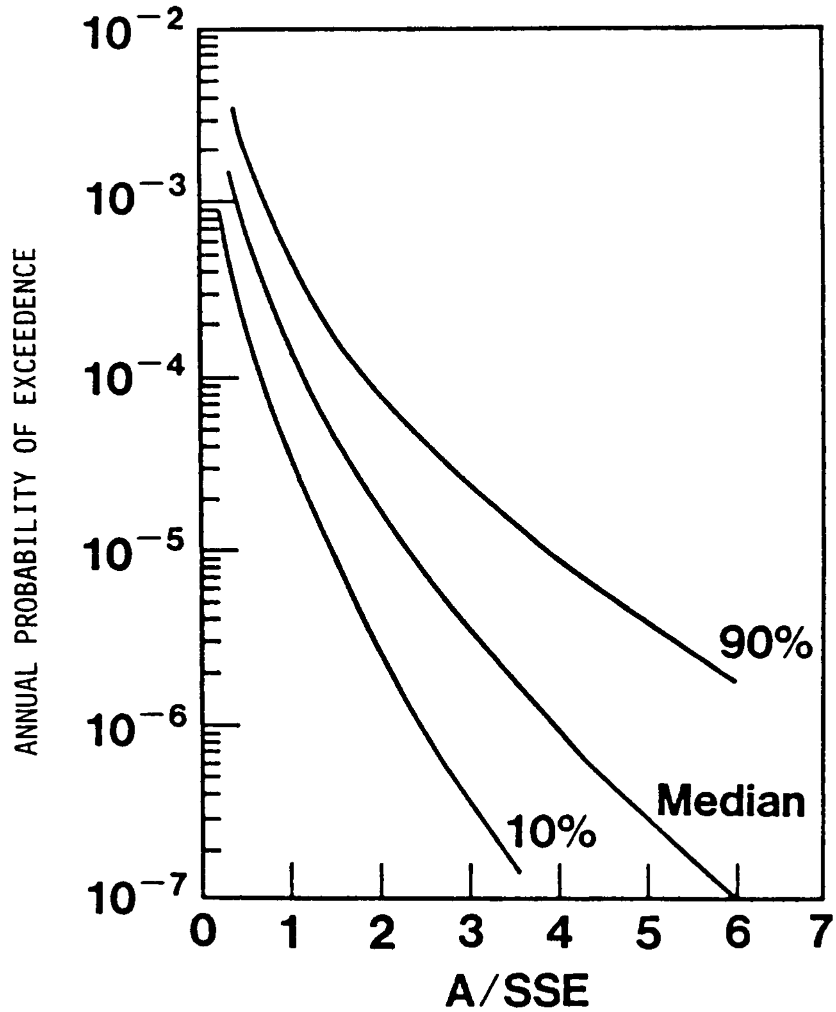

FIGURE 2-8 GENERIC SEISMIC HAZARD CURVES - MEDIAN AND 10% AND 90% CURVES

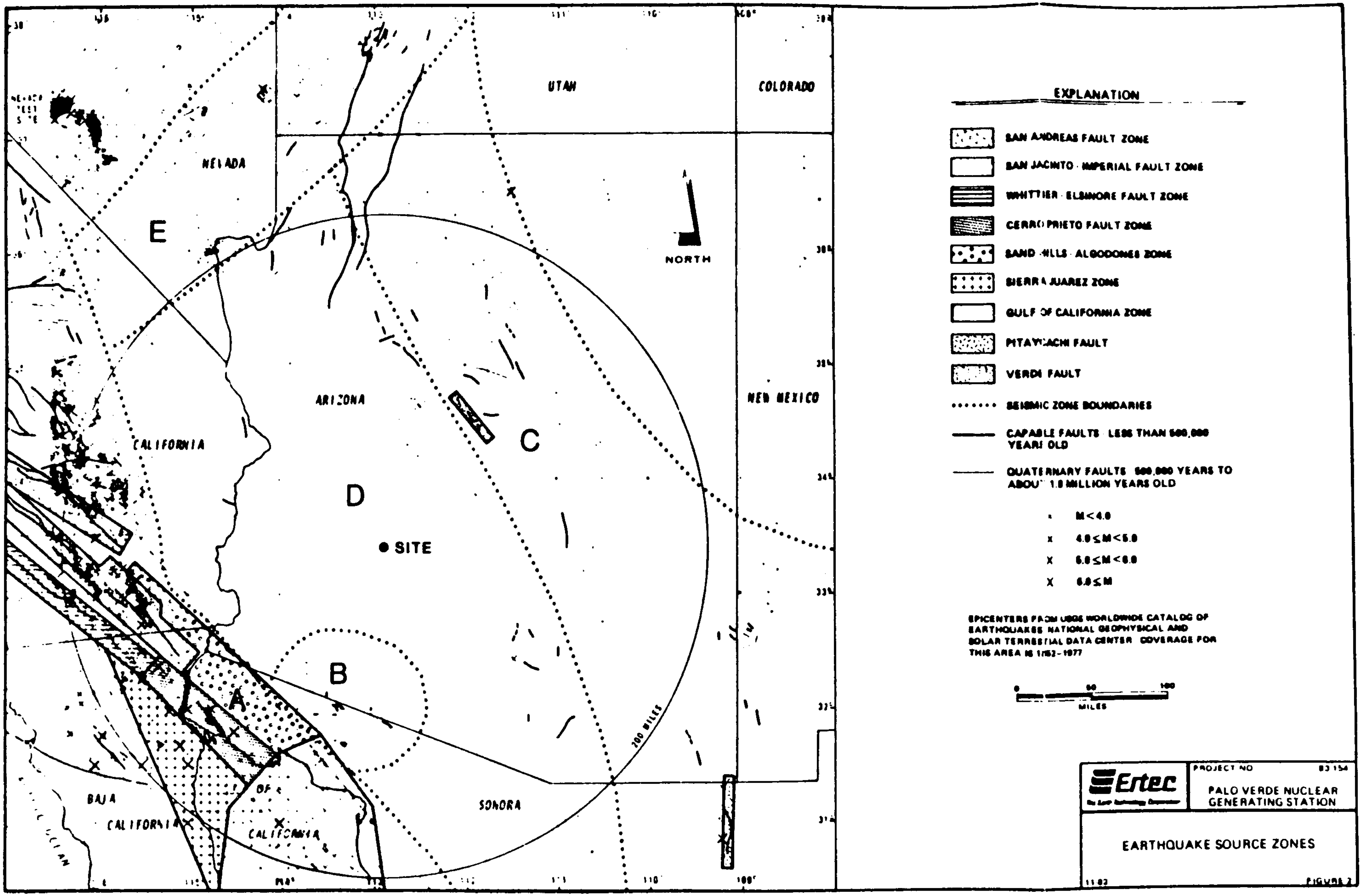

FIGURE 2-9 PVNGS - SEISMIC SOURCE ZONES

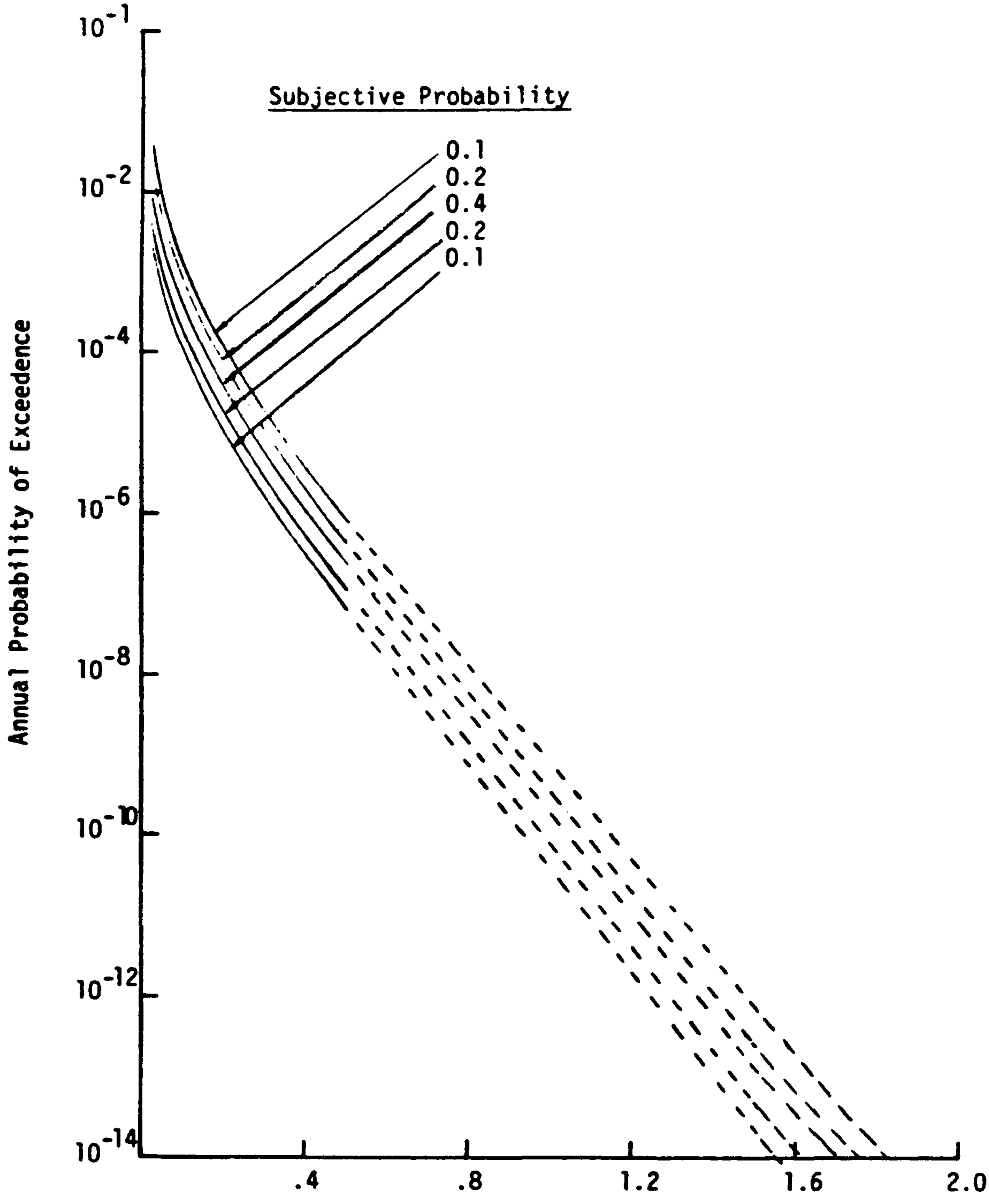

FIGURE 2-10 PVNGS SEISMIC HAZARD CURVES - DEVELOPED BASED ON ERTEC SEISMIC HAZARD STUDY

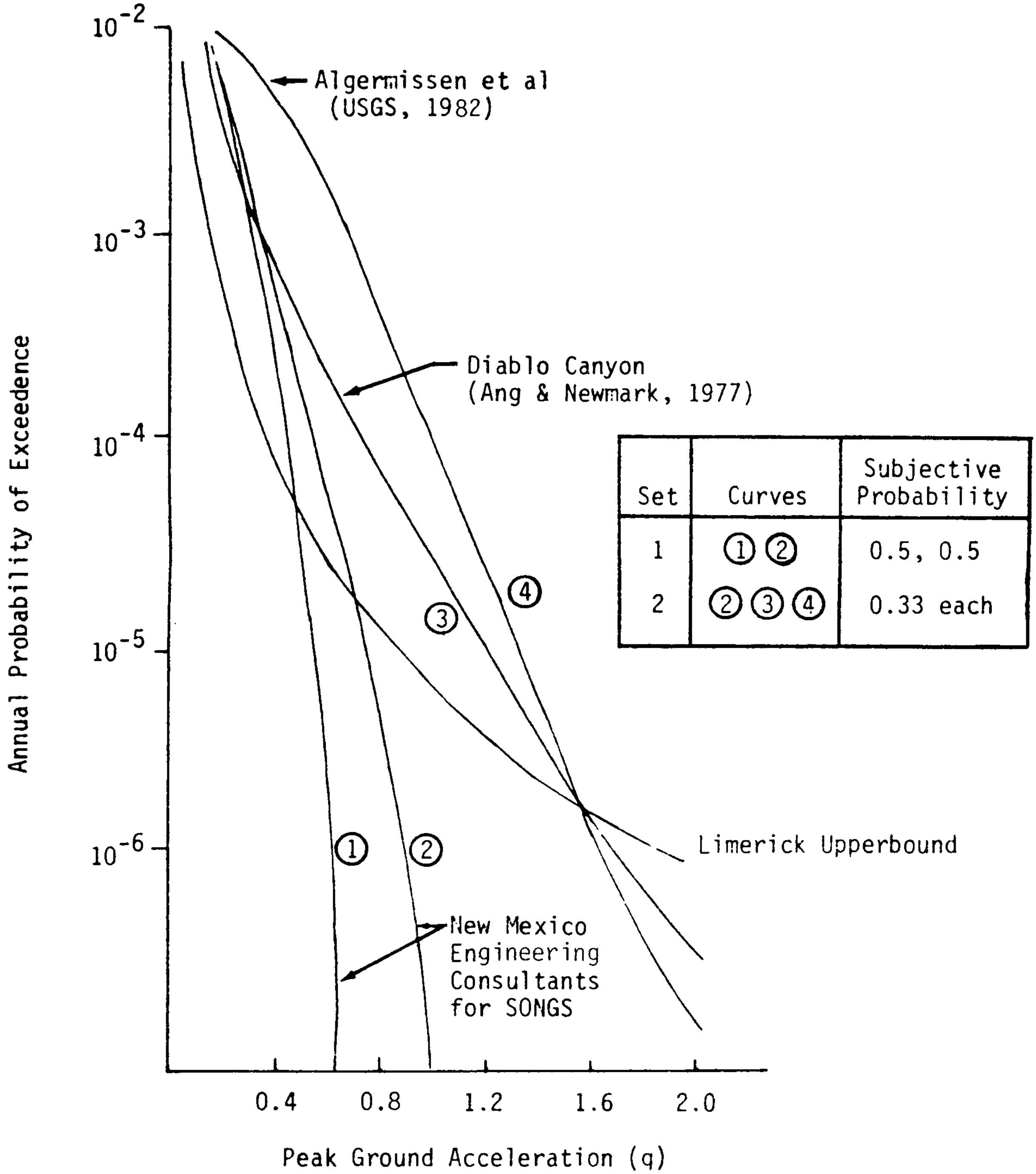

FIGURE 2-11 SONGS SEISMIC HAZARD CURVES

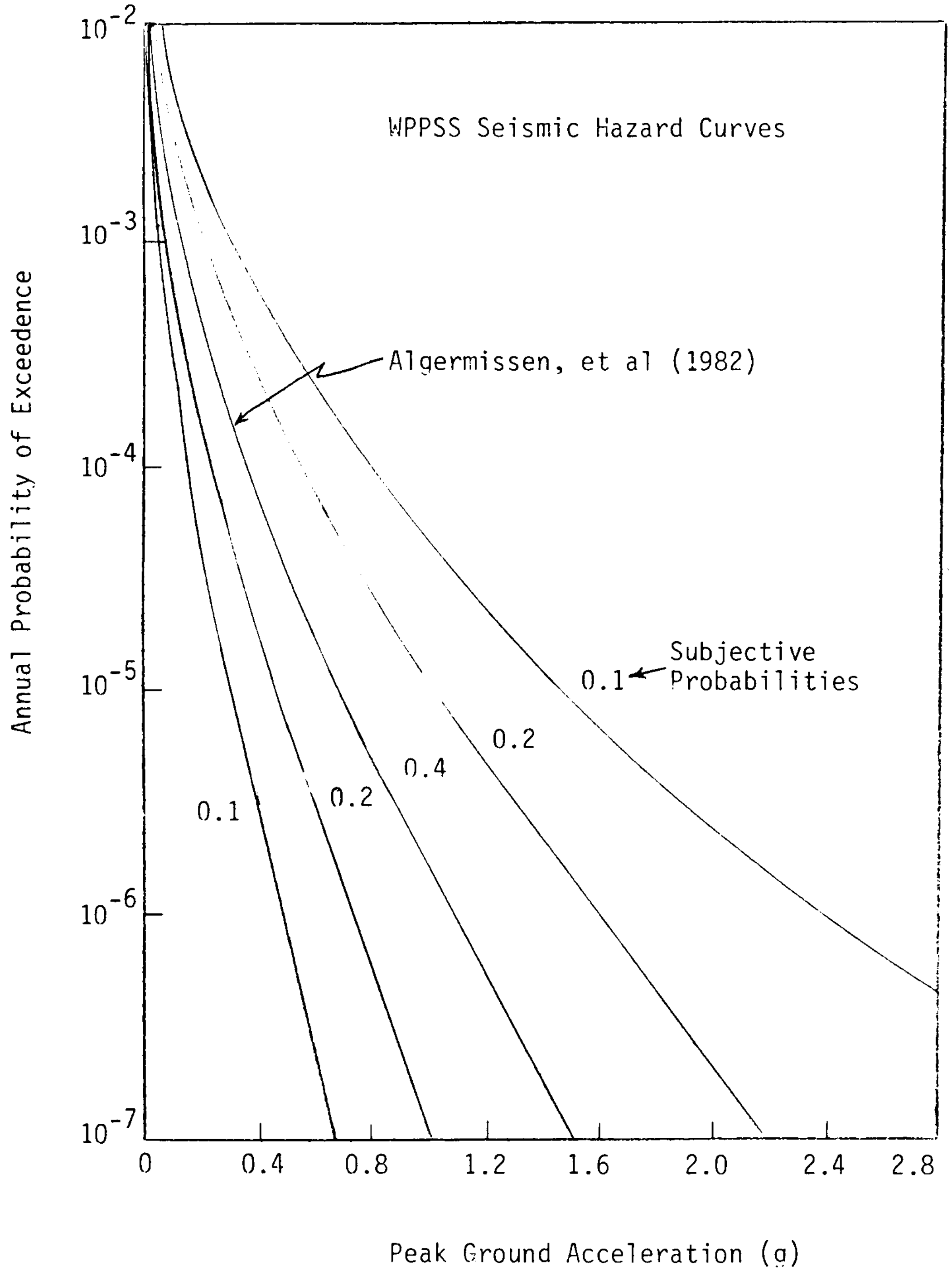

FIGURE 2-12. WPPSS SEISMIC HAZARD CURVES

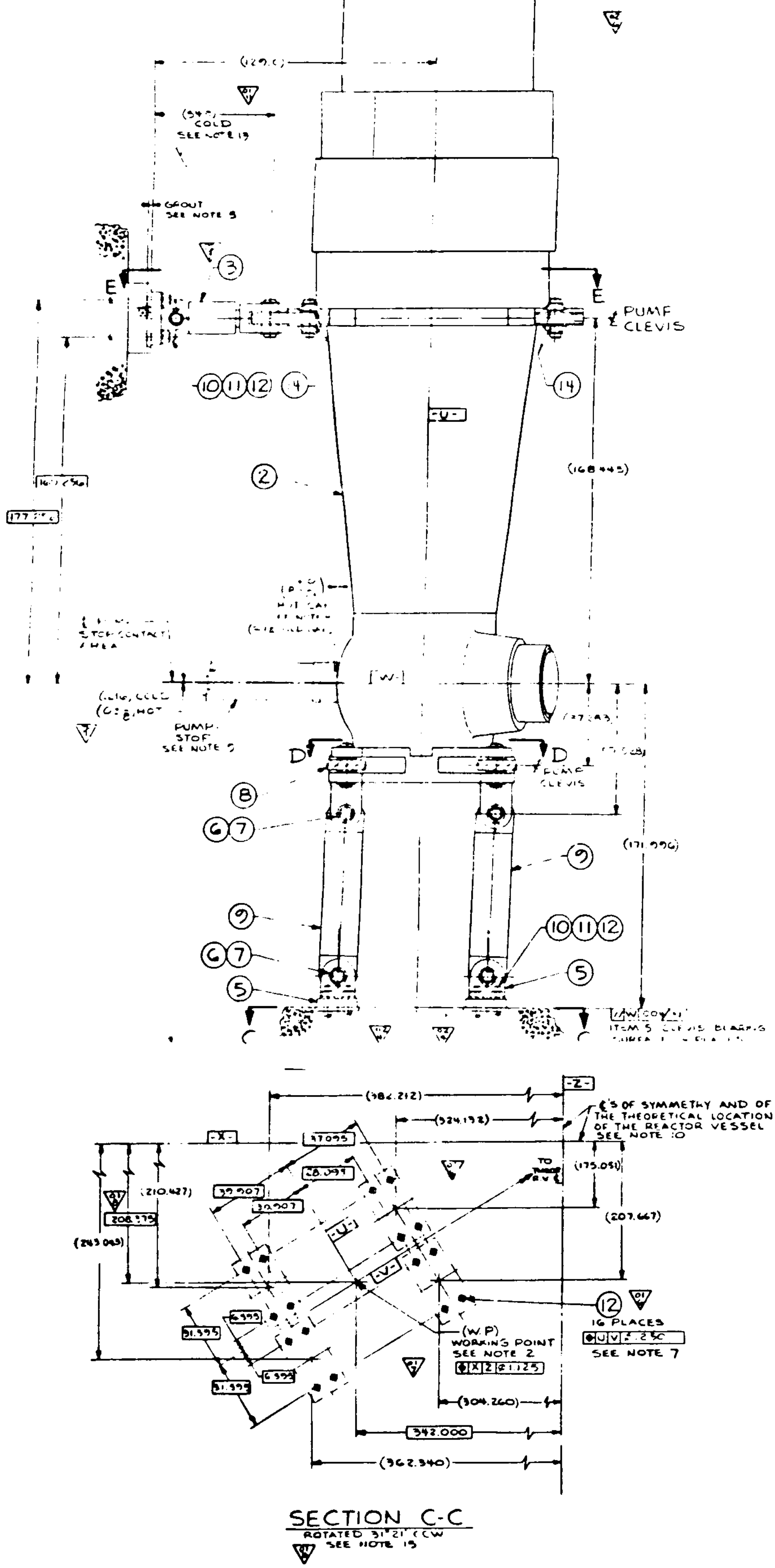

FIGURE 1-6. REACTOR COOLANT PUMP SUPPORTS

CHAPTER 3

RESULTS AND CONCLUSIONS

3.1 RESULTS

3.1.1 Probability of Indirect DEGB

For each plant in Groups A and C, the fragilities of equipment supports (i.e., RPV, steam generator and reactor coolant pump) were estimated using the procedure described in Sections 2.1 and 2.4. These fragilities were convolved with the appropriate set of seismic hazard curves (i.e., generic or site-specific) according to Equation 2-1 to obtain the probability of indirect DEGB. Tables 3-1 and 3-2 show the values of P_{DEGB} for plants in Groups A and C, respectively.

For earlier vintage Group A plants, the median value of the probability of indirect DEGB varies from 6.6×10^{-8} to 6.4×10^{-6} per reactor-year. The 90% confidence (subjective probability) value of P_{DEGB} ranges from 1.2×10^{-6} to 5.2×10^{-5} per reactor-year. Palisades is the plant with the lowest median factor of safety of 4.1. This plant was designed using static analysis and the Housner spectrum which is considered to be less conservative than R.G. 1.60 response spectrum. However, the generic seismic hazard curves used in estimating P_{DEGB} may be too conservative for the Palisades site.

For the more modern Group C Plants, the median value of P_{DEGB} varies from 3.8×10^{-16} to 1.3×10^{-8} per reactor-year. The 90% confidence (subjective probability) value of P_{DEGB} ranges from 3.2×10^{-14} to 3.0×10^{-7} per reactor-year. Different sets of seismic hazard curves were used to derive the above P_{DEGB} values. The site-specific seismic hazard studies for Palo Verde and San Onofre Units 2 and 3 yielded extremely low values of indirect DEGB probabilities. However, the use of alternative sets of hazard curves (i.e., site-specific or generic hazard) also resulted in indirect DEGB probabilities lower than those obtained for Group A plants.

For the Group B plant (Ft. Calhoun), the median value of P_{DEGB} was estimated as 1.6×10^{-6} per reactor-year using the generic seismic hazard curves. The 90% confidence value of P_{DEGB} was obtained as 1.4×10^{-5} per reactor-year.

3.1.2 Response Factors

An intermediate result of this study is the response factor of safety, F_R, for the equipment supports in each plant. This result was used to scale the responses of RCL piping for different earthquake ground accelerations in calculating the probability of direct DEGB of RCL piping. The response factor, F_R, was described in terms of the median value, $\check{F}_R$

and the variability measures, $\beta_{R,R}$ and $\beta_{R,U}$. These parameters were estimated by assessing the conservatism or non-conservatism present in different stages of the structure/equipment response analysis as described in Sections 2.1 and 2.4. Tables 3-3 through 3-11 show the response factors for the plants studied under Groups A, B and C. The basis for estimating the factor of safety for each variable (e.g., spectral shape, damping, mode combination, and modeling) is briefly presented.

The response factor for Palisades was evaluated by comparing the spectral acceleration for which the equipment support was designed to the spectral acceleration derived in the SEP analysis (Nelson, et al, 1981) scaled to the site-specific ground response spectrum. The following median response factors were obtained for Palisades RCL equipment supports:

	$\check{F}_R$
Reactor Pressure Vessel	3.20
Steam Generator	1.03
Reactor Coolant Pump	1.16

The variability measures were $\beta_{R,R}$ = 0.35 and $\beta_{R,U}$ = 0.50. The above approach of comparing the design spectral acceleration to the median spectral acceleration was possible because a median-centered analysis was available; it also permitted the response factor evaluation to bypass the steps usually followed (Section 2.1) of estimating the safety factors on each variable. Therefore, a table of response factors similar to Tables 3-3 through 3-11 is not included for Palisades.

3.2 COMPARISON WITH PREVIOUS STUDIES

In a previous study of evaluating the probability of indirect DEGB in RCL piping of Westinghouse reactors east of the Rocky Mountains, the median probability of indirect DEGB was estimated as 3.3×10^{-6} per reactor-year with the 10% to 90% confidence bounds as 2.0×10^{-7} to 2.0×10^{-5} per reactor-year. This was based on the plant with the lowest seismic capacity for the RCL equipment supports among all the W reactors. Generic seismic hazard curves were utilized in this computation. Tables 3-1 and 3-2 show that all CE reactors except Palisades have lower P_{DEGB} values than the above W plant. The lowest capacity plant in the W study was a modern plant designed using more sophisticated analytical techniques (i.e., time history analysis of coupled RCS and containment building model). The median overall response factor, $\check{F}_R$, was calculated as 1.52. The median response factors for Palo Verde, San Onofre, Waterford, and WPPSS (modern plants of Group C) were calculated in the present study as 1.88, 2.93, 1.38 and 3.07, respectively, i.e., the median response factors for Palo Verde and Waterford are comparable to that of the lowest capacity plants. However, the capacity factors of Palo Verde and Waterford equipment supports are much larger than that of the lowest capacity plant (Waterford SG support $\check{F}_C$ = 8.8; Palo Verde SG Snubber $\check{F}_C$ = 11.4; W lowest capacity plant $\check{F}_C$ = 3.1). All of these support elements fail locally (i.e., $\check{F}_\mu$ = 1.0) and have equal median safety factor to failure beyond ASME code allowable i.e., 1.1/0.7 = 1.57. Therefore, the margins to the code allowable are larger in the case of CE RCL equipment supports. This is due to a combination of factors; support arrangement is different (the supports are tied together and to the internal structure at more locations), and the design criteria (stress allowables used in design) are different.

For Group A plants, the median response factors were calculated as 3.95 (Calvert Cliffs), 2.87 (Millstone), 3.61 (St. Lucie Unit 1) and 2.66 (St. Lucie Unit 2). These are larger than the median response factor calculated for the lowest capacity W plant; showing large conservatism in the response analysis techniques used in these early designs. However, the median capacity factors for the equipment supports in these plants were calculated as 2.00 (Calvert Cliffs), 3.36 (Millstone Unit 2), and 2.75 (St. Lucie Units 1 and 2). These are smaller than that of the lowest capacity W plant. Therefore, for the early plants, the response calculations were more conservative and the equipment support design was less conservative than for the modern plants. The net result is that the probability of indirect DEGB in CE RCL piping is generally lower than that of the lowest capacity W plant.

3.3 DISCUSSION

In the following, the sensitivity of the study results to some important parameters are discussed. The impact of potential gross design and construction errors is studied.

3.3.1 Sensitivity of Results

An important variable influencing the calculated P_{DEGB} value is the seismic hazard at the site. For some plants, site-specific seismic hazard curves were available and these were used in estimating P_{DEGB}. Since these seismic hazard studies (Palo Verde and San Onofre) appeared to be optimistic when compared to the seismic hazard studies performed for the eastern and midwestern United States sites, alternative sets of seismic hazard curves were used to calculate P_{DEGB}. Although the calculated value of P_{DEGB} was found to be sensitive to the seismic hazard curves used, the values of P_{DEGB} reflecting the more conservative sets (generic or San Onofre Set 2) were seen to be much less than the P_{DEGB} value for the lowest capacity W plant.

For Group A plants, generic seismic hazard curves were utilized. The wide spread of the uncertainty in these generic hazard curves is expected to cover all the sites in the Eastern and Midwestern U.S. If site-specific hazard curves are used for any plant, the calculated P_{DEGB} should be lower than that reported in Table 3-1. For Palisades, the use of generic seismic hazard curves may be too conservative.

3.3.2 Design and Construction Errors

The calculation of the probability of indirect DEGB in this study was based on extrapolating the CE calculated seismic code margins to the margins against ultimate failure of the equipment supports. This extrapolation assumed that there were no gross errors in the design and construction of the RCL equipment supports. Gross errors are very unlikely in an important system such as the reactor coolant loop which is usually designed and installed under the careful supervision of the reactor vendor. However, the topic of design and construction errors (DCE) in nuclear power plants has been brought up on many previous occasions. The concern is that potential gross DCE's may reduce the safety margins well below the calculated values and that the probability of indirect DEGB may be higher than calculated. This possibility was examined in depth in the Load Combination Program phase on the Westinghouse reactors (Ravindra, et al, 1983). Several sensitivity studies were conducted to evaluate the significance of potential DCE's. It was concluded that only gross errors of implausible magnitude may substantially increase the P_{DEGB} values beyond the calculated levels.

In the present report, a description of the quality assurance and quality control procedures adopted by CE in the design and construction of reactor coolant loop equipment supports is included to qualitatively support the assertion regarding the absence of gross errors (Appendix B).

A review of Tables 3-1 and 3-2 indicates that the lowest median factor of safety for plants in the Group A plants, other than Palisades, ranges from 7.3 to 9.9. The critical support is the reactor coolant pump horizontal support (snubber or strut). A gross error that could reduce this safety factor could be the use of a wrong material, improper connection to the internal structure, or error in the calculation of the SSE design force in the member. The RCP horizontal strut support at Calvert Cliffs, which has a median factor of safety of 7.9, is made of A36 steel. Use of a wrong grade of steel can only reduce this factor marginally (because the lowest grade steel has a yield stress of 30 ksi). The RCP supports in question in the other plants are snubbers which are prefabricated. Improper connection may not be a severe problem since there are several bolts connecting the snubber to the concrete internal structure and a significant number of the bolts being improperly installed is a very unlikely event. It is assumed that the quality assurance procedures of CE have eliminated any gross design errors (i.e., in the calculation of SSE force) in a system as important as RCL piping. Even if a gross error is present that would reduce the capacity of the support element to as low as 2/3 of its "error-free" capacity, the resulting increase in the probability of indirect DEGB would be less than an order of magnitude. The 90% confidence value of P_{DEGB} would still be lower than the value calculated for the lowest seismic capacity W reactor.

The plants in Group C have much larger seismic margins and are not sensitive to gross errors of plausible magnitude.

It is concluded that the quality assurance and quality control procedures adopted for this important system combined with the lack of sensitivity of P_{DEGB} to gross errors of plausible magnitude make the issue of gross design and construction errors unimportant in using the results of this study.

3.4 SUMMARY AND CONCLUSIONS

In this study, the probability of indirectly-induced DEGB of RCL piping in CE reactors was calculated. Three groups of CE reactors were studied. Group A contains plants of early design; Group C includes more modern plants and Group B has one plant with stainless steel RCL piping. The seismic margins to ultimate failure of the equipment supports (RPV, steam generator, and reactor coolant pump) were estimated using the design information provided by CE. Appropriate seismic hazard curves (generic or site-specific) were used along with these seismic margins to calculate the indirect DEGB probability.

Based on the insight gained and the results of this study, the following conclusions may be derived:

1. The probability of indirectly-induced DEGB in RCL piping due to earthquakes is very small for CE reactors. Using very conservative assumptions, the 90% confidence value of P_{DEGB} is found to be less than 3×10^{-7}/yr for modern plants and less than 5×10^{-5}/yr for older plants.

2. Sensitivity studies have shown that only very unlikely design and construction errors of implausible magnitude could substantially change the P_{DEGB} values calculated in this study.

TABLE 3-1

ANNUAL PROBABILITY OF INDIRECT DEGB

GROUP A PLANTS

Plant	Lowest Median Factor of Safety	Seismic Hazard Curves	P_{DEGB} For Subjective Probability of		
			10%	50%	90%
Calvert Cliffs (0.15g)	7.9	Generic	2.3×10^{-8}	6.1×10^{-7}	6.1×10^{-6}
Millstone #2 (0.17g)	9.6	Generic	9.0×10^{-10}	6.6×10^{-8}	1.2×10^{-6}
Palisades (0.20g)	4.1*	Generic	5.0×10^{-7}	6.4×10^{-6}	5.2×10^{-5}
St. Lucie #1 (0.10g)	9.9	Generic	1.2×10^{-8}	3.8×10^{-7}	4.1×10^{-6}
St. Lucie #2 (0.10g)	7.3	Generic	6.6×10^{-8}	1.4×10^{-6}	1.1×10^{-5}
Westinghouse Lowest Capacity Plant	4.65**	Generic	2.3×10^{-7}	3.3×10^{-6}	2.3×10^{-5}

* $\beta_R = 0.35$ $\beta_U = 0.50$

** $\beta_R = 0.23$ $\beta_U = 0.35$

TABLE 3-2

ANNUAL PROBABILITY OF INDIRECT DEGB

GROUP C PLANTS

Plant (SSE)	Lowest Median Factor of Safety	Seismic Hazard Curves	P_{DEGB} For Subjective Probability of		
			10%	50%	90%
Palo Verde (0.25g)	21.4	Site Specific	4.0×10^{-19}	3.8×10^{-16}	1×10^{-13}
		Generic	2.4×10^{-12}	5.4×10^{-10}	2.6×10^{-8}
SONGS 2 & 3 (0.67g)	12.0	Site Specific Set 1	3.5×10^{-18}	4.6×10^{-17}	3.2×10^{-14}
		Site Specific Set 2	5.0×10^{-17}	1.1×10^{-11}	2.1×10^{-9}
WPPSS (0.32g)	11.1	Site Specific	8.0×10^{-11}	2.9×10^{-9}	1.5×10^{-7}
Waterford (0.10g)	12.1	Generic	1.1×10^{-10}	1.3×10^{-8}	3.0×10^{-7}
Westinghouse Lowest Capacity Plant	4.65	Generic	2.3×10^{-7}	3.3×10^{-6}	2.3×10^{-5}

TABLE 3-3

RESPONSE FACTORS FOR CALVERT CLIFFS

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
Structure Spectral Shape	Soil Site; first mode frequency = 3.2 Hz Design spectrum is exceeded by the WASH 1255 median alluvium spectrum	0.94	0.21	0.10
Damping	Design damping 5%	1.09	0.09	0.10
Modeling	State-of-the-art modeling	1.00	—	0.15
Soil Structure Interaction	Spectrum defined at foundation level; no soil property variation;max.soil damping=7%	1.50	0.20	0.30
Equipment Spectral Shape	Response spectrum method Peak broadening and smoothing of floor spectra. use of envelope spectrum	1.36	—	0.19
Damping	Design damping 1% Median damping 7%	1.64	0.05	0.19
Modeling	State-of-the-art methods complex systems	1.00	—	0.20
Mode Combination	Modes combined using SRSS	1.00	0.15	—
Earthquake Component Combination	Design based on maximum horizontal plus vertical earthquake components	1.15	0.10	0.15
	Overall Response Factor F_R	3.95	0.35	0.51

TABLE 3-4

RESPONSE FACTORS FOR MILLSTONE #2

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
Structure Spectral Shape	Rock site; fundamental frequency of internal structure = 10 Hz Design spectrum is conservative wrt WASH 1255 median spectrum	1.30	0.16	0.10
Damping	Design damping 5% Median damping 7%	1.05	0.05	0.10
Modeling	State-of-the-art modeling	1.00	—	0.15
Soil Structure Interaction	Rock site; spectrum defined at foundation level	1.15	—	0.07
Equipment Spectral Shape	Uncoupled RCS model using time history analysis, multipoint time histories input at NSSS supports	1.00	—	—
Damping	Design damping = 1% Median damping = 7%	1.59	0.05	0.16
Modeling	State-of-the-art methods; complex system	1.00	—	0.20
Mode Combination	Time history analysis	1.00	—	—
Earthquake Component Combination	Design based on maximum horizontal plus vertical earthquake components	1.15	0.10	0.15
	Overall Response Factor F_R	2.87	0.20	0.37

TABLE 3-5

RESPONSE FACTORS FOR ST LUCIE #1

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
<u>Structure</u> Spectral Shape	Soil site; the response spectrum obtained from the synthetic time history is approximately median centered	1.05	0.10	0.24
Damping	Design damping 5% Median damping 7%	1.15	0.14	0.10
Modeling	Stato-of-the-art modeling	1.00	—	0.15
Soil Structure Interaction	Spectrum defined at foundation; soil property variation, maximum soil damping = 10%, current SSI methods	1.65	0.20	0.30
<u>Equipment</u> Spectral Shape	Time history input at multipoint supports	1.00	—	—
Damping	Design damping 1% Median damping 7%	1.57	0.21	0.07
Modeling	State-of-the-art methods, complex systems	1.00	—	0.20
Mode Combination	Time history analysis	1.00	—	—
Earthquake Component Combination	Design based on maximum horizontal plus vertical earthquake components	1.15	0.10	0.15
	Overall Response Factor F_R	3.61	0.35	0.49

TABLE 3-6

RESPONSE FACTORS FOR ST. LUCIE #2

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
Structure Spectral Shape	Soil site; important frequencies 1.4 Hz and 3.4 Hz. compared the time history spectrum with WASH 1255 median spectrum	1.21	0.24	0.10
Damping	Design damping 7% Median damping 7%	1.00	0.14	0.10
Modeling	State-of-the-art modeling	1.00	—	0.15
Soil Structure Interaction	Spectrum defined at foundation; soil property variation; maximum soil damping 10%, current SSI methods	1.65	0.20	0.30
Equipment Spectral Shape	Time history imput at multipoint supports	1.00	—	—
Damping	Design damping 2% Median damping 7%	1.33	0.07	0.21
Modeling	State-of-the-art methods; complex systems	1.00	—	0.20
Mode Combination	Time history analysis	1.00	—	—
Earthquake Component Combination	Three dimensional time history analysis, randomness in phasing of time histories	1.00	0.10	—
	Overall Response Factor F_R	2.66	0.36	0.47

TABLE 3-7

RESPONSE FACTORS FOR SAN ONOFRE 2 & 3

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
Structure Spectral Shape	Median stiff-soft foundation; site specific spectrum used; dominant modal frequency 1.7 Hz. Synthetic time history is conservative	1.40	0.15	0.10
Damping	Soil modes dominate response Design structural damping = median	1.00	0.10	0.10
Modeling	State-of-the-art modeling	1.00	—	0.15
Soil Structure Interaction	State-of-the-art methods, input defined at foundation, soil property variation, maximum soil damping = 10%	1.65	0.20	0.30
Equipment Spectral Shape	Coupled time history analysis	1.00	—	—
Damping	Soil site; equipment damping variation has little influence on floor response	1.10	0.10	0.10
Modeling	State-of-the-art methods, complex system	1.00	—	0.20
Mode Combination	Time history analysis	1.00	—	—
Earthquake Component Combination	Design based on maximum horizontal plus vertical earthquake components	1.15	0.10	0.15
	Overall Response Factor F_R	2.93	0.30	0.45

RESPONSE FACTORS FOR WATERFORD #3

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
Structure Spectral Shape	Very soft foundation, first mode frequency = 1.07 Hz. Design spectrum conservative wrt WASH 1255	1.20	0.10	0.10
Damping	Soil site; structural damping Variation has minimum effect	1.00	0.10	0.10
Modeling	State-of-the-art modeling	1.00	---	0.15
Soil Structure Interaction	Little amplification through soil column state-of-the-art methods; soil property variation had no effect	1.00	0.10	0.10
Equipment Spectral Shape	Time history analysis	1.00	---	---
Damping	Soil site; equipment damping variation has little influence on floor response	1.00	0.10	0.10
Modeling	State-of-the-art methods; complex system	1.00	---	0.20
Mode Combination	Time history analysis	1.00	---	---
Earthquake Component Combination	Design based on maximum horizontal plus vertical earthquake components	1.15	0.10	0.15
	Overall Response Factor F_R	1.38	0.22	0.35

TABLE 3-9

RESPONSE FACTORS FOR PALO VERDE

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
Structure Spectral Shape	Soil site; composite modal damping = 11.5%; synethetic time history used had a margin of 1.1 for 1.7 - 8 Hz	1.19	0.10	0.10
Damping	Soil site; structural damping variation has minimal influence	1.00	0.10	0.10
Modeling	State-of-the-art modeling	1.00	—	0.15
Soil Structure Interaction	State-of-the-art methods, current NRC criteria for input definition, parametric variation	1.44	0.20	0.25
Equipment Spectral Shape	Coupled time history analysis	1.00	—	—
Damping	Soil site; motion is highly filtered; no marked effect due to damping variation	1.10	0.10	0.10
Modeling	State-of-the-art methods; complex system	1.00	—	0.20
Mode Combination	Time history analysis	1.00	--	—
Earthquake Component Combination	Three dimensional time history analysis; randomness in phasing of time histories	1.00	0.10	—
	Overall Response Factor F_R	1.88	0.28	0.39

TABLE 3-10

RESPONSE FACTORS FOR WPPSS #3

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
<u>Structure</u> Spectral Shape	Rock site; Internal structure frequency = 15.3 Hz. Design spectrum conservative with respect to WASH 1255	1.44	0.20	0.10
Damping	Design damping = 2.5% Median damping = 7%	1.33	0.06	0.08
Modeling	State-of-the-art modeling	1.00	—	0.15
Soil Structure Interaction	Rock site	1.00	—	0.05
<u>Equipment</u> Spectral Shape	Time history analysis with multi-point time histories input at NSSS supports	1.00	—	—
Damping	Design damping = 2% Median damping = 7%	1.60	0.10	0.25
Modeling	State-of-the-art methods Complex system	1.00	—	0.20
Mode Combination	Time history analysis	1.00	—	—
Earthquake Component Combination	Three dimensional time history analysis; randomness in phasing of time histories	1.00	0.10	—
	Overall Response Factor F_R	3.07	0.25	0.38

TABLE 3-11

RESPONSE FACTORS FOR FT. CALHOUN

Response Factor	Basis for Response Factor Evaluation	Median Factor Safety	β_R	β_U
<u>Structure</u> Spectral Shape	Soil site, El Centro and Taft records used in design; dominant mode frequency =3.02Hz Comparison with WASH 1255 median alluvium spectrum	0.95	0.18	0.10
Damping	Design damping 2%, Comparison with floor spectra at median damping of 7%	1.45	0.08	0.10
Modeling	Simple model	1.00	-	0.20
Soil Structure Interaction	Soil strata upto 70 ft; simple SSI analysis	1.30	0.15	0.21
<u>Equipment</u> Spectral Shape	Floor spectra smoothed; psuedo dynamic analysis; no allowable for multi-mode response and frequency uncertainties	1.00	0.15	0.20
Damping	Design damping 2%, Median = 7% comparison of floor spectra	1.57	0.10	0.20
Modeling	Complex systems, static analysis of RCL system	1.00	—	0.25
Mode Combination	Psuedo-dynamic analysis	1.00	0.15	—
Earthquake Component Combination	Design based on maximum horizontal plus vertical earthquake components	1.15	0.10	0.15
	Overall Response Factor F_R	3.23	0.36	0.52

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NOMENCLATURE

Symbol	Definition
A	Peak ground acceleration; a random variable.
A_c	Ground acceleration capacity.
A_{SSE}	Safe shutdown earthquake peak horizontal ground acceleration.
a	Specific value of ground acceleration.
b	Richter slope parameter.
C	Capacity of a structural element, $\check{C}$ = median; $\bar{C}$=mean, β_C= logarithmic standard deviation.
d	Closest distance to the surface projection of the fault rupture.
F	Factor of safety; $\check{F}$ = median, $\bar{F}$ = mean.
F_C	Capacity factor.
F_δ	Damping factor representing the variability in response due to difference in actual damping and design damping.
F_{EC}	Earthquake component combination factor accounting for the variability in response due to the method used in combining the earthquake components.
F_M	Modeling factor accounting for the uncertainty in response due to modeling assumptions.
F_{MC}	Mode combination factor accounting for the variability in response due to the method used in combining dynamic modes of response.
F_{RE}	Equipment response factor.
F_{RS}	Structure response factor.

NOMENCLATURE (Continued)

Symbol	Definition
F_S	Strength factor representing the ratio of ultimate strength (or strength at loss-of-function) to the stress calculated for reference earthquake acceleration (A_{SSE}).
F_{SA}	Spectral shape factor representing the variability in ground motion and the associated ground response spectra and how they affect the response.
F_{SSI}	Factor to account for the effect of soil-structure interaction.
F_a	Faulted allowable stress in buckling.
F_u	Specified ultimate capacity of snubber.
F_{ult}	Ultimate buckling strength of a column.
$f_A(a)da$	Frequency of occurrence of earthquakes with peak ground acceleration between a and a+da.
ℓ	Length of column between support points.
M	Moment magnitude.
m_b	Bodywave magnitude.
P_{DEGB}	Probability of double-ended guillotine break of RCL piping.
P_N	Normal operating load.
P_T	Total load on the structural element.
R	Response of structural element or equipment.
r	Radius of gyration, distance from the site to the earthquake source.

NOMENCLATURE

Symbol	Definition
S	Strength of structural element for the particular failure mode.
S_y	Specified yield strength of material.
X	Normalized peak ground acceleration obtained by dividing A by A_{SSE} for the plant.
$\beta_{(\cdot),R}$	Logarithmic standard deviation representing the inherent randomness of the variable specified in parenthesis.
$\beta_{(\cdot),U}$	Logarithmic standard deviation representing the uncertainties in the parameter (median) describing the variable specified in parenthesis.
μ	Ductility ratio.
λ	Slenderness parameter.

APPENDIX A

DESIGN INFORMATION FOR PALO VERDE NUCLEAR GENERATING STATION

GLOSSARY

Activity Rate	Mean annual rate of occurrence of earthquakes over a seismic source.
Attenuation	Decrease in the intensity of ground shaking with distance.
DEGB	A postulated event of an instantaneous double-ended guillotine break of the reactor coolant loop piping.
Factor of Safety	The ratio of the ground acceleration capacity A to the SSE acceleration used in plant design.
Failure Mode	The way in which a component may fail to perform its intended function. Examples of failure modes are excessive deformation, rupture of the pressure boundary, relay chatter and binding of a valve.
Fragility	Conditional probability that a structure or equipment would fail for a specified ground motion or response parameter value.
Ground Acceleration Capacity	The seismic capacity of a structure or equipment measured in terms of the peak ground acceleration value at which it would fail.
Inherent Randomness	The variability inherent to a physical phenomenon; it cannot be reduced by more detailed evaluation or by gathering of more data.
Magnitude	Magnitude is a measure of the size of an earthquake and is related to the energy released in the form of seismic waves. Richter magnitude (m) is equal to the common logarithm of the maximum trace amplitude (expressed in microns) written by a standard torsion seismometer (free period 0.8 sec, damping ratio about 50:1, and static magnification of 2,800) at an epicentral distance of 100 km. The bodywave magnitude, m is a function of the bodywave amplitude to period ratio.

Seismic Hazard Analysis	The process of estimating the frequency distribution of the peak ground motion parameter value at the site due to earthquakes in the region.
Seismic Source	A fault or a seismotectonic province over which an earthquake may occur.
Uncertainty	Refers to the state of knowledge concerning a physical phenomenon; it can be reduced by a more detailed evaluation or by gathering of additional data.
Upperbound Magnitude	Magnitude of the largest earthquake that a seismic source is capable of producing.

Site Characteristics & Structural Response

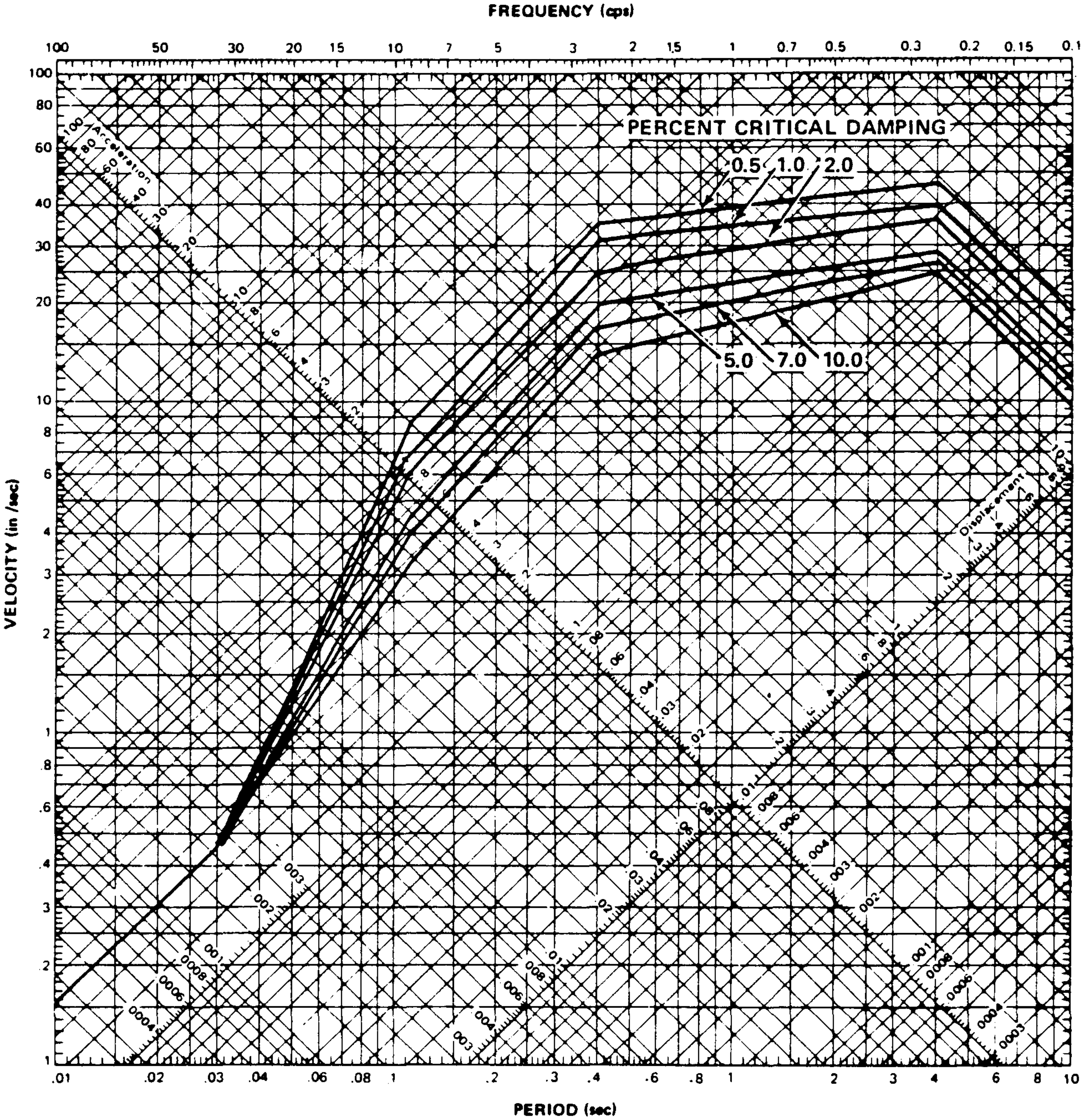
Design Ground Spectra	-	Appendix B1 (4 pages)
Zero Period Acceleration	-	0.25G SSE
	-	0.13G OBE
Spectra of Synthetic Time Histories	-	Appendix B2 (4 pages)
	-	Spectra for horizontal direction provided is representative of spectra for all three directions.
Site Characteristics	-	Multi-layer system of sand and clay over bedrock (See Appendix B3 (1 page))
Structural Damping	-	Appendix B4 (4 pages)
Fundamental Frequency of Internal Structure	-	N/A; coupled analysis of building & RCS
RCS Support Point Spectra	-	N/A; coupled analysis

NSSS Response

Method of Analysis	-	Time History, Three Dimensional
Modeling of NSSS and Structure	-	Coupled building - RCS
NSSS System Damping	-	1% OBE
	-	2% SSE
	-	See Appendix B5 (2 pages) for coupled model frequencies and composite modal damping values
Envelope or Multipoint Spectra	-	N/A; coupled model analysis
NSSS Fundamental Frequency or Frequency Range	-	See Appendix B6 (3 pages) for RCS fixed support frequencies

Opinion on Effects of Increased Seismic Excitation on Response

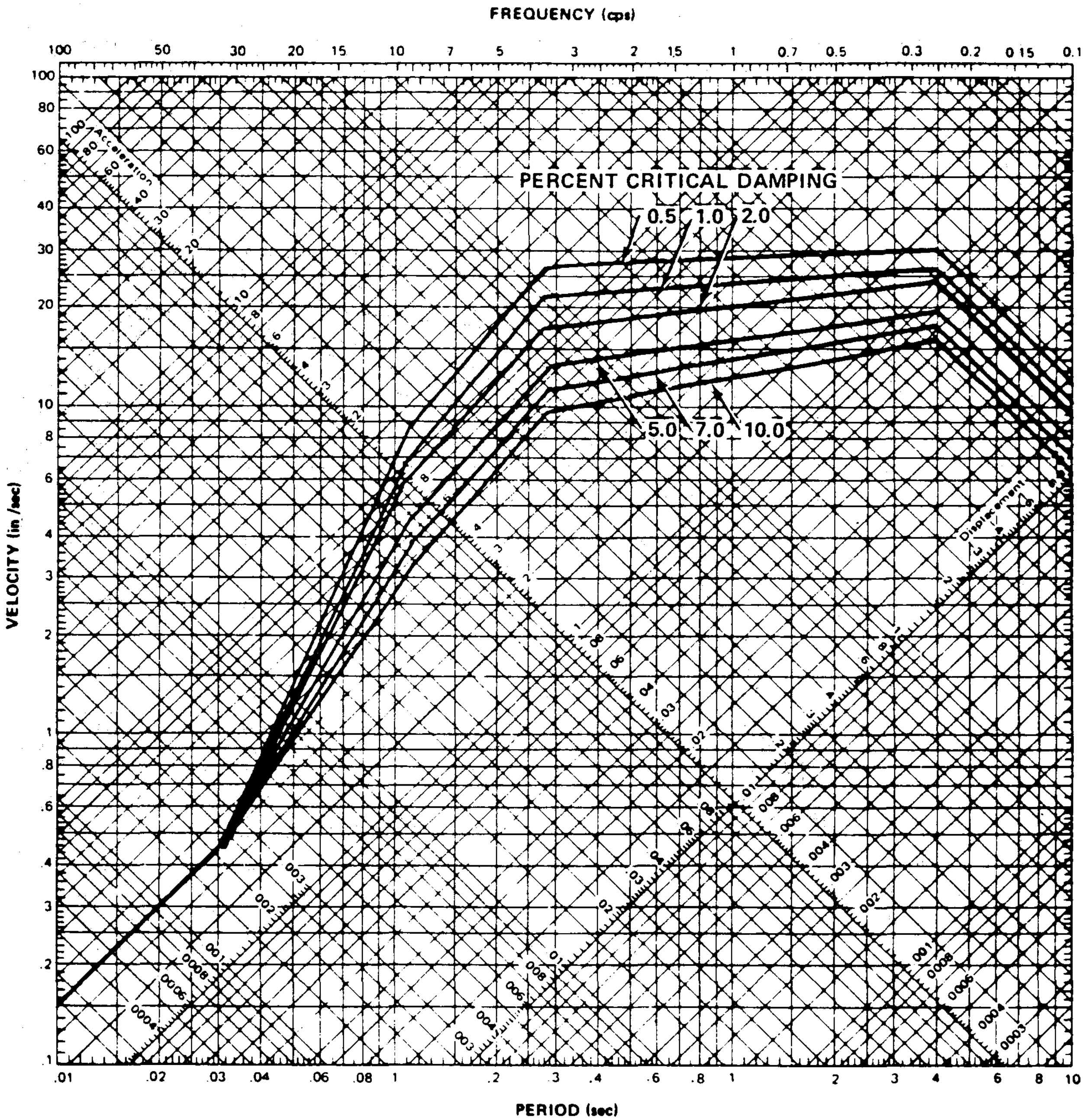
Excitation	Response
SSE	Calculated Value (CV)
2 x SSE	1.9 CV
3 x SSE	2.7 CV
5 x SSE	3.8 CV



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

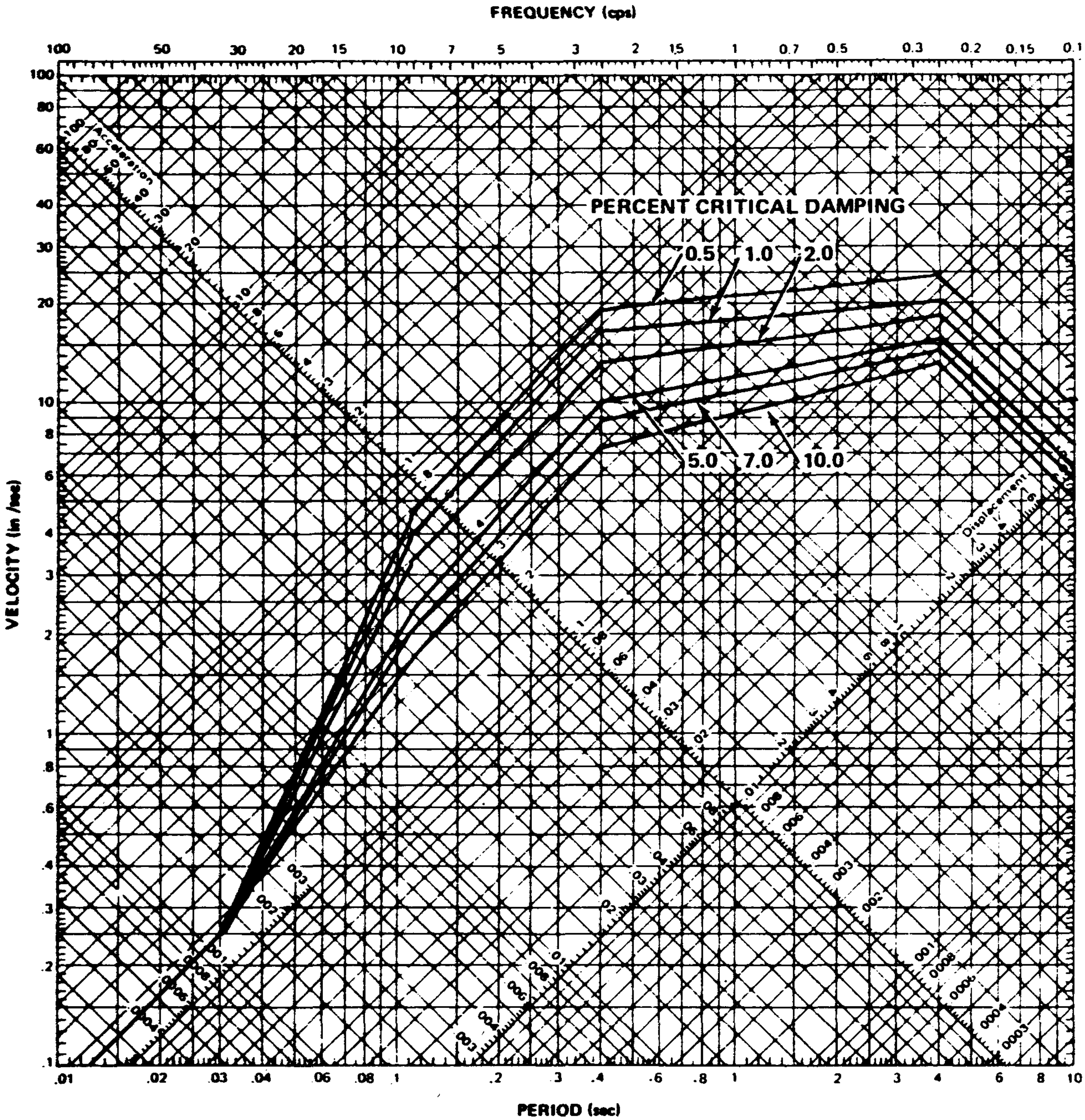
Figure 3.7-1



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

Figure 3.7-2



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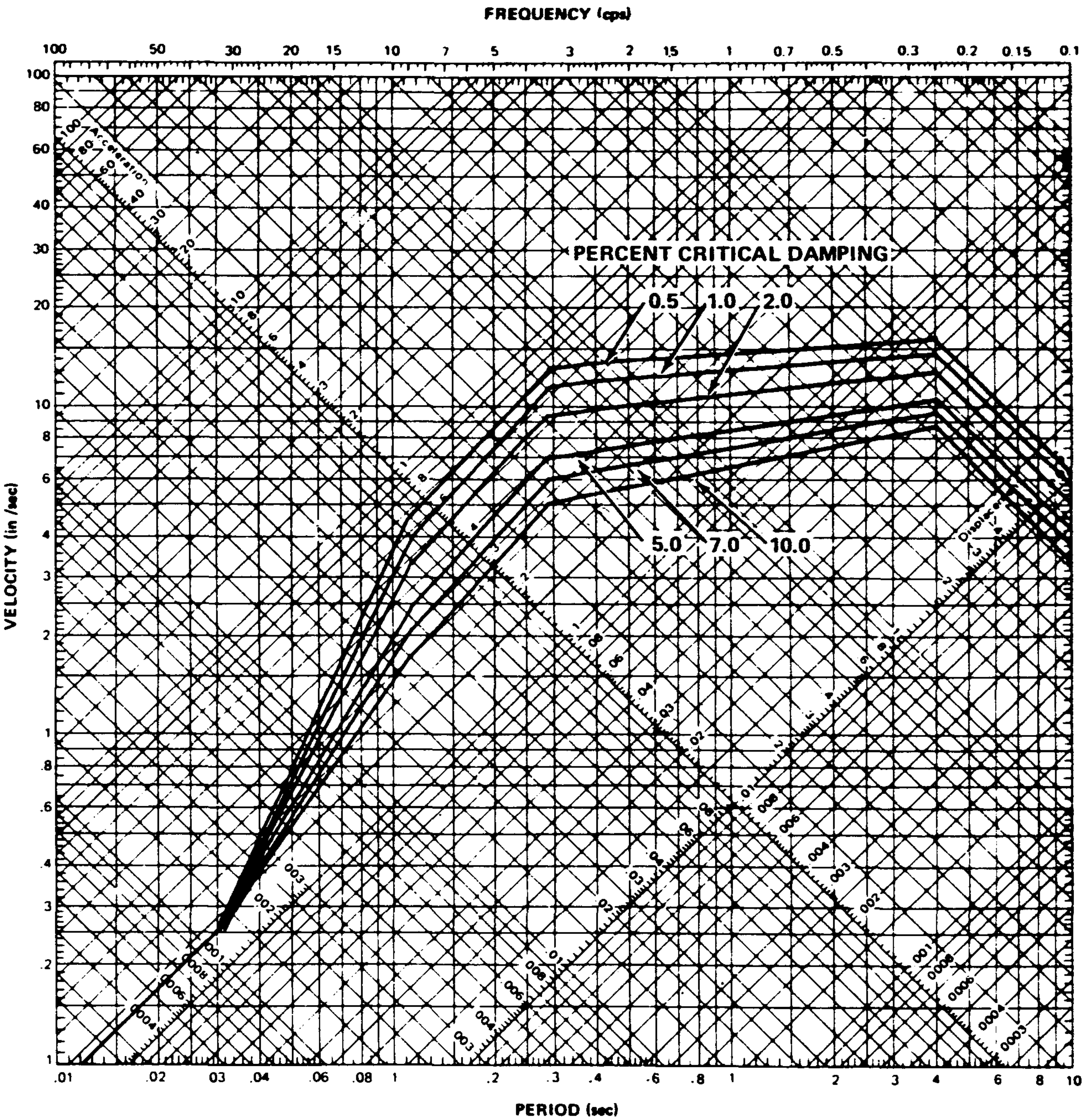
HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-3



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VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

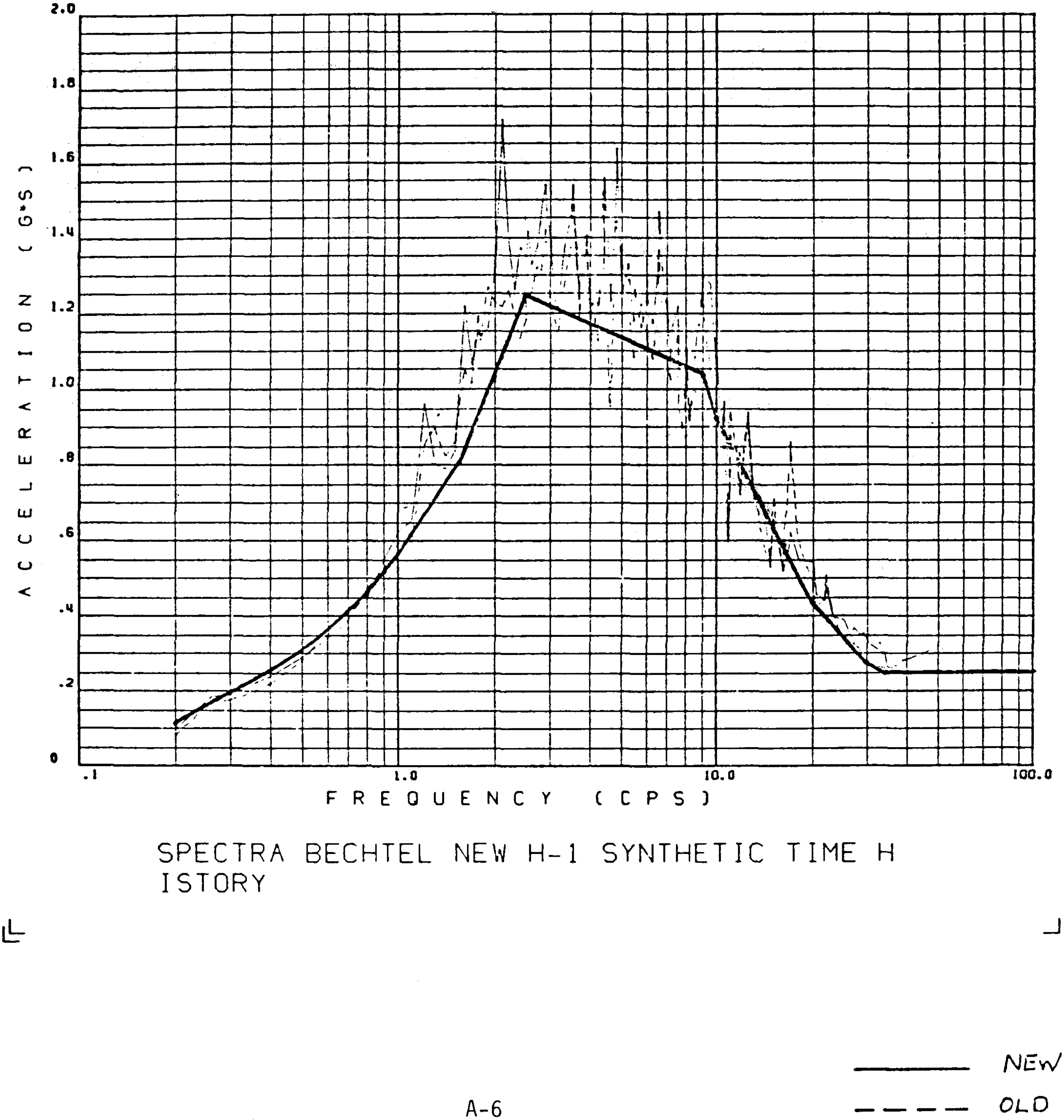
Figure 3.7-4

04/12/79 2
04 SFXFP32 1
MAR 01P7

DAMPING VALUES

1.0000,

BECHTEL CORPORATION

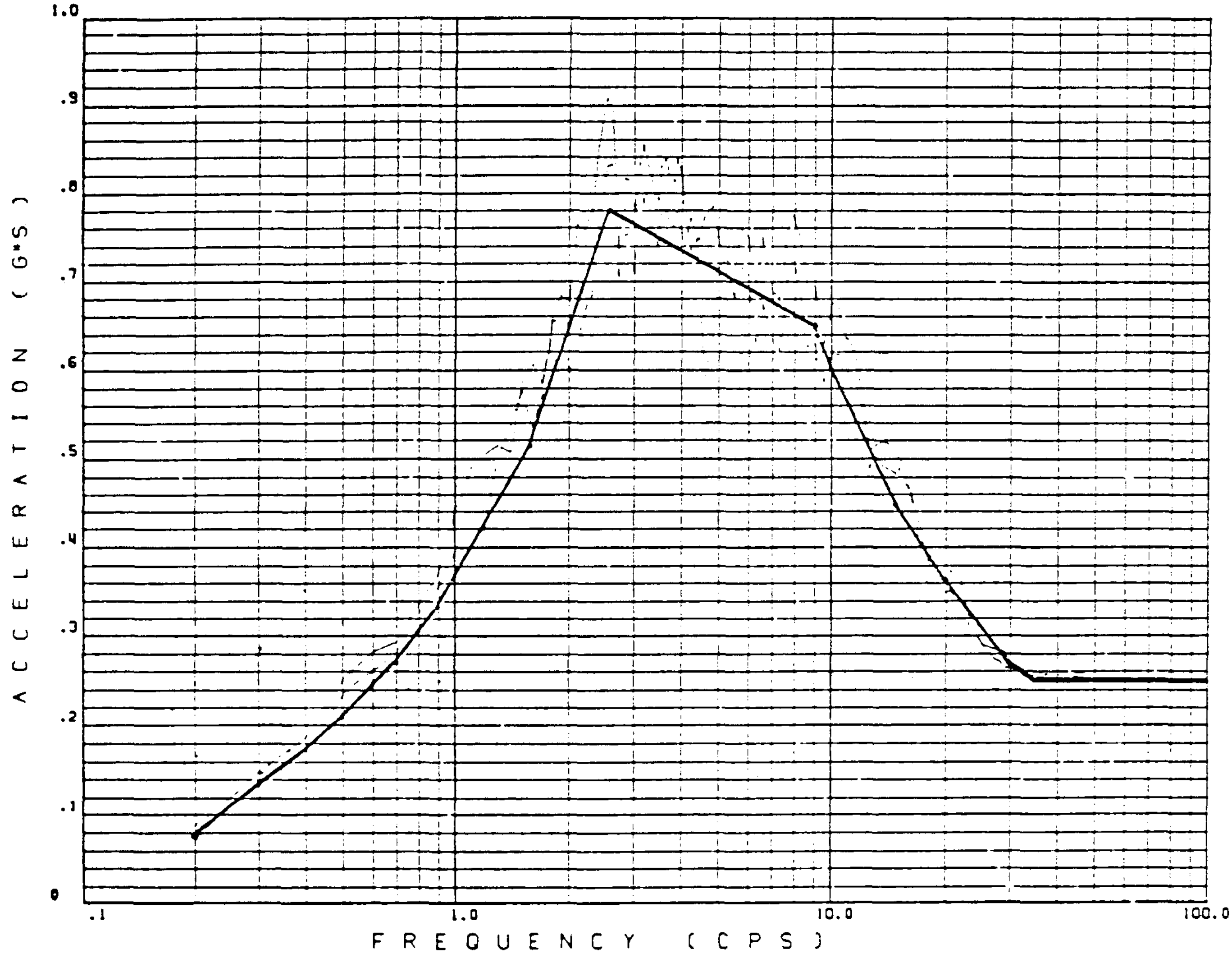


SPECTRA BECHTEL NEW H-1 SYNTHETIC TIME H
ISTORY

DAMPING VALUES

5.0000.

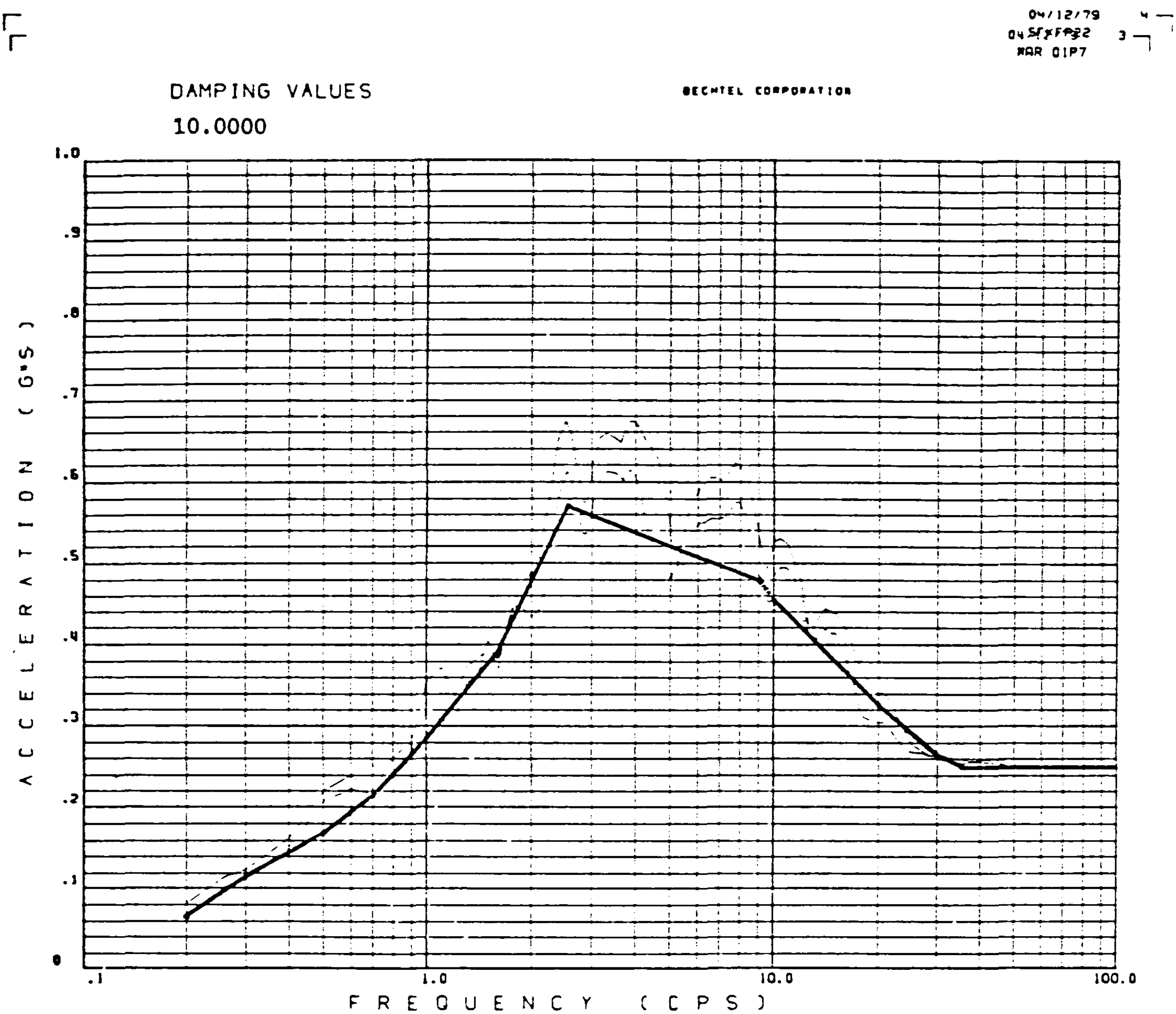
BECHTEL CORPORATION



SPECTRA BECHTEL NEW H-1 SYNTHETIC TIME H
ISTORY

——— NEW

– – – – – OLD



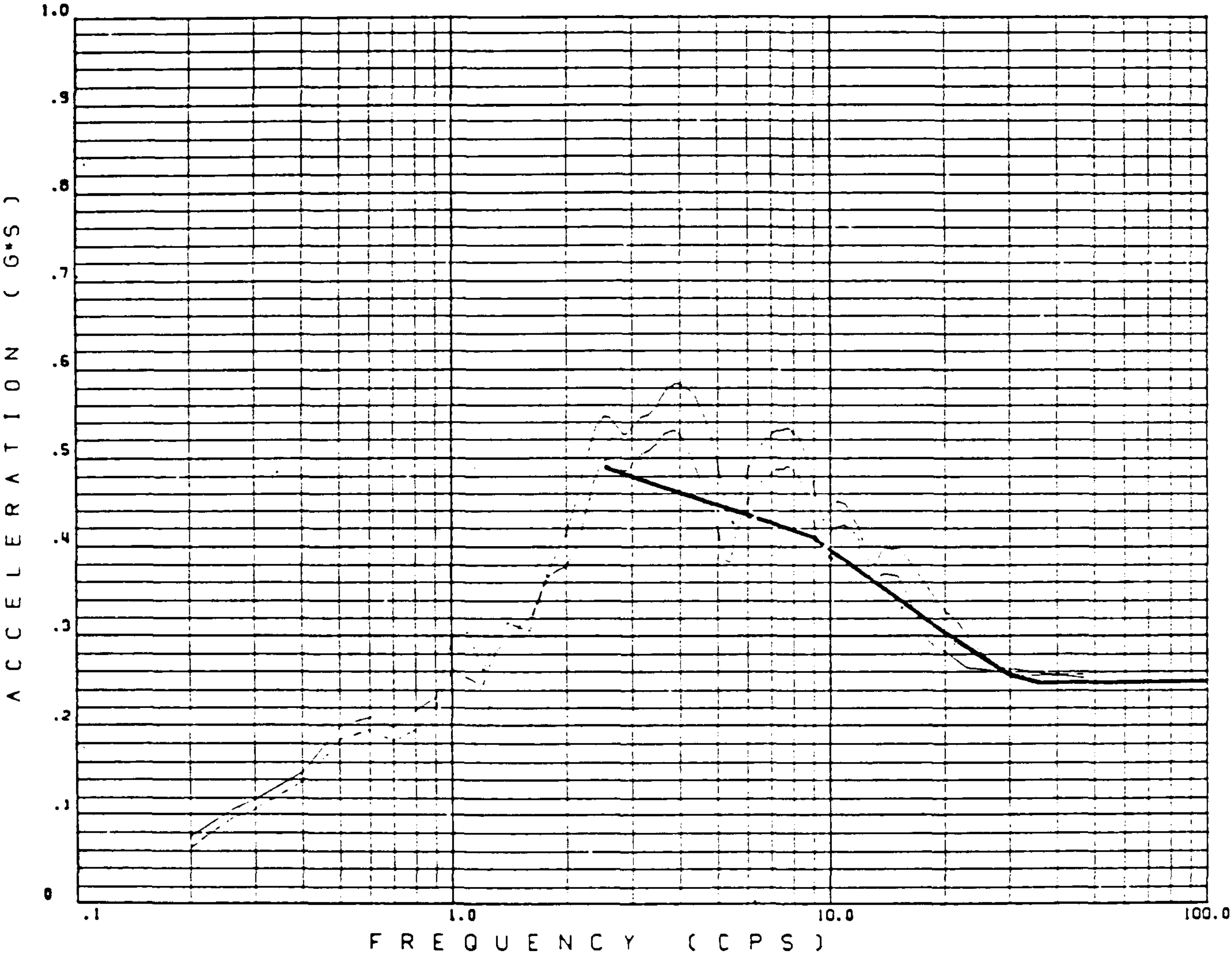
SPECTRA BECHTEL NEW H-1 SYNTHETIC TIME H
ISTORY

04/12/79 5
MAR 01P7

DAMPING VALUES

15.0000

BECHTEL CORPORATION



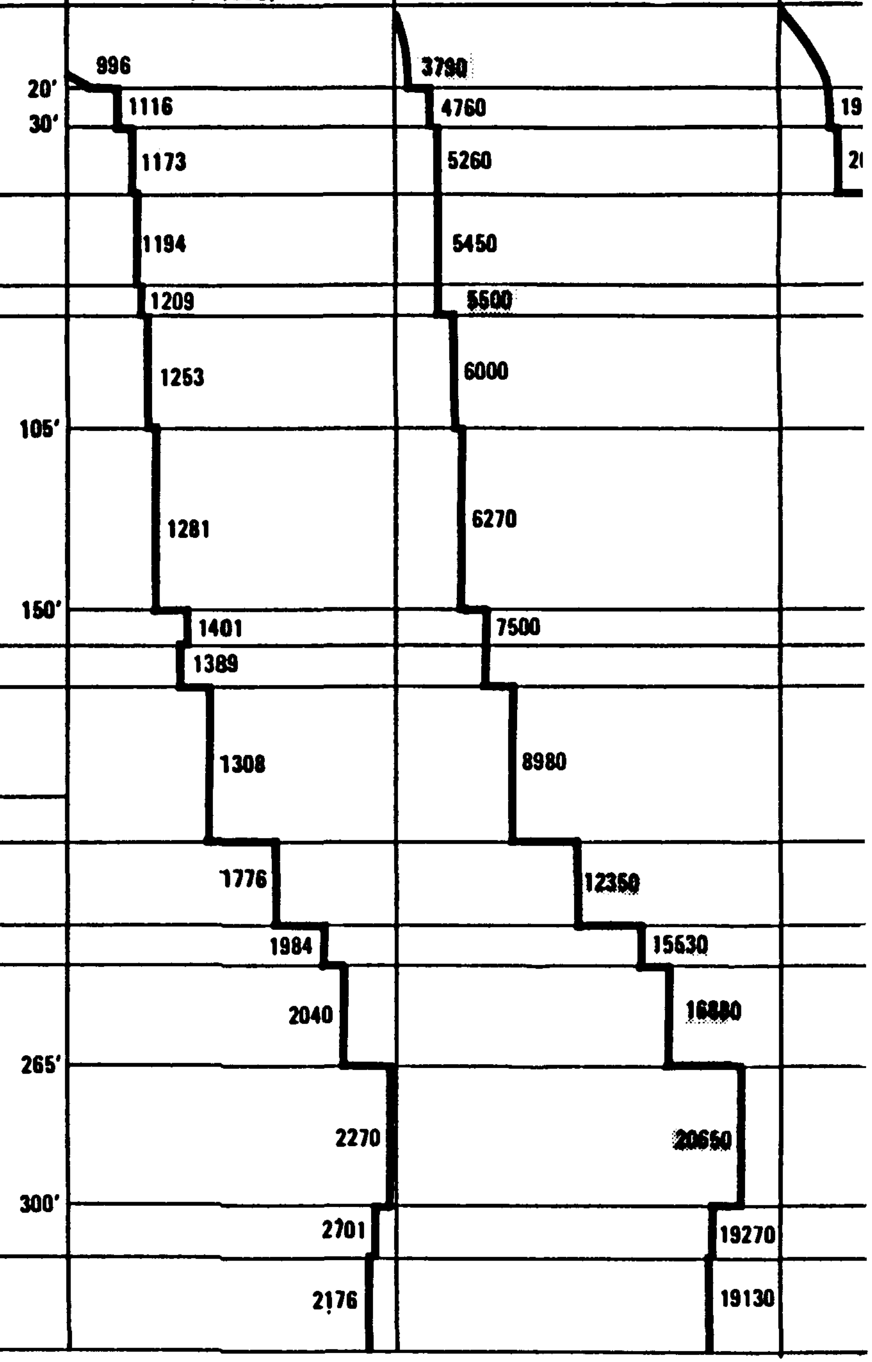
SPECTRA BECHTEL NEW H-1 SYNTHETIC TIME H
ISTORY

NEW

OLD

DEPTH (FT)	LAYER DEPTH (FT.)	DESCRIPTION	LAYER THICKNESS	UNIT WEIGHT (PCF)	POISSON'S RATIO
ERAGE PERCHED TER LEVEL 44'		SAND (2)	47'	123	.27
50	47'	CLAY (1)	23'	121	.44
	70'	SAND (2)	7'	121	.48
100	77'	CLAY (1)	82'	123	.47
150	159'	SAND (2)	10'	126	.465
	169'	CLAY (1)	27'	127	.46
200	196'	SAND (2)	12'	127	.46
	206'	CLAY (1)	20'	126	.45
	228'	SAND (2)	10'	127	.44
250, 300	238'	CLAY (1)	73	128	.44
	311'	SAND (2)	23'	130	.44
350	334'	BEDROCK			

IN-SITU SHEAR WAVE VELOCITY (FT/SEC) | LOW STRAIN SHEAR MODULUS (KSF) | LOW S



A-10

NOTES: 1. LOW-STRAIN SHEAR MODULI AT VARIOUS DEPTHS ARE AVERAGE VALUES

3.7.1.2 Design Time History

A synthetic earthquake time-history is generated because the response spectra of a recorded earthquake motion does not necessarily envelop the site design spectra. A 24-second earthquake duration is used which is comparable to the strong motion duration of the earthquake records used, and is therefore considered to be adequate for the time-history type of analysis of structures and equipment. Comparison between the free-field time-history response spectra and the design spectra for both horizontal and vertical motions, and the basis for the generation of the synthetic time history are discussed in Section 2.5 of BC-TOP-4-A. The time history of the design earthquake is assumed to be the free-field motion at the base of the foundation for each Category I structure.

3.7.1.3 Critical Damping Values

Refer to CESSAR Section 3.7.1.3 for NSSS seismic systems. 5

The damping values (percent of critical damping) used for seismic design of Category I structures are listed in table 3.7-1, and are the same as those specified in Regulatory Guide 1.61. Strain-corrected damping values for the foundation materials were developed using the computer program SHAKE [1] and soil properties from field and laboratory test results. The average strain-dependent damping ratios for clay and sand are shown in figures 3.7-5 and 3.7-6, respectively.

Frequency-dependent soil damping values were obtained using the LUCON computer program[2] and the strain-dependent relationships for use in the time-history analysis of lumped-mass models of structure-foundation systems. For the design response spectrum method of analysis, soil damping values for the structure-foundation system were computed using the expressions given in Table 3-2 of BC-TOP-4-A.

Table 3.7-1
DAMPING VALUES
(PERCENT OF CRITICAL DAMPING)

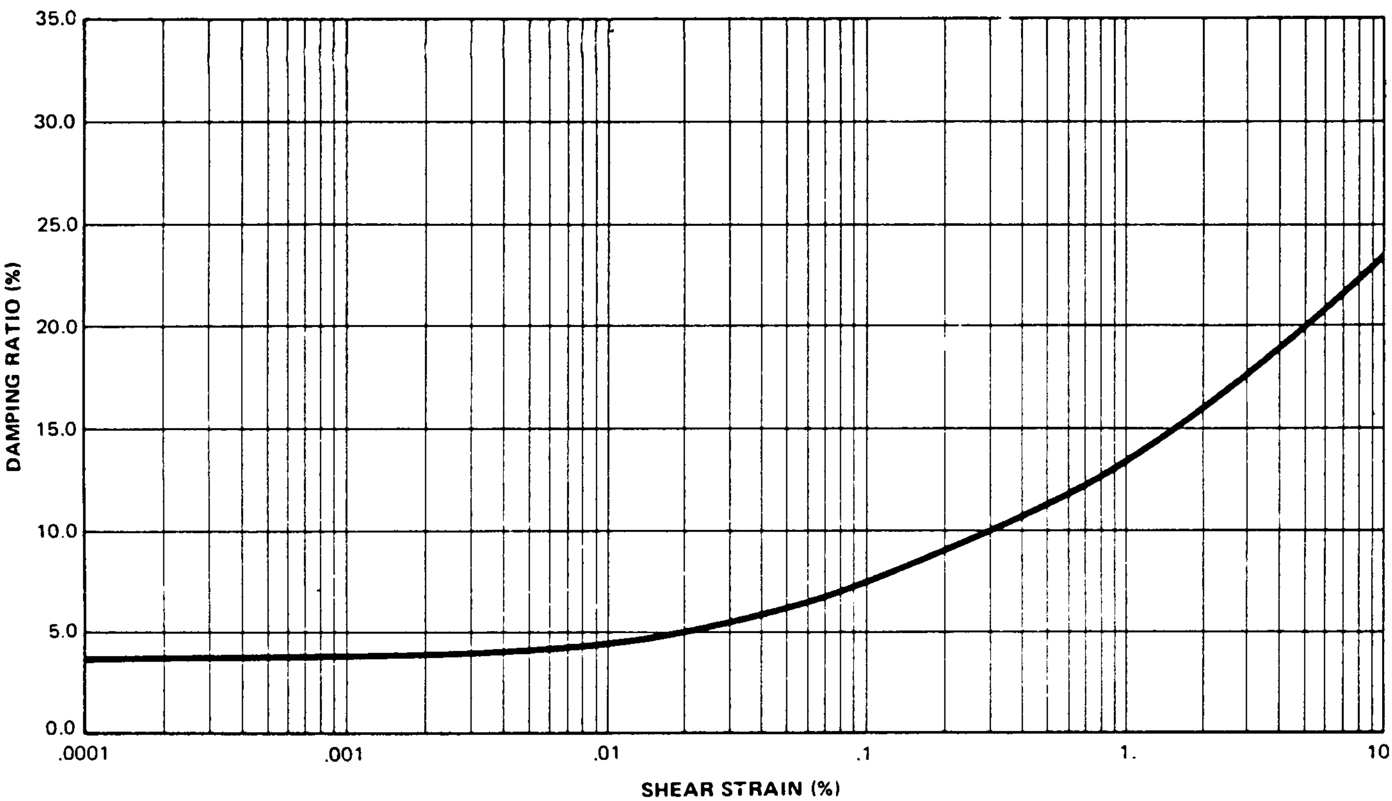
Structure or Component	Operating Basis Earthquake	Safe Shutdown Earthquake
Equipment and large-diameter piping systems, pipe diameter greater than 12 in.	2	3
Small-diameter piping systems, diameter equal to or less than 12 in.	1	2
Welded steel structures	2	4
Bolted steel structures	4	7
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7

The applicable allowable design levels are given in section 3.8 for the various loading combinations which include seismic loadings.

3.7.1.4 Supporting Media for Seismic Category I Structures

For purposes of the seismic analysis, the site is assumed to be a multi-layer system consisting of soil over bedrock. The approximate depth of soil deposit over bedrock for each unit at the site is as follows:

	Unit 1	Unit 2	Unit 3
Depth of Soil, ft	330	350	295



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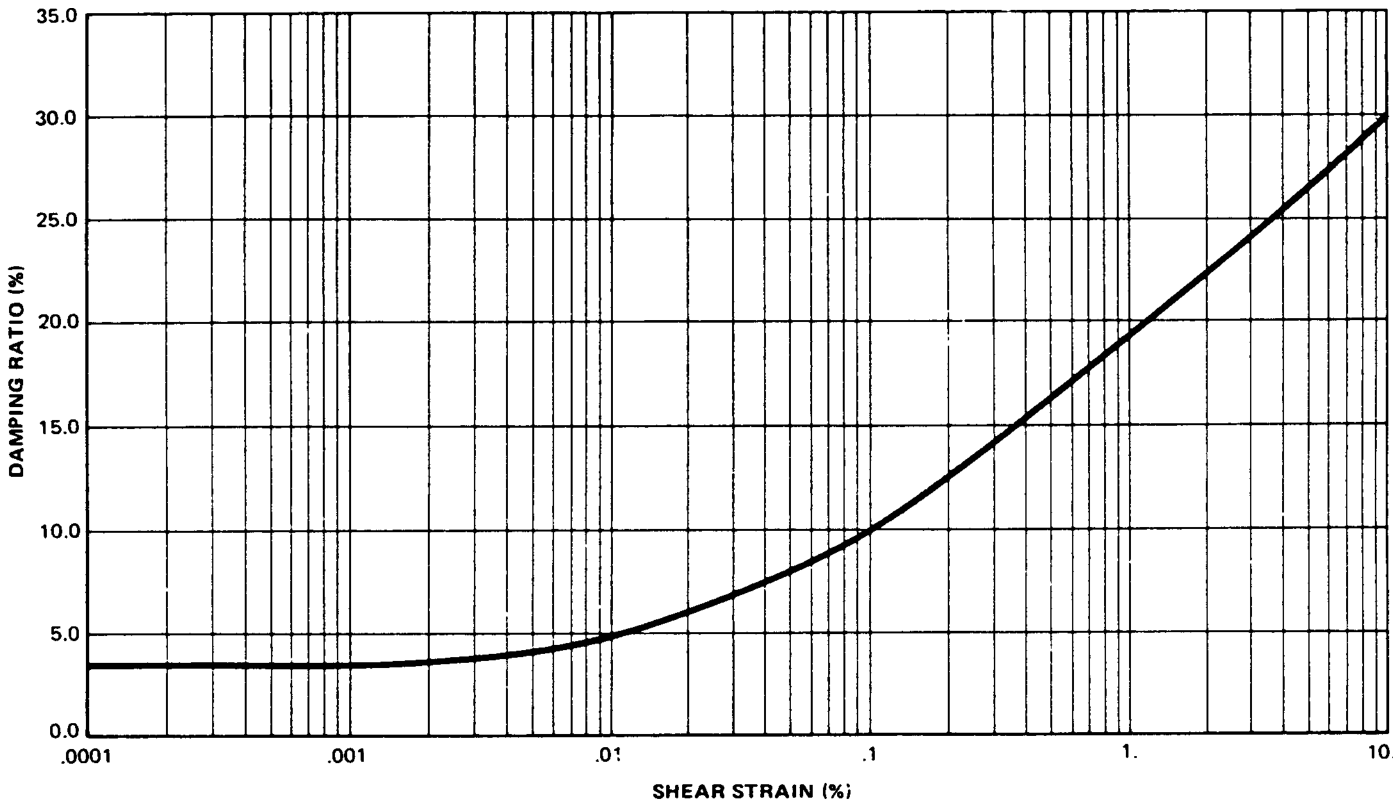
DAMPING VS. STRAIN - CLAY

FIGURE 3.7-5



Palo Verde Nuclear Generating Station
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DAMPING VS. STRAIN - SAND

FIGURE 3.7-6

FREQUENCIES AND COMPOSITE MODAL DAMPING VALUES*

MODE NO.	OPERATING BASIS EARTHQUAKE (OBE) Frequency CPS	OPERATING BASIS EARTHQUAKE (OBE) Composite Modal Damping Value	SAFE SHUTDOWN EARTHQUAKE (SSE) Frequency CPS	SAFE SHUTDOWN EARTHQUAKE (SSE) Composite Modal Damping Value
1	1.735	1.1	1.668	11.5
2	1.736	1.1	1.673	11.5
3	1.877	10.4	1.740	2.3
4	1.883	10.5	1.740	2.4
5	3.266	63.8	3.266	63.8
6	4.235	50.1	3.746	56.5
7	4.249	49.8	3.762	56.3
8	7.734	2.0	7.724	3.0
9	7.858	1.0	7.858	2.0
10	10.060	5.4	9.981	9.3
11	10.691	5.7	10.608	9.8
12	12.018	32.6	10.792	36.4
13	12.190	1.2	12.189	2.2
14	12.500	1.0	12.500	2.0
15	12.873	1.0	12.872	2.0
16	12.876	1.1	12.874	2.0
17	13.209	1.2	13.207	2.3
18	13.857	1.7	13.851	2.9
19	14.685	1.0	14.685	2.0
20	14.743	1.1	14.741	2.1
21	14.836	1.0	14.836	2.0
22	14.858	1.1	14.857	2.1
23	15.831	2.7	15.816	4.7
24	17.190	10.4	16.915	8.1
25	17.567	3.6	17.551	6.5
26	17.660	3.5	17.646	6.3
27	17.870	1.1	17.870	2.3
28	17.957	1.4	17.948	2.1
29	17.982	1.6	17.979	2.8
30	17.985	1.3	17.983	2.5
31	19.397	2.9	19.397	5.0
32	20.630	1.1	20.629	2.1
33	20.948	4.3	20.948	6.4
34	21.278	1.5	21.277	2.9
35	21.576	1.2	21.570	2.2
36	21.602	1.2	21.602	2.4
37	21.654	1.5	21.639	2.3
38	22.646	4.0	22.632	6.4
39	23.818	1.4	23.816	2.7
40	24.030	1.0	24.030	2.0

* Composite Modal Damping Values are expressed as a percentage of critical modal damping.

FREQUENCIES AND COMPOSITE MODAL DAMPING VALUES*

MODE	OPERATING BASIS EARTHQUAKE (OBE)		SAFE SHUTDOWN EARTHQUAKE (SSE)	
NO.	Frequency CPS	Composite Modal Damping Value	Frequency CPS	Composite Modal Damping Value
41	24.801	3.4	24.790	5.5
42	25.523	2.5	25.515	4.4
43	25.776	2.0	25.776	3.2
44	26.148	1.5	26.145	2.7
45	28.069	1.2	28.063	2.2
46	28.915	2.2	28.914	4.8
47	30.948	3.5	30.933	7.0
48	31.448	1.0	31.448	2.1
49	31.562	1.0	31.561	2.0
50	31.930	1.0	31.930	2.0
51	31.970	1.2	31.969	2.3
52	32.327	1.0	32.317	6.6
53	32.333	3.8	32.327	2.0
54	34.204	2.4	34.201	5.4
55	34.226	2.5	34.223	5.5
56	38.536	1.1	38.536	2.2
57	38.542	1.0	38.542	2.0
58	38.598	1.2	38.597	2.4
59	38.716	1.0	38.716	2.0
60	40.817	3.7	40.813	6.3
61	41.536	1.0	41.535	2.1
62	45.848	1.1	45.848	2.2
63	46.469	2.1	46.468	5.0
64	46.645	2.2	46.644	5.2
65	47.415	1.0	47.415	2.0
66	47.451	1.0	47.451	2.0
67	47.752	1.0	47.752	2.0
68	48.040	2.6	48.040	5.6
69	48.069	1.0	48.069	2.0
70	50.883	1.0	50.883	2.0
71	51.105	1.0	51.105	2.1
72	51.188	1.0	51.188	2.0
73	51.581	2.2	51.580	3.9
74	52.502	1.0	52.502	2.0
75	52.510	1.0	52.510	2.0
76	52.886	5.0	52.878	8.2
77	53.717	3.4	53.713	5.9
78	55.823	2.4	55.821	5.4
79	56.057	2.6	56.054	5.5
80	57.880	7.5	57.880	11.3

* Composite Modal Damping Values are expressed as a percentage of critical modal damping.

Table 3.7-

NATURAL FREQUENCIES AND DOMINANT DEGREES OF FREEDOM
FIXED SUPPORT REACTOR COOLANT SYSTEM

		Dominant Degrees of Freedom		
Mode No.	Frequency (Hertz)	Joint Number	Direction	Location
1	1.74	9911	Z	Reactor Internals
2	1.74	9911	X	Reactor Internals
3	12.29	9916,1103,2103 etc.	Z,X	Reactor Vessel, Pumps
4	12.60	1103,2103,etc.,9916	X	Pumps & Reactor Vessel
5	13.01	2103,4103	X	Pumps 1B & 2B
6	13.01	1103,5103	X	Pumps 1A & 2A
7	13.27	1103,2103,etc.	X	Pumps
8	13.51	1103,2103,etc.,9916	X	Pumps & Reactor Vessel
9	14.89	1103,2103,etc.	Z	Pumps
10	14.90	1103,2103,etc.	Z	Pumps
11	14.90	1103,2103,etc.	Z	Pumps
12	14.90	1103,2103,etc.	Z	Pumps
13	14.99	404,3404	X	Steam Generators
14	15.37	404,3404	X	Steam Generators
15	17.90	408,412,3408,3412	Z	Steam Generator Internals
16	17.90	408,412,3408,3412	Z	Steam Generator Internals
17	18.00	1103,2103,etc.	Y	Pumps
18	18.01	1103,2103,etc.	Y	Pumps
19	18.04	1103,2103,etc.	Y	Pumps
20	18.04	1103,2103,etc.	Y	Pumps

Table 3.7-

NATURAL FREQUENCIES AND DOMINANT DEGREES OF FREEDOM
FIXED SUPPORT REACTOR COOLANT SYSTEM

Mode No.	Frequency (Hertz)	Joint Number	Dominant Degrees of Freedom	
			Direction	Location
21	20.22	9911	Y	Reactor Vessel Internals
22	20.77	9995	Z	Reactor Vessel
23	21.52	2101,4101	Z	Pumps 1B & 2B
24	21.56	2101,4101	Z	Pumps 1B & 2B
25	21.62	1101,5101	Z	Pumps 1A & 2A
26	21.63	1101,5101	Z	Pumps 1A & 2A
27	24.10	9916	X	Reactor Vessel
28	24.24	408,3408	X	Steam Generator Internals
29	26.08	9905	X	Reactor Vessel Internals
30	26.78	404,3404	Y	Steam Generator
31	26.79	404,3404	Y	Steam Generator
32	29.51	404,3404	Z	Steam Generator
33	29.51	404,3404	Z	Steam Generator
34	31.57	2580,4580,2101,4101	X	Suction Leg Piping & Pumps 1B & 2B
35	31.72	2580,4580,2101,4104	X	Suction Leg Piping & Pumps 1B & 2B
36	32.05	1580,5580,1101,5101	X	Suction Leg Piping & Pumps 1A & 2A
37	32.08	1580,5580,1101,5101	X	Suction Leg Piping & Pumps 1A & 2A

Table 3.7-

NATURAL FREQUENCIES AND DOMINANT DEGREES OF FREEDOM
FIXED SUPPORT REACTOR COOLANT SYSTEM

Mode No.	Frequency (Hertz)	Joint Number	Dominant Degrees of Freedom: Direction	Dominant Degrees of Freedom: Location
38	32.40	9911	Y	Reactor Vessel Internals
39	38.58	2580,4580	X,Z	Suction Leg Piping 1B & 2B
40	38.58	2580, 4580	X,Z	Suction Leg Piping 1B & 2B
41	38.70	1580,5580	X,Z	Suction Leg Piping 1A & 2A
42	38.78	1580,5580	X,Z	Suction Leg Piping 1A & 2A
43	41.62	9995	Z	Reactor Vessel
44	45.94	9995	X	Reactor Vessel
45	47.81	412,3412,408,3408	X	Steam Generator Internals
46	48.14	412,3412,408,3408	X	Steam Generator Internals
47	48.40	412,3412,408,3408	Z	Steam Generator Internals
48	48.40	412,3412,408,3408	Z	Steam Generator Internals

SUPPORT SYSTEM INTEGRITY

Sheet 1. of 1

PLANT ANPP: UNITS # 1, 2 & 3. COMPONENT REACTOR VESSEL SUPPORT SUPPORTS

Item	Material	Dsgn/Fabr Code	Supplementary Mat/Fabr NDE	Welding		Bolting		Design Margin	Actual Seismic Margin
				PWHT	NDE	Pre(% Ult)	Type		
NOZZLE TO PAD WELD. (UNIT#1 ONLY)	E-8018 C3	ASME III NB	NONE	YES	MT			7.7	12.9
NOZZLE PAD	SA-533 GR.B CL-1	ASME III NB	NONE					6.2	21.6
UPPER EXPANSION PLATE ASSEMBLY (1)	MEEHANITE, GA-50	ASME III NF	NONE					21.2	21.2
UPPER LATERAL EMBED. STRUCTURE	A-516 GR.70	AISC	UT	YES	PT, MT			17	-
NOZZLE PAD TO COLUMN BOLTS	SA-540 CL-3.	ASME III NF	IMPACT TESTED M.T. U.T			52	TENSIONER (BOLT ELONG.)	13.1	73.1
COLUMNS	SA-508 CL-2	ASME III NF	IMPACT TESTED					3.1	17.2
SHELL TO LOWER KEY WELD	INCONEL 182	ASME III NB	NONE	YES	MT			4.1	21.4
R.V. LOWER KEY	SA-533 GR.B, CL-1	ASME III NB	MT					9.8	9.8
LOAD LIMITER ASSY (2)	SA-240 TYPE-304	ASME III NF	VISUAL					3.8	3.8
COLUMN BASE & ANCHOR BOLTS	A-540 GR.B22 CL-4	ASME III NF/ASTM	UT			39	TENSIONER (BOLT ELON-G)	78	-

Remarks: (1) CONSISTS OF SA-533 GR.B CL-2, SA-540 GR.B23 CL-3 w/ SA-193-GR.B7 & LIMITING PART IS MEEHANITE, GA-50

(2) CONSISTS OF A-514 GR.ALL; SA-564 GR.630; MEEHANITE GA-50. & LIMITING PART IS SA-240 TYPE-304

SUPPORT SYSTEM INTEGRITY

PLANT ANPP: UNITS #1,2&3 COMPONENT STEAM GENERATOR SUPPORT UPPER SUPPORT

Item	Material	Dsgn/Fabr Code	Supplementary Mat/Fabr NDE	Welding		Bolting		Design Margin	Actual Seismic Margin
				PWHT	NDE	Pre(% Ult)	Type		
UPPER KEY LUG TO S.G. WELD	E-8018 -C3	ASME III NB	NONE	YES	PT			3.0	74.5
LUG	SA-533 GR.B CL-1	ASME III NB	NONE					27.9	27.9
UPPER KEY SUPPORT	A 336 F5A	AISC	UT, MT	YES	UT			8.	–
UPPER KEY SUPPORT EMBEDS.	A-588	AISC	NONE					8.	–
KEY SUPPORT ANCHOR BOLTS	A-540 GR.B23 CL-1	ASME III NF/ASTM	UT, MT			75	TORQUE (BOLT ELONG)	6.	–
SNUBBER-LEVER ASSEMBLY LUG TO S.G. WELD	E-8018 -C3	ASME III NB	NONE	YES	PT			3.0	67.8
LUG (INNER)	SA-533 GRB CL-1	ASME III NB	NONE					21.9	21.9
WELD BETWEEN LUGS.	E-8018 -C3	ASME III NB	NONE	YES	MT			29.6	29.6
LUG (OUTER)	SA-533 GRB CL-1	ASME III NB	NONE					16.5	16.5
LINKS	SA-542 CL-1.	ASME III NF	IMPACT TESTED					16.4	16.4
PINS-BETWEEN LINKS/LUGS/LEVER	SA-540 GR.B23/24 CL-3	ASME III NF	IMPACT TESTED					11.6	11.6

Remarks:

SUPPORT SYSTEM INTEGRITY

Sheet 2 of 3

PLANT ANPP: UNITS #1, 2 & 3 COMPONENT STEAM GENERATOR SUPPORT UPPER SUPPORT

Item	Material	Dsgn/Fabr Code	Supplementary Mat/Fabr NDE	Welding		Bolting		Design Margin	Actual Seismic Margin
				PWHT	NDE	Pre(% Ult)	Type		
SELF-ALIGNING BEARINGS.	AMS-5643	ASME III NF	NONE					34.5	34.5
LEVER	SA-542 CL-1	ASME III NF	IMPACT TESTED					16.2	16.2
PIN BETWEEN LEVER & BRACKET	SA-540 GR.B23/24 CL-3	ASME III NF	IMPACT TESTED					17.8	17.8
BUSHING	AISI 4340	ASME III NF	NONE					25.7	25.7
WALL BRACKET	A-148-73 GR. 90/60	ASME III NF	IMPACT TESTED					12.8	12.8
BOLTS-WALL BRACKET TO STUB-COLUMN	A-540 GR.B23 CL-1.	ASME III NF/ASTM	UT			55	TENSIONER (BOLT ELONG.)	24	-
STUB COLUMN	A-588	AISC	UT	YES	UT			15	-
STUB-COLUMN ANCHOR BOLTS	A-540 GR.B23 CL-1	ASME III NF/ASTM	UT			55	TENSIONER (BOLT ELONG.)	18	-
SNUBBER ASSY.(3)	SA-540 GR.B23 CL-1	ASME III NF	NONE			57	TORQUE	7.3	7.3
SNUBBER INTER-FACE PLATE	A-588	AISC	NONE					35	-
SNUBBER ASSY. ANCHOR BOLTS.	A-540 GR.B23 CL-1	AISC	UT			55	TENSIONER (BOLT ELONG.)	18.	-

Remarks: (3) CONSISTS OF SA-106 GR.B, A-588 CL-1, SA-515 GR.70, A-148-73; SA-194 GR.7 ETC. & LIMITING PART IS SA-540 GR.B23 CL-1.

SUPPORT SYSTEM INTEGRITY

PLANT ANPP: UNITS # 1, 2 & 3 COMPONENT STEAM GENERATOR SUPPORT LOWER SUPPORT

Item	Material	Dsgn/Fabr Code	Supplementary Mat/Fabr NDE	Welding		Bolting		Design Margin	Actual Seismic Margin
				PWHT	NDE	Pre(% Ult)	Type		
WELD-SKIRT TO STEAM GENERATOR	LINDE #0091	ASME III NB	NONE	YES	MT			5.9	33.9
SKIRT	SA-533 GR.B CL-1	ASME III NB	NONE					5.9	33.9
WELD: SKIRT TO FLANGE	LINDE #0091	ASME III, NB	NONE	YES	MT			4.1	18.6
FLANGE	SA-533 GR.B CL-1	ASME III, NB	NONE					4.1	18.6
SKIRT HOLD DOWN BOLTS	SA-540 CL-3	ASME III NF	IMPACT TESTED			41	TENSIONER	7.2	22.5
SLIDING BASE	SA-533 GR.B CL-2	ASME III NF	NONE					4.67	35.0
LATERAL SUPPORT KEYS.	A-336 GR.F5A	AISC	UT, MT					26	-
SLIDING BASE LATERAL KEY ANCH-BOLTS	A-540 GR.B24 CL-1	ASME III NF/ASTM	UT, MT			44	TENSIONER (BOLT ELONG.)	43	-
VERTICAL BEARING SOCKETS	SA-515 GR.70	ASME III NF	MT, PT						
VERTICAL BEARING SLIDES	ASTM-B-22-CU. ALLOY 863	ASME III NF	MT, PT					4.4	8.0
SLIDING BASE EMBDX	A-516-70	AISC	NONE					36	-
VERTICAL HOLD DOWN ANCHOR BOLTS	A-540 GR.B23 CL-3	ASME III NF	UT			0	NONE	∞	∞

Remarks:

SUPPORT SYSTEM INTEGRITY

PLANT ANPP: UNITS #1, 2 & 3 COMPONENT R.C. PUMP SUPPORT SUPPORTS

Item	Material	Dsgn/Fabr Code	Supplementary Mat/Fabr NDE	Welding		Bolting		Design Margin	Actual Seismic Margin
				PWHT	NDE	Pre(% Ult)	Type		
UPPER HORIZ. SUPP. UPPER FLANGE	SA-516 GR. 70	ASME III NF	NONE					6.3	61.0
SPH. BEARINGS	AMS 5649	ASME III NF	NONE					40.1	88.2
PINS	A-540 GR. B23/24, CL-2	ASME III NF	NONE					12.2	26.6
COLUMNS	SA-320 GR. L43	ASME III NF	NONE					11.4	27.1
WALL CLEVIS	A-148-73 TYPE 90/60	ASME III NF	NONE					25.4	62.1
WALL CLEVIS ANCHOR BOLTS	A-540 GR. B22 CL-4	ASME III NF/ASTM	UT			41	TENSIONER (BOLT ELONG.)	8	–
SNUBBER ASSY. (4)	ASTM-A-543-74 CL-2	ASME III NF	NONE					10.1	10.1
SNUBBER ASSY. ANCHOR BOLTS	A-540 GR. B22 CL-4	ASME III NF/ASTM	UT			22	TENSIONER (BOLT ELONG.)	7	–
LOWER HORIZ. SUPP. BOLTS: CONNECTING CASING & SKIRT	SA-540 GR. B23 CL-3	ASME III NF	NONE				TURN OF NUT	8.	74.2
PUMP SKIRT	ASTM-A-148 TYPE 90/60	ASME III NF	NONE					6.3	61.7
SPH. BEARING	AMS-5649	ASME III NF	NONE					23.1	209

Remarks: (4) CONSISTS OF A-668-72; A-508 CL-4a; A-514-74; A-434-64, CL-BD; & THE LIMITING PART IS MOUNTING BRACKET ASTM A-543-74, CL-2.

SUPPORT SYSTEM INTEGRITY

PLANT ANPP: UNITS #1,2&3 COMPONENT R.C. PUMP SUPPORT SUPPORTS

Item	Material	Dsgn/Fabr Code	Supplementary Mat/Fabr NDE	Welding		Bolting		Design Margin	Actual Seismic Margin
				PWHT	NDE	Pre(% Ult)	Type		
PINS	SA-540 GR.B23/24 CL-2	ASME III NF	NONE					8.2	38.4
COLUMNS	SA-533 CL-2	ASME III NF	NONE					12.6	103.2
WALL CLEVIS	A-148-73 TYPE 90/60	ASME III NF	NONE					9.1	74.5
WALL CLEVIS ANCHOR BOLTS	A-540 GR.B22 CL-4	ASME III NF/ASTM	UT			41	TENSIONER (BOLT ELONG)	8	–

Remarks:

SUPPORT SYSTEM INTEGRITY

PLANT ANPP: UNITS # 1, 2 & 3 COMPONENT R.C. PUMP SUPPORT VERTICAL SUPPORTS.

Item	Material	Dsgn/Fabr Code	Supplementary Mat/Fabr NDE	Welding		Bolting		Design Margin	Actual Seismic Margin
				PWHT	NDE	Pre(% Ult)	Type		
UPPER FLANGE WELD TO MOTOR SUPPORT	E-8018	ASME III N.F	NONE	YES	X-RAY MT, PT			6.3	61.0
MOTOR SUPPORT	SA-516 GR. 70	ASME III NF	NONE					7.2	65.7
BOLTS-MOTOR SUPP. TO CASING	SA-540 GR.B24 CL-3	ASME III NB	NONE			22	TENSIONER	6.9	13.0
CASING	SA-508 CL-2	ASME III NB	NONE					8.	18.5
BOLTS: CASING TO PUMP SKIRT	SA-540 GR.B23 CL-3	ASME-III NF	NONE				TURN OF NUT	8.	74.2
PIN-SKIRT TO VERT. COLUMN.	SA-540 GR.B23/24 CL-2	ASME III NF	NONE					5.3	47.7
SPH. BEARING	AMS 5643	ASME III NF	NONE					15.1	146.
VERTICAL COLUMN	SA-533 CL-2	ASME III NF	NONE					7.4	55.3
FLOOR CLEVIS	A-148-73 TYPE 90/60	ASME III NF	NONE					6.0	53.2
FLOOR CLEVIS ANCHOR BOLTS	A-540 GR.B22 CL-4	ASME III NF/ASTM	UT			41	TENSIONER (BOLT ELONG.)	8	-

Remarks:

APPENDIX B

COMBUSTION ENGINEERING QUALITY ASSURANCE PROCEDURES FOR REACTOR COOLANT LOOP PIPING

C-E Power Systems
Combustion Engineering, Inc.
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Tel. 203/688-1911
Telex 99297

POWER
SYSTEMS

March 5, 1984

Mr. Garry S. Holman
Lawrence Livermore Laboratory
P. O. Box 808 L-46
Livermore, CA 94550

Dear Mr. Holman:

This letter addresses the steps Combustion Engineering takes to prevent, or at least minimize, design and construction errors in the Reactor Coolant System Supports. The contents of this letter may be used for any purpose and disseminated to any individual or company.

Three Types of Design errors are possible.

(1) Improper or incorrect analysis resulting in incorrect design loads.

(2) Improper or incorrect analysis of a design resulting in hardware which cannot accommodate the design loads.

(3) Design whch does not address or improperly accounts for the behavior of the system.

With respect to type (1), C-E has developed, checked and re-checked its seismic analysis techniques. C-E has used response spectrum analysis for some plants but has used time history for final analysis of the RCS since 1971. The behavior of the C-E RCS is thoroughly understood and, since almost all C-E plants are 2 loop, the seismic response for each plant can be addressed by comparing it with other plants to determine effects of size, excitation, soil. This has been done during the design process for each succeeding plant.

With respect to type (2), the C-E supports are simple, analyses are standard, chances for error minimal. The C-E QA procedures require independent review of all calculations.

Type (3), errors are addressed below, followed by potential material, fabrication and installation errors.

Combustion Engineering has always held the philosophy of assuming responsibility for the design of RCS supports which affect the capability of the RCS components to freely expand, contract and withstand the excitations imposed by system transients, earthquakes, and posutlated pipe ruptures. Thus C-E designs practically all support members which are not embeded. In addition, C-E calculates and specifies all support interface loads and requirements to the architect engineer. (If it moves or allows motion, C-E designs it). Thus the possibility of support behavior which is incompatible with the RCS has been eliminated.

All plants covered by this study are of the two loop design and, except for one, they all have the same pipe sizes and, essentially, the same configuration. In spite of this, each plant (except duplicate units) is analyzed in its own right. Thus the behavior of the RCS is reconfirmed for each plant; there are obviously changes in component sizes, and weights and these are fully accounted for in the analyses; any unpredicted change in the results is thoroughly investigated to detect potential errors in previous analyses.

The design of C-E supports shuns welds other than attachments to components; thus there are no welds which are not furnace stress relieved and non-destructively examined. The design of C-E supports, including bolting, does not require exotic or ultra high strength materials. In this manner the potential for design errors resulting from unaccounted residual stresses and/or stress corrosion cracking is minimized.

The design of C-E supports includes spherical bearings at all pinned joints. Thus potential for pin overstress or fatiguing as a result of loading in an unaccounted for direction is eliminated.

The requirements for each component of the support system are detailed in a procurement specification. Compliance with specified requirements is thoroughly checked.

The design interfaces between C-E designed RCS supports and the architect engineer designed embedments are defined at the start of each job and each party is kept informed of progress, problems, and necessary changes.

The installation procedures for RCS supports are written or reviewed in detail by Combustion Engineering. Cognizant construction engineers are briefed by Combustion Engineering design engineers on the design of the RCS supports and their interface with embedments.

C-E engineering personnel at the site furnish technical assistance to construction personnel during installation of supports in understanding and implementing procedures and resolving questions.

Combustion Engineering writes and/or reviews test procedures to be performed.

(1) Prior to hydrostatic test to confirm that all supports have been properly installed.

(2) During hot functional tests to confirm that predicted RCS motions.

Combustion Engineering personnel assist site personnel in conducting hydrostatic, precore, and post core hot functional tests. These tests provide confirmation of proper functioning of the Reactor Coolant System and its supports.

Very truly yours,

T. E. Natan
Manager, Plant Structures

TEN/kmv
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PSE-84-020