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Probability of Pipe Failure in the Reactor Coolant Loops of Babcock and Wilcox PWR Plants
Volume 2: Guillotine Break Indirectly Induced by Earthquakes

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Prepared for
U.S. Nuclear Regulatory Commission

Lawrence
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ABSTRACT

The requirements to design nuclear power plants for the effects of an instantaneous double-ended guillotine break (DEGB) of the reactor coolant loop (RCL) piping have led to excessive design costs, interference with 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 exploring 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 estimates the probability of indirect DEGB in RCL piping as a consequence of seismic-induced structural failures within the containment of Babcock & Wilcox supplied pressurized water reactor nuclear power plants in the United States. The median probability of indirect DEGB was estimated to range between $6x10^{-11}$ and $1x10^{-7}$ per year. Using very conservative assumptions, the 90% subjective probability value (confidence) of P_{DEGB} was found to be less than $1x10^{-5}$ per year.

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

EXECUTIVE SUMMARY

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, with the DEGB and the Safe Shutdown Earthquake (SSE) events being considered to occur simultaneously. This requirement has led to excessive design costs (i.e., provision of pipe whip restraints), interference with 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), established to estimate the probability of a DEGB of the 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 a DEGB of the RCL piping. Two broad classes of DEGB induced by earthquakes have been identified. The directly-induced DEGB is defined as a double-ended pipe break of the RCL piping due to fatigue crack growth under the combined effects of thermal, pressure, seismic, and other cyclic loads while the indirectly-induced DEGB is a 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 a DEGB indirectly-induced by structural failures under earthquakes which was developed in a previous phase of the program has been applied to Babcock & Wilcox 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, seismic responses, and capacities are explicitly treated in this methodology to produce subjective probability bounds on the estimated probability of a DEGB. By reviewing the plant arrangement and design bases for both the raised loop and lowered loop B&W reactor configurations, 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 a DEGB of the RCL piping. Fragility descriptions of these supports have been developed using information on plant design criteria and by appro-

priately extrapolating the responses calculated at the design analysis stage to failure levels of the structural elements of the component supports. Fragility is expressed in terms of a factor of safety over the SSE peak ground acceleration. The median factor of safety $\check{F}$ and the variability estimates $\beta_{F,R}$ and $\beta_{F,U}$ have been calculated.

The probability of indirectly-induced DEGB in the RCL piping has been 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 other B&W reactor plants, generic seismic hazard curves developed in a previous phase of this research program were utilized.

The median probability of indirect DEGB was estimated to range between 6×10^{-11} and 1×10^{-7} per year. Using very conservative assumptions, the 90% subjective probability (confidence) value of P_{DEGB} was found to be less than 1×10^{-5} per year.

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 B&W 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

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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 a 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 high design costs (i.e., provision of pipe whip restraints), interference with normal plant operation and added radiation exposure of 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 the RCL piping. The first two phases of the program addressed the issue for Westinghouse (W) and Combustion Engineering (CE) PWR plants. The present phase of the program concentrates on the PWRs supplied by Babcock & Wilcox (B&W). The objective of the program is to recommend changes to 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 1) evaluate the asymmetric blowdown loading, 2) combine SSE and DEGB loads and 3) install and maintain pipe whip restraints for the RCL piping.

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

1.1.1 Previous Studies on Westinghouse and Combustion Engineering Reactors

In the first phase of the Load Combination Program, the probability of an indirectly-induced DEGB in the RCL piping of Westinghouse reactors was evaluated (Ravindra, et al., 1984). 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 an indirect DEGB was evaluated. The median probability of an indirect DEGB was calculated to be 1.3×10^{-8} per year with the 10 and 90 percent subjective probability bounds on this probability being estimated to be 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 the results of 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 accelerations 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, 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 an indirect DEGB was calculated to be 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 an indirect DEGB was calculated 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 an indirect DEGB in the RCL piping due to earthquakes is verv low 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).

In the second phase of this program, a similar but reduced scope evaluation of the probability of an indirect DEGB of the RCL piping was undertaken for Combustion Engineering supplied reactor systems. A total of 13 Combustion Engineering plants were investigated with Palo Verde Units 1, 2, and 3 being used as the reference plant. Six of the plants were characterized as "early" plants (three nozzle support for the reactor vessel) and the remaining seven were characterized as "modern" plants (four nozzle supports for the reactor vessel). The median probability of an indirect DEGB was calculated to be in the range of 10^{-6} per year for older plants and less than 10^{-8} for modern plants. Using very conservative assumptions, the 90% subjective probability (confidence) value an the value of P_{DEGB} was found to be less than 5×10^{-5} per year for the older plants and less than 3×10^{-7} per year for the modern plants.

Consistent with the results of the study of Westinghouse plants, the study of Combustion Engineering reactors showed that the probability of an indirect DEGB in the RCL piping due to earthquakes is very low.

1.1.2 Reactor Coolant Loop Arrangement in B&W Reactors

The primary reactor coolant system in a B&W nuclear steam supply system (NSSS) typically consists of two loops and includes the reactor vessel, two steam generators, four reactor coolant pumps and the pressurizer. Two NSSS configurations have been developed by B&W. For the first, designated the "lowered-loop" configuration, the reactor vessel and steam generators are both skirt supported and are anchored to the base mat at essentially common elevations. The skirt flanges are fixed against translation. Thermal expansion is accomodated by means of the flexibility of the RCL piping and the freedom of movement allowed the reactor coolant pumps which are snubbed. With the exception of Midland, the reactor vessels do not have upper lateral support, however, the lateral displacement of the tall vertical steam generators is limited by an upper bumper support system. The Midland Energy Center Project was chosen as the reference plant for the "lowered-loop" design. Figures 1-1 through 1-6 give details of the RCL arrangement at the Midland plant operated by Consumers Power Company.

For the second configuration, designated as the "raised-loop" configuration, the reactor vessel is supported on nozzle pads fabricated as part of the four cold leg inlet nozzles which are set on lubrite plates allowing radial expansion but essentially restraining lateral and vertical translation. The steam generators are restrained against lateral motion by an upper shear key restraint system and by a lower trunnion support on a snubbed sliding base which allows thermal expansion radial to the axis with the reactor vessel but restrains all other vertical and lateral motion. The reactor coolant pumps are supported by vertical and upper and lower horizontal struts and are snubbed allowing thermal expansion radial to the axis with the reactor vessel. The Washington Public Power Supply System (WPPSS) Unit 1 was selected as the reference "raised-loop" configuration plant.

1.2 GENERAL APPROACH

1.2.1 Objective and Scope

The objective of this present study has been to evaluate the probability of a seismically-induced indirect DEGB in the reactor coolant loop piping of "raised-loop" and "lowered-loop" B&W reactors. The study consisted of the following major tasks:

1. Review available seismic hazard curves for the B&W plant locations to assess the validity of using generic seismic hazard curves in the proposed study and to select the appropriate hazard curves.

2. Review the seismic design bases and the B&W calculated code seismic margins for equipment support failures in B&W plants.

3. Estimate the seismic fragilities of equipment supports whose seismic failure may lead to an indirect DEGB of the reactor coolant loop piping for the two selected reference plants.

4. Calculate the probability of an indirectly induced DEGB in the RCL of these two plants using the relevant seismic hazard curves.

5. Generalize the results obtained in item 4 to the other B&W plants by reviewing the code seismic margins for the equipment (i.e. steam generator, reactor coolant pump and reactor pressure vessel).

1.2.2 Plants Studied

A total of ten plants whose NSSS was designed and fabricated by B&W were evaluated during the course of this study. The plants are separated into two groups on the basis of the reactor coolant loop configuration ("raised" or "lowered") as follows:

Lowered Loop Configuration	Raised Loop Configuration
Midland 1 & 2 (Reference Plant)	WPPSS 1 (Reference Plant)
Oconee 1,2 & 3	Davis - Besse 1
Crystal River 3	
Arkansas Nuclear One 1	
Rancho Seco	

All of the operating plants were placed on line between 1973 and 1977 with the remaining plants either cancelled or not yet operating. For the most part, all of the plants are situated on rock sites, were based upon similar design criteria, and employed similar methods of analysis.

1.3 OUTLINE OF THE REPORT

The technical approach employed in this study is described in Chapter 2. A general methodology for estimating the probability of a seismically-induced indirect DEGB in the RCL piping is outlined. The major elements of this methodology are seismic hazard analysis, seismic fragility evaluation and assessment of the consequences of structural failures within the containment on the RCL piping. The plant design information provided by B&W 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 an indirect DEGB in the Midland and WPPSS Unit 1 RCL piping are described.

The results of this study are presented and discussed 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 also discussed. The chapter ends with a summary of the study and significant conclusions.

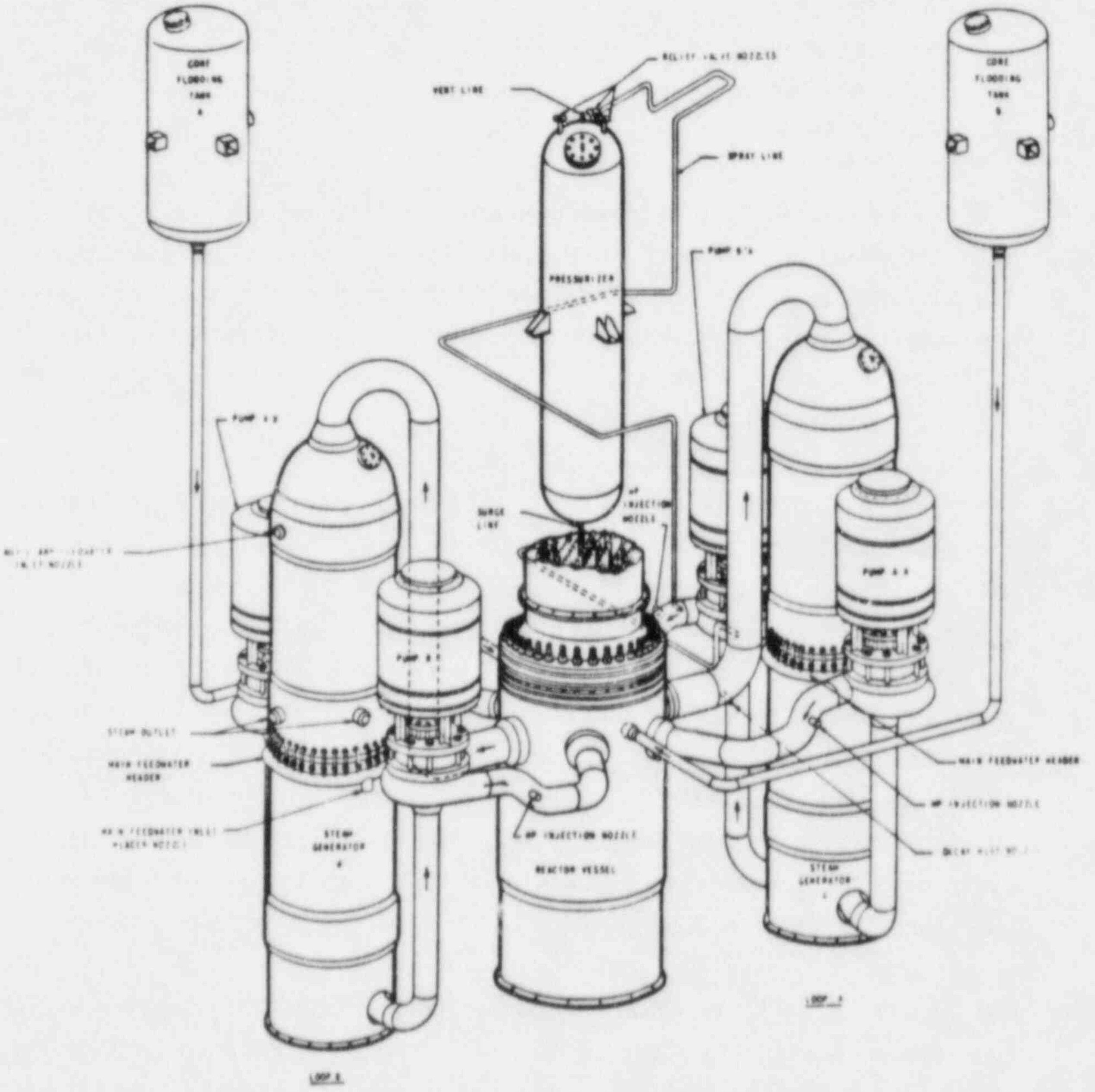

FIGURE 1-1. MIDLAND REACTOR COOLANT SYSTEM ISOMETRIC

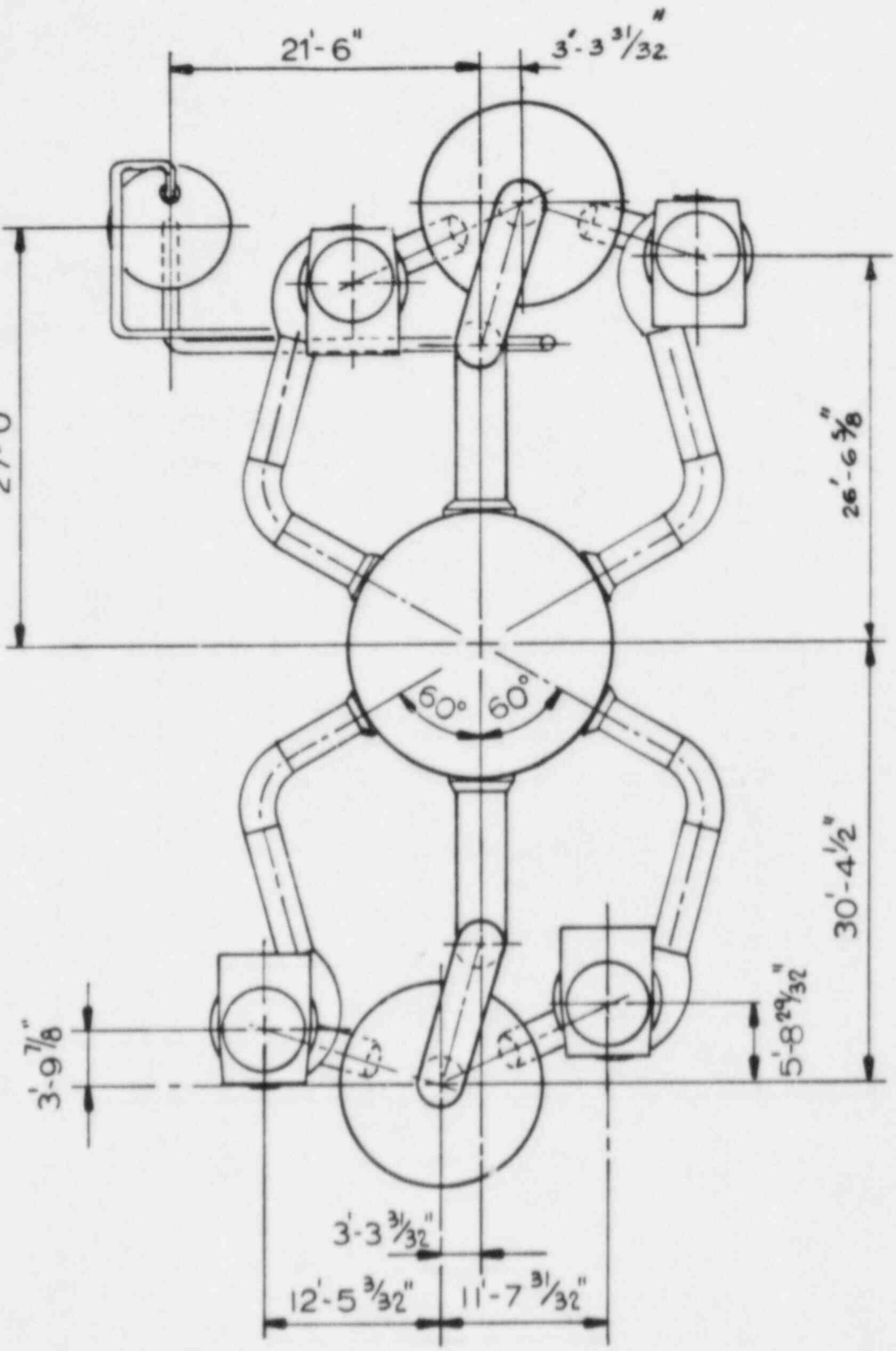

FIGURE 1-2. MIDLAND REACTOR COOLANT SYSTEM PLAN VIEW

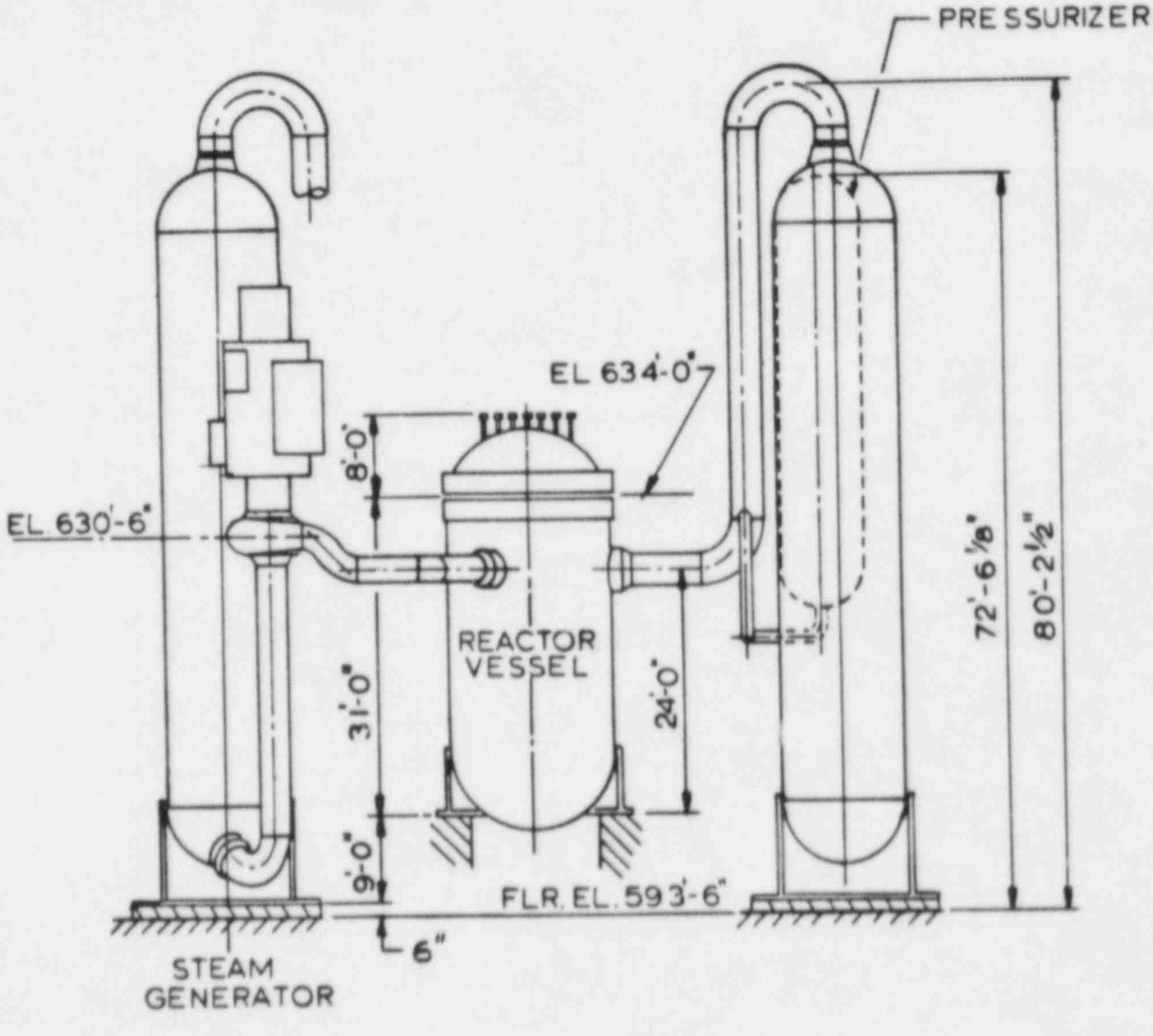

FIGURE 1-3. MIDLAND REACTOR COOLANT SYSTEM ELEVATIONS

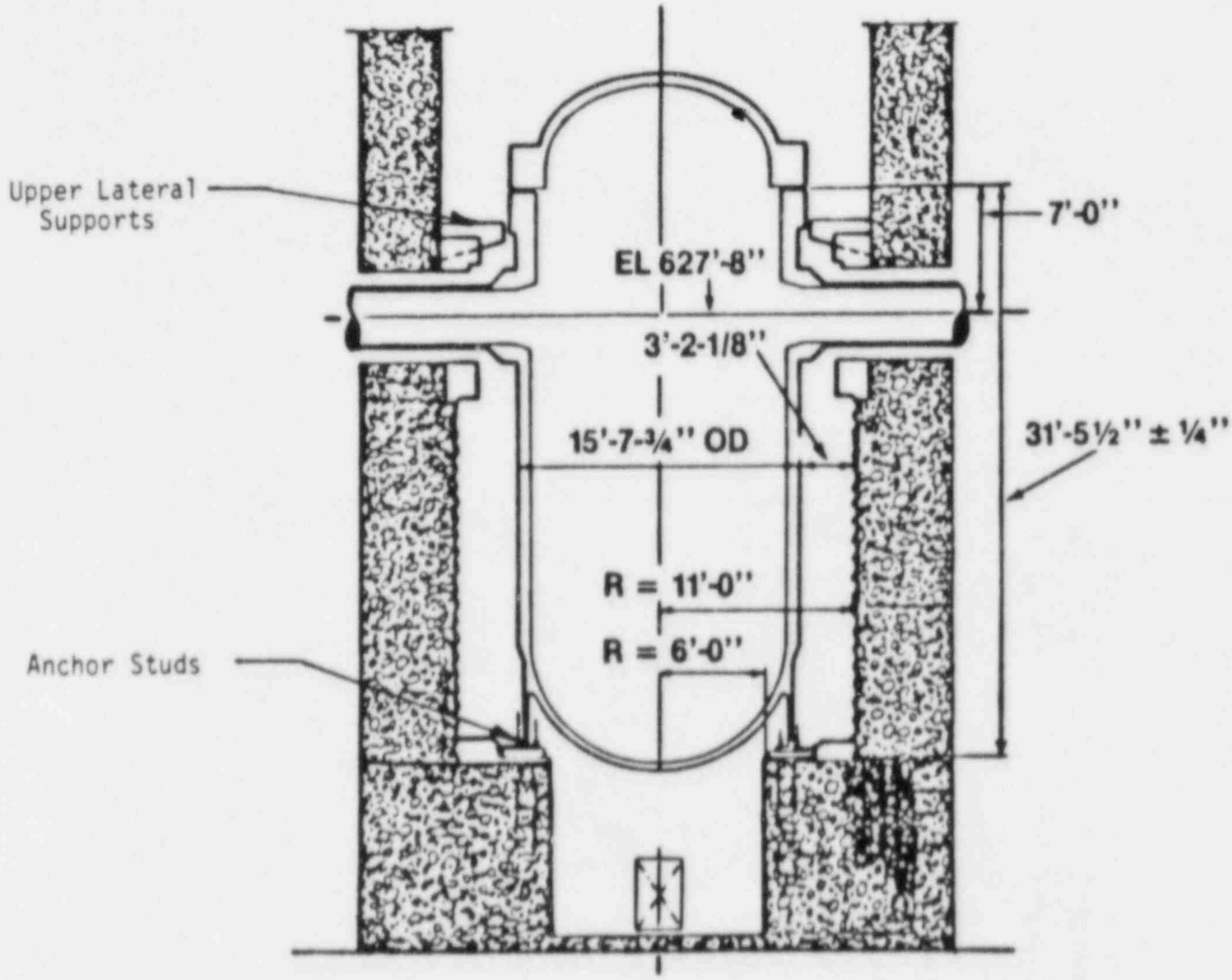

FIGURE 1-4. REACTOR PRESSURE VESSEL SUPPORT SYSTEM

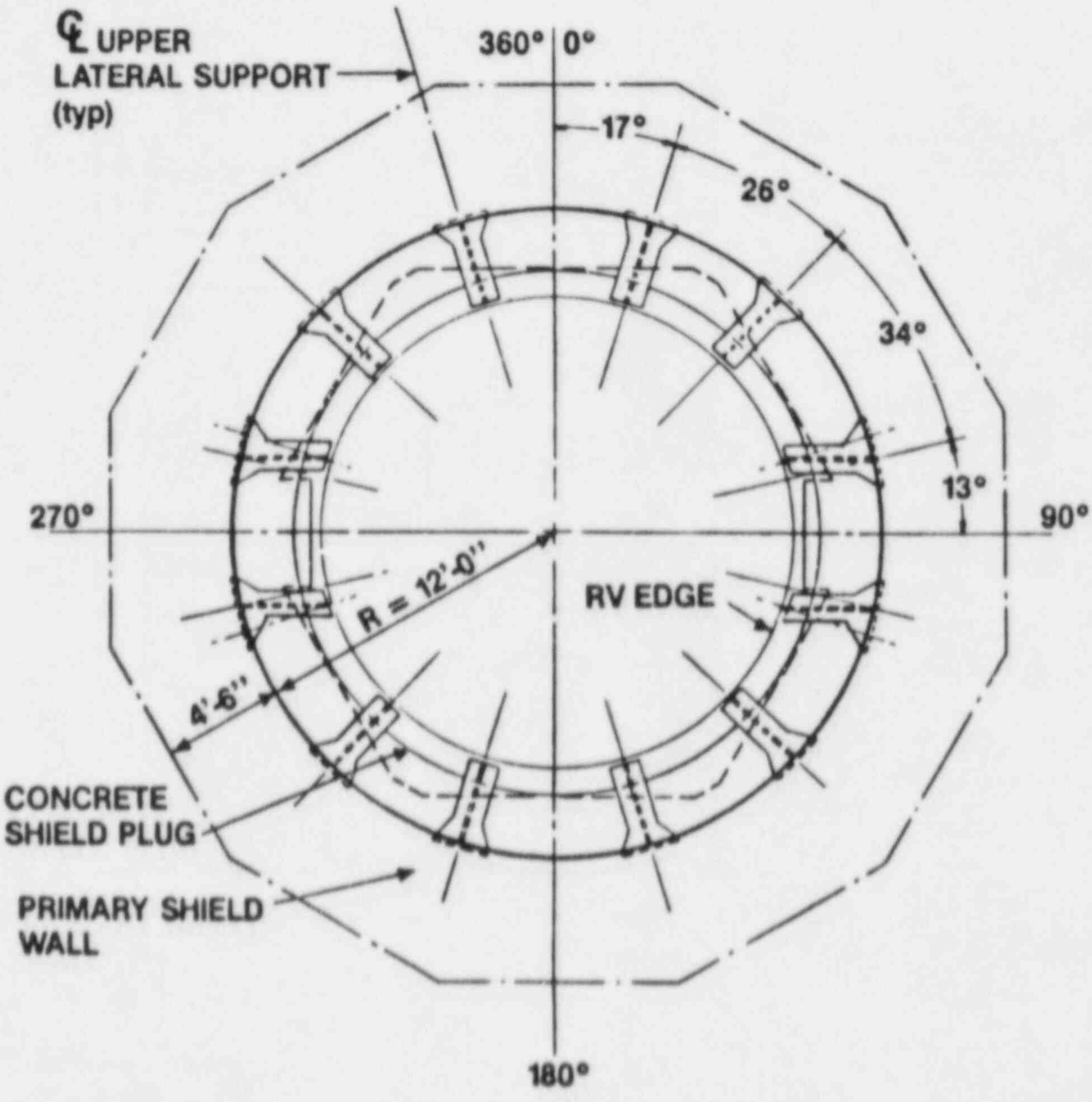

FIGURE 1-5. RPV UPPER LATERAL SUPPORT ARRANGEMENT

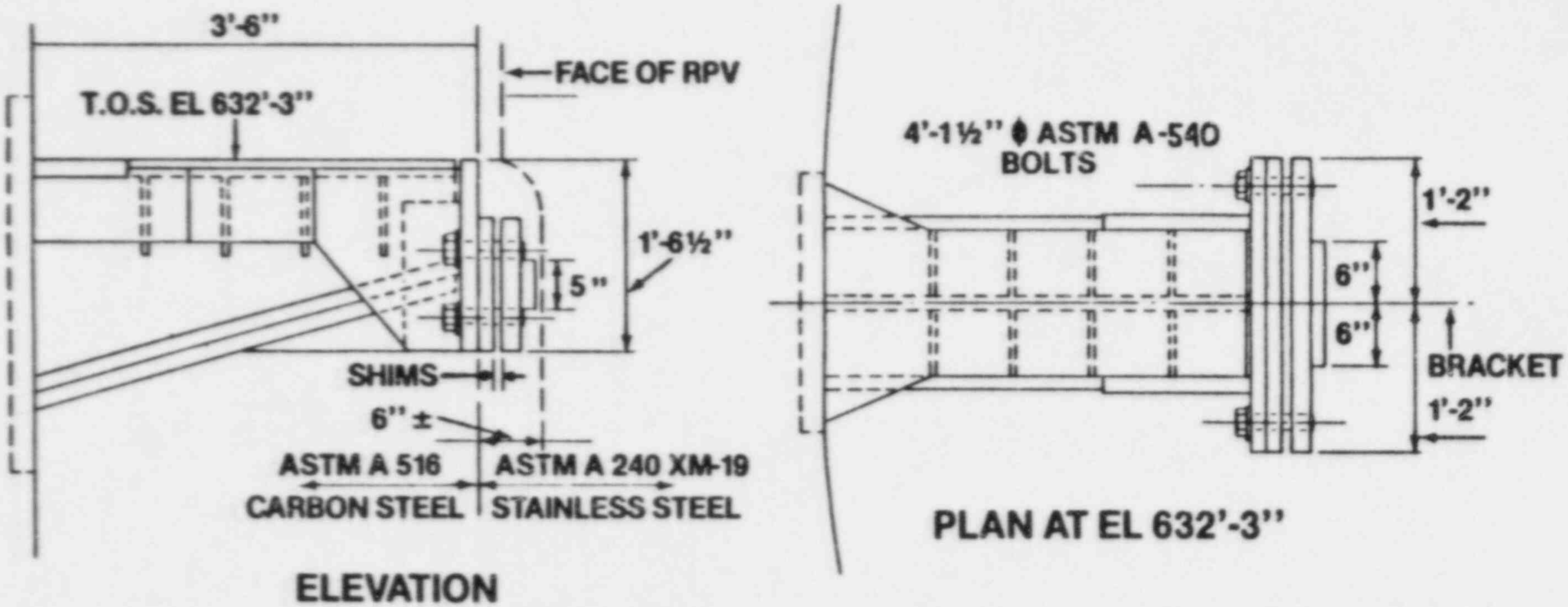

FIGURE 1-6. RPV UPPER LATERAL SUPPORT BRACKET DETAIL

CHAPTER 2

TECHNICAL APPROACH

In this chapter, the analytical approach pursued in the calculation of the probability of an indirect DEGB induced by structural failures under earthquakes is described. 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 the RCL piping.

2.1 METHODOLOGY

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

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

where

C_i = Capacity of structural element i (e.g., reactor pressure vessel skirt, steam generator support snubber, or 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 a peak ground accelearation between a and a+da at the site.

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 on the probability of an indirectly-induced DEGB is 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 a 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 a DEGB of the RCL piping. Therefore, the focus of this study is only on those structural elements within the containment whose failure can result in a DEGB. Among these, some elements may have large margins of safety against seismic failure and thus may not contribute significantly to the probability of a DEGB. Therefore, critical elements are defined as those whose failure could contribute significantly to the probability of an 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 a DEGB is evaluated by treating the failure events of individual structural elements as statistically independent and is derived from the conditional probabilities of failure of these structural elements. This gives a conservative upper bound on the probability of a 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 concerning the 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:

$$A_c = F \cdot A_{SSE} \quad (2\text{-}2)$$

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 the 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 the SSE. Since C_i and R_i are random variable, 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 $\check{F}$ 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. 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):

$$F = F_C \cdot F_{RS} \cdot F_{RE} \qquad (2\text{-}3)$$

The capacity factor, F_C, for the equipment support is a product of a strength factor, F_S, and an inelastic energy abosrption 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_S, the non-seismic portion of the total load acting on the support is subtracted from the strength as follows:

$$F_S = \frac{S-P_N}{P_T-P_N} \qquad (2\text{-}4)$$

where S is the ultimate structural strength for the specific failure mode, P_N is the stress due to the normal operating load (i.e., dead load, operating temperature load, etc.) and P_T is the stress resulting from 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 stress due to 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 exhibit little or no system inelastic energy abosrption 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 or bending

The strength of the element failing in a ductile mode is taken to be the yield strength of the material for tensile loading while for flexural loading, the strength is defined as the stress at which a plastic hinge is developed.

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 supports 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:

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

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

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

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 structural 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 response calculated in the design analyses for a given peak ground acceleration level.

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

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

where

F_{SA} = Spectral shape factor representing the variability in ground motion defined by the median site-specific ground response spectra and the ground spectra used for design.

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 of embeddment of the structure.

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 Equations 2-8 and the properties of lognormal probability law:

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

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

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

Similarly, the equipment response factor, F_{RE}, is a measure of the conservatism inherent in the calculation of equipment response in the design analyses. F_{RE} is equal to the ratio of the equipment response calculated during design to the realistic equipment response level based upon the appropriate design floor response spectra. The equipment response factor is also expressed as a product of the factors influencing the variability on response.

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

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 conservatism in method used to combine modal responses.

F_{EC} = Factor to account for conservatism in method used to combine earthquake components.

The median $\check{F}$ and the variability estimes, 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.

The overall factor of safety F is calculated as the product of the capacity factor and the equipment and structural response factors. Using Equation 2-2, the ground acceleration capacity of the structural element then becomes:

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

where

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

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

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

The overall factor of safety is thus decomposed into factors that can be modeled and for which data and information exist. 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 individuals 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 the support for each major equipment component has been expressed in this study as the lowest capacity for all credible failure modes of the component support. This is a realistic assumption since the failure modes are highly correlated due to common structural material and method of fabrication. Again, if one of the failure modes of the structural element 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 has been 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 earthquake (Figure 2-3).

2.2 DESIGN INFORMATION PROVIDED BY BABCOCK & WILCOX

For this study, Babcock & Wilcox (B&W) provided information on the design bases and features of the reactor coolant loop and primary equipment supports for the nuclear power plants with B&W reactors listed in Section 1.2.2. As explained above, the reactors were grouped into two categories based upon the loop configuration and the design information was obtained for both.

2.2.1 Information on Seismic Hazard

A site-specific seismic hazard study was previously conducted for the Midland plant and the results were used to define the Midland seismic hazard in this study. Similarly, a seismic hazard study was previously conducted for the region of south-eastern Washington in which the WPPSS-1 nuclear facility is located and, although not site-specific, was judged to be adequate to define the seismic hazard for the plant. Site-specific seismic hazard curves were not available for any of the remaining plants and for these same generic curves used in the earlier studies of Westinghouse and Combustion Engineering plants were utilized.

2.2.2 RCL Equipment Support Details

B&W furnished a description of the reactor coolant loop for each of the selects plants describing details of certain of the primary equipment supports. These were reviewed during this project together with engineering drawings for the reference plants to identify critical elements in the equipment supports and to assess the effects of their seismic failures on the RCL piping.

2.2.3 Information on Seismic Response

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

Structural Response

- Ground spectrum used for design
- Structural damping
- Site characteristics (rock site or soil site)
- 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 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 B&W for the Washington Public Power Supply System (WPPSS) - Unit 1 Plant.

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

The B&W reactor sites are dispersed throughout the United States. Ideally, the site-specific seismic hazard curves should be used 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 have been 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, 1984) were utilized in this study. For the purpose of completeness, a brief description of the development of 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 siesmic hazard analyses have been performed (Figure 2-4). Some of these analyses have been published (e.g., Zion and Indian Point Seismic Hazard Analyses), while others are part of PRA studies yet to be published. In order to preserve the anonimity 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 earthquake) 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 being 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 0.10 to 0.25 peak ground acceleration. Hence, the seismic hazard curves must 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 A_{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 had not been 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 01, respectively. This was done by developing a subjective probability distribution of the probability of exceedence at each specified value of X (i.e., A divided by the larger of A_{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 exceedance 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% exceedance subjective probabilities. At a value of X=1, (i.e., at peak ground acceleration equal to A_{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 Midland Plant

A site-specific seismic hazard analysis was performed for the Midland Nuclear Plant by Weston Geophysical (1982). A summary of the analysis techniques and results from that study are presented here.

The seismic hazard analysis requires that some definition be given to the geometry of seismogenic zones from which earthquakes could originate and affect the site, the largest earthquake magnitude in these zones, the recurrence frequency of earthquakes of various sizes, and the ground motion attenuation model.

Three models of seismogenic zones were hypothesized to characterize the uncertainty in the tectonic nature of local geology. These models have been defined as Model 1, the Michigan Basin-Cincinnati Arch; Model 2, the Central Stable Region with seismic activity constrained to geologic features at Anna, Ohio, and Attica, New York; and Model 3, the Central Stable Region without these constraints. All models include the effects of distant activity sources to the extent they could additionally affect the site. Based on the geology of the region and the statistical distribution of historical earthquakes, the weights assigned in Models 1,2, and 3 were 0.5, 0.3, and 0.2, respectively, reflecting that Model 1 is the most credible of the three.

In the absence of instrumentation, most historical earthquakes have been reported in terms of the level of human perception or structural damage they caused. The damage information was subsequently reported in terms of the Modified Mercalli Intensity (MMI), I_o. More recently, with instrumentation, earthquakes have also been expressed in terms of instrumentally measured body wave magnitude, m_b. In the current study, the historical intensity values were converted to m_b using the relationship of Nuttli and Herrmann 1978.

$$m_b = 0.5\ (I_o + 3.5) \qquad (2\text{-}16)$$

Then, the annual number, n, of earthquakes equal to or greater than earthquakes of body wave magnitude, m_b, was determined from the expression

$$\log_{10} n(m_b) = a - bm_b \qquad (2\text{-}17)$$

where a and b are parameters fit to available earthquake activity level and rate data, respectively.

The maximum earthquake magnitude assigned to each seismogenic zone model is seen in Table 2-1. Note that for Models 1 and 2, the maximum magnitude at the site (Michigan Basin or Central Province background) was estimated to be 5.3, and for Model 3, 6.0. The latter is extremely conservative considering the infrequent low level seismic activity in the Michigan Basin (m_b = 4.7 is the largest observed event and that was about 180 to 190 km from the site). Some variability in this parameter was considered. The maximum earthquake magnitude for Models 1 and 2 of 5.3 and for Model 3, 6.0, roughly corresponds to maximum site intensities of VII and VIII, respectively. A 95% probability was assigned to the occurrence of all earthquakes with intensities up to these maximums. An upper bound hypothesis of site intensity was made for each model by presuming extremely poor construction on similarly poor foundation material. This increase was assigned one standard deviation, which corresponds to an increase in the maximum site intensity to VIII for Models 1 and 2, and to IX, for Model 3. A 5% probability was assigned to this maximum intensity hypothesis.

A generalized ground motion attenuation model was used in the seismic hazard analysis. Its applicability was checked using the attenuation characteristics of several specific events located in the central U.S. region and is as follows:

$$I_{mm} = 2.53 + 1.20m_b - .0027R - 1.84 \log R \qquad (2\text{-}18)$$

where

R = the distance in kilometers from the earthquake epicenter.

The median sustained ground acceleration, a_s, at the site is obtained by conversion of the site I_{mm} using the relationship

$$\log a_s = 0.326 + 0.214 \, I_{mm} \qquad (2\text{-}19)$$

The resultant annual frequency of exceedance of various sustained ground accelerations is presented in Figure 2-9. Four sets of curves are comprised of three curves each, with each of the three corresponding to the three seismogenic zone models. To obtain the four sets, a lognormal distribution of the possible sustained ground accelerations for given annual exceedance frequencies was assumed, with the second set of curves from the left representing the median. Additional sets were defined at standard deviations of minus 1 (left-most set), plus 1 (third set), and plus 2.5 (right-most set). Preserving the assigned weights to each seismogenic zone model in each of the four sets of curves, a probability was assigned to each curve in Figure 2-9, normalized as shown such that their total probability is equal to 1.0.

2.3.3 WPPSS Seismic Hazard Curves

For the present study, no site-specific seismic hazard curves were available for WPPSS site. Therefore, the seismic hazard maps published by Algermissen, et al (1982) have been utilized to develop the seismic hazard curves for the WPPSS site. The middle curve in Figure 2-10 is based on the peak ground acceleration values reported by Algermissen, et al (1982) corresponding to annual probabilities of exceedence of $1x10^{-2}$, $2x10^{-3}$ and $4x10^{-4}$. This curve is considered to be the best estimate of the seismic hazard at the site. The work of Algermissen, et al (1982) did not consistently treat uncertainty in the 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 INDIRECT DEGB PROBABILITY CALCULATION FOR MIDLAND

In this section, the procedure for calculating the probability of indirect DEGB is illustrated using the Midland lowered-loop configuration reference plant as an example.

2.4.1 Support Arrangement

The Midland NSSS consists of two loops which include the reactor vessel, two steam generators, four reactor coolant pumps and the pressurizer.

The reactor vessel has lower support provided by a skirt bolted into the concrete pedestal and upper lateral support provided by bumpers bolted to the primary shield wall. A gap of approximately 0.12 inches exists between the reactor vessel and the bumpers under normal operating conditions. Dynamic analysis of the NSSS shows that earthquakes larger than SSE must occur for the reactor vessel to impact the bumpers. The steam generator has a similar support system, lower support being provided by a skirt and upper support by bumbers which are assumed to have negligible gap. The upper support bumpers of the steam generator are welded to girders which span the concrete D-ring. For the reactor coolant pump, vertical dead weight support is provided by spring hangers and lateral and vertical seismic support is provided by sets of upper and lower snubbers. The snubbers are bolted to the concrete D-ring.

It is assumed that seismic failure of the supports of either the reactor vessel, steam generator, or reactor coolant pump would unconditionally result in a DEGB of the NSSS piping. It is also assumed that failure of both steam generators is perfectly dependent and failure of all four coolant pumps is perfectly dependent. Thus, only one steam generator and one reactor coolant pump with the lowest capacity are considered in the fragility development. This is a realistic assumption because the supports are all similar and responses are correlated. Additionally, the various possible failure modes for a component are assumed dependent, thus, the capacity of each component is governed by its lowest capacity failure mode.

2.4.2 Capacity Factors

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

2.4.2.1 Reactor Vessel

Failure of the RPV is assumed to occur when either the upper bumpers or the skirt fails. From the seismic margin review performed by Structural Mechanics Associates (SMA, 1983), an earthquake of 3.54 times the Seismic Margin Earthquake (SME) is required for the RPV to impact the upper bumpers. Also, the moment in the skirt at 3.54xSME is less than one-half of its design moment. For earthquakes above the SSE, the increase of the moment in the skirt will be small relative to that which exists at this level since the upper bumpers prevent additional significant rotation of the RPV about its base. Upon yielding of the upper bumpers, it is assumed that local plastic buckling will occur, thus, the skirt must resist all additional overturning moment. At this point, displacements will increase significantly and failure is assumed to occur. Thus, the RPV capacity evaluation is based on yielding of the upper bumpers.

As previously noted, the SME dynamic analysis shows that the gap between the RPV and the upper support bumpers will not close during the SME. There are, therefore, no stresses in the bumpers at the SME level. After the gap has closed, it is assumed that moment in the skirt will remain relatively constant. This is reasonable, since the RPV is essentially rigid and the bumpers restrict additional rotation about the base. Hence, it is possible to calculate loads in the bumpers by setting the condition that the moment about the base, after the gap is closed, remains constant.

If the bumpers were not present, the moment in the skirt for an earthquake with a ZPA of 1g would be by extrapolation

$$\frac{M_{SME}}{A_{SME}} \times g$$

where M_{SME} is the moment in the skirt at the SME level and A_{SME} is the ZPA of the SME earthquake. Since the moment is assumed to remain constant at $3.54xM_{SME}$ (the moment in the skirt at the point of gap closure), the difference between the extrapolated moment and this moment must be in equilibrium with the moment induced by the upper support reaction, F_u, i.e.,

$$\frac{M_{SME}}{A_{SME}} \times g - 3.54 \times M_{SME} = F_u \ell$$

where ℓ is the distance from the upper support bumpers to the skirt base.

As discussed earlier, failure occurs when the upper bumpers yield. By simplifying the above expression, we obtain:

$$g_{cap} = \frac{F_y \times \ell + 3.54 M_{SME}}{M_{SME}} \times A_{SME}$$

indicating that the strength factor, F_S, is

$$F_S = \frac{F_y \times \ell}{M_{SME}} + 3.54$$

where F_y is the yield load of the upper bumpers.

It should be noted that this methodology neglects the change in frequency of the RPV system which occurs at gap closure since it is based on an extrapolation of the base moment prior to closure of the gap. The methodology is, therefore, conservative since the frequency will move into the rigid portion of the spectra after gap closure. Additionally, the assumption of DEGB occurrence upon yielding of the upper bumpers is conservative; however, the seismic capacity of the RPV supports was

found to be high enough so that RPV support failure does not dominate the DEGB probability.

The bumper material is A516 Grade 70 which has a yield strength, f_y, of 38.0 ksi, and is fabricated with 5/16" fillet welds. The weld material is unknown and is assumed to be E70.

In order for the failure condition to be reached, the bumpers must yield such that rotation about the skirt can increase appreciably. Due to the close spacing of the bumpers, at least three bumpers must yield in order to reach the postulated failure state. F_y for the bumper system was calculated to be 6742 kips with a β_U of 0.15. The distance from the bumpers to the base is 24.75 ft and the moment at the skirt base at the SME level is 10,262 ft-k. β_U on the 24.75 ft length is negligible and β on M_{SME} is considered in the response factor evaluation. β_U on the 3.54 term in the F_S equation (the factor on the SME level required for the gap to close) is evaluated at 0.39, and β_U on the failure mode uncertainty and the uncertainty in the methodology used to predict the failure level is taken as 0.25. The median strength factor is, therefore, calculated as:

$$\check{F}_S = \frac{6742 \times 24.75}{10,262} + 3.54 = 19.80$$

From the second moment method, the variability in the strength factor is calculated as:

$$\beta_{S,R} = 0.00$$

$$\beta_{S,U} = \left[\left\{(16.26^2 \times 0.15^2 + 3.54^2 \times 0.39^2)^{1/2}/19.80\right\}^2 + 0.25^2\right]^{1/2}$$
$$= 0.29$$

A final check on the shear stresses in the skirt and the anchor bolt load at the point at which the upper bumpers fail indicates that the skirt maintains its structural integrity and that the yielding of the bumpers is in fact, the controlling failure mode.

Since failure has been taken to occur upon yielding of the upper bumpers, as discussed, no credit for ductility is taken. Thus,

$$\check{F}_C = \check{F}_S = 19.80$$

$$\beta_{C,R} = \beta_{S,R} = 0.00$$

$$\beta_{C,U} = \beta_{S,U} = 0.29$$

2.4.2.2 Steam Generator

Derivation of F_S for the OTSG is similar to that for the RPV, however, the upper support bumpers for this case are continuously in contact with the steam generator.

The skirt material is SA-533 Grade B, Class 1. At 200°F, f_y = 47.1 ksi. Based on stresses provided by Babcock & Wilcox (B&W, 1983), it is calculated that the skirt will yield at 19.5 times the SME, with a β_U on this factor of 0.21. It is also reported that the largest load in a single bumper at the SME is 311 kips, and it was calculated that the yield capacity of a bumper is 10,900 kips. Thus, it is clear that the bumpers can take additional load after the skirt has yielded.

Noting that the skirt will not take additional moment after it has yielded and using the methodology discussed in Section 2.4.2.1, the capacity of the steam generator may be evaluated from:

$$\frac{M_{SME}}{A_{SME}} \times g - M_y = \left(F_y - \frac{F_{SME}}{A_{SME}} \times g\right) \times \ell$$

where M_y is the skirt yield moment (equal to $19.5 \times M_{SME}$), F_{SME} is the upper support reaction due to the SME and the other terms are as defined in Section 2.4.2.1. Simplifying this expression gives:

$$F_S = \frac{19.5 \times M_{SME} + F_y \times \ell}{M_{SME} + F_{SME} \times \ell}$$

Since there are only four bumpers surrounding the steam generator, it is assumed that contact is made with a single bumper and that failure occurs when that single bumper has yielded. Again, as in the case of the RPV, yield is taken as the failure level for the bumper, since local plastic buckling is assumed to occur at this point.

From B&W's calculated seismic margin earthquake response, M_{SME} = 3650 ft-k and F_{SME} = 311 kips. The length, ℓ, is 64.25 ft and F_y is calculated to be 10,900 kips with $\beta_u = 0.14$. β on the skirt yield moment, which is based on an extrapolation of the skirt stresses reported by B&W for the DBE, is evaluated as 0.21. The values of β on the other terms in the F_S equation are accounted for by the response factors and a β_U of 0.25 is taken for the failure mode and modeling uncertainties. Thus,

$$\check{F}_S = \frac{19.5 \times 3650 + 10,900 \times 64.25}{3650 + 311 \times 64.25} = 32.6$$

From the second moment method,

$$\beta_{S,R} = 0.00$$

$$\beta_{S,U} = \sqrt{\left[\frac{\{(19.5 \times 3650)^2 \times 0.21^2 + (10,900 \times 64.25)^2 \times 0.14^2\}^{1/2}}{(19.5 \times 3650 + 10,900 \times 64.25)}\right]^2 + .25^2}$$

$$= 0.28$$

A final check on the shear stresses in the skirt and the anchor bolt loads at the point at which the upper bumper yields indicates that the skirt maintains its structural integrity and that the bumper fails before the skirt, as was assumed.

As discussed earlier, failure has been taken to occur at the point at which the bumper yields; therefore, no credit is taken for ductility. Thus,

$$\check{F}_C = \check{F}_S = 32.6$$

$$\beta_{C,R} = \beta_{S,R} = 0.00$$

$$\beta_{C,U} = \beta_{S,U} = 0.28$$

2.4.2.3 Reactor Coolant Pump

As discussed in Section 2.4.1, failure of the four reactor coolant pumps and all snubbers are assumed to be perfectly dependent. Since failure of a single set of snubbers on a particular RCP will result in instability of the RCP, the snubber with the lowest capacity will govern the capacity of the RCPs.

From B&W's calculated seismic margin earthquake response, the most critical snubber set has an applied SME load of 357 kips. The design load for this set is 2000 kips, which is based on an allowable stress of 0.9 S_y. Failure of the snubber is assumed to occur in the clevis which is an SA-487 casting. For this material, S_u/S_y = 1.5 and taking $\check{S}_U$ = 1.20 x S_u(code) with a β_u = 0.11 gives:

$$\check{P}_C = (2000/0.9) \times 1.5 \times 1.2 = 4000 \text{ kips}$$

$$\beta_U = 0.11$$

Thus,

$$\check{F}_S = \frac{4000}{357} = 11.2$$

$$\beta_{S,R} = 0.00$$

$$\beta_{S,U} = 0.11$$

It should be noted that the snubber-to-concrete connections are designed to similar or more restrictive criteria than the snubbers and therefore would show similar or greater strength factors.

The clevis capacity has been based upon ultimate strength as there is insignificant system ductility, thus,

$$\check{F}_C = \check{F}_S = 11.2$$

$$\beta_{C,R} = \beta_{S,R} = 0.00$$

$$\beta_{C,U} = \beta_{S,U} = 0.11$$

2.4.3 Structure Response Factor, F_{RS}

The NSSS component support loads used in the capacity factor evaluation were developed by Babcock & Wilcox (B&W) using base mat spectra generated by Structural Mechanics Associates (SMA) for the seismic margin earthquake. Thus, the structure response factor must address any conservatism/unconservatism and variability associated with developing these spectra.

As discussed earlier, the structure response factor, F_{RS}, is modeled as a product of several factors.

$$F_{RS} = F_{SA} \cdot F_{\delta} \cdot F_M \cdot F_{SD} \cdot F_{SS}$$

The evaluation of each of these factors is discussed in the following sections.

2.4.3.1 Spectral Shape Factor, F_{SA}

The free-field ground spectra used by SMA in developing the reactor building base mat spectra were site-specific spectra developed by Weston Geophysical (1981). The spectra shape is thus assumed to be median-centered and $\check{F}_{SA} = 1.0$ is used.

Possible variations in the spectral shape were not provided by Weston Geophysical; hence, random variability, $\beta_{SA,R}$ was estimated using statistical spectra for alluvium from WASH 1255 (Newmark, 1973). This is slightly conservative since the WASH 1255 +1σ spectra includes the effects of soil variability.

From the SMA seismic margin review, the dominant soil-structure modes (median case) are at approximately 2 Hz with 6% composite model damping and 62% of the mass participating, and at 5 Hz with 22% damping and 35% of the mass participating. For Mode 1, since composite modal damping is 6%, it is clear that the damping is primarily structural and it is assumed that median damping would be 10% since the structure is of reinforced concrete. From the WASH 1255 alluvium spectrum, then, at 2 Hz and 10% damping,

$$\beta_{SA,R_1} = \ln\left(\frac{2.01}{1.65}\right) = 0.20$$

For Mode 2, composite modal damping is 22%. At this level of damping, the spectra show very little amplification and spectral shape variation is minor. Thus, we can take $\beta_{SA,R_2} = 0$. Since Mode 1 is expected to be dominant for the base mat response, we take:

$$\beta_{SA,R} = \beta_{SA,R_1} = 0.20$$

This is conservative since $\beta_{SA,R_2} = 0$.

Since a site-specific spectra was used for the response analysis, $\beta_{SA,U}$ is taken as one-third $\beta_{SA,R}$. Therefore,

$$\beta_{SA,U} = \frac{1}{3} \times 0.20 = 0.07$$

2.4.3.2 Damping Factor, F_δ

The base mat spectra developed by SMA were obtained using 7% structural damping and median soil dampings. Hence, there is a conservatism in the contribution of the first mode response (which is governed by structural damping) to the base mat response. For the sake of simplicity, it is assumed that Mode 1 is dominant.

Again, 10% damping is taken as median for Mode 1, 7% is taken as -1β, and 6% composite modal damping was the actual resulting damping in the structural model. From the ground spectra, S_a (2 Hz, 10%) = 0.12g, S_a (2 Hz, 7%) = 0.136g and S_a (2 Hz, 6%) = 0.130g. Thus,

$$\check{F}_\delta = \frac{0.13}{0.12} = 1.08$$

and

$$\beta_{\delta,U} = \ln\left(\frac{0.136}{0.120}\right) = 0.13$$

and $\beta_{\delta,R}$ is taken as $0.2 x \beta_{\delta,U}$, giving,

$$\beta_{\delta,R} = 0.2 \times 0.13 = 0.03$$

2.4.3.3 Modeling Factor, F_M

A state-of-the-art soil-structure analysis was performed, thus, the modeling is assumed to be median-centered and $\check{F}_M = 1.0$. The system is highly complex however, warranting a $\beta_{M,U}$ of 0.25. $\beta_{M,R}$ is taken as zero.

2.4.3.4 Soil-Structure Interaction, F_{SSI}

As discussed earlier, the soil-structure interaction effects are considered by using two factors, F_{SS} and F_{SD}. Since soil-structure interaction was considered in the evaluation of the base mat spectra, $\check{F}_{SS}$ = 1.0. The variability due to the uncertainties in the soil-structure interaction methodology are considered in the equipment response factor.

The reactor building is embedded approximately 20 feet; thus, some correction is required since the analysis was performed using the ground surface spectra. For the Oyster Creek nuclear power plant (Kennedy, et al., 1980) an embedment of 50 feet gave rise to a 15% reduction in maximum ground acceleration. Based on those results, it is assumed that the 20 foot embedment gives rise to a 5% reduction, with zero reduction in maximum acceleration being the upper bound.

Therefore,

$$\check{F}_{SD} = 1.05$$

and taking ground surface acceleration as a 95 percentile upper bound;

$$\beta_{SD,U} = \frac{1}{1.65} \ln (1.05) = 0.03$$

$$\beta_{SD,R} = 0.00$$

The median and β's for the structure response factor are calculated as,

$$\check{F}_{RS} = 1.0 \times 1.08 \times 1.05 \times 1.0 = 1.13$$

$$\beta_{RS,R} = (0.20^2 + 0.03^2)^{1/2} = 0.20$$

$$\beta_{RS,U} = (0.07^2 + 0.13^2 + 0.25^2 + 0.03^2)^{1/2} = 0.29$$

2.4.4 Equipment Response Factor, F_{RE}

The equipment response factor must address any conservatism/unconservatism and variabilities associated with the approaches used to evaluate the support loads from the base mat spectra. The equipment response factor is modeled as the product of several factors.

$$F_{RE} = F_{SA} \cdot F_{\delta} \cdot F_{M} \cdot F_{MC} \cdot F_{EC}$$

The evaluation of these factors is discussed in the following sections.

2.4.4.1 Spectral Shape Factor, F_{SA}

Three base mat spectra were developed by SMA; a lower bound, a median, and an upper bound case corresponded to uncertainties in the soil characteristics and soil behavior. For the support loads evaluation, B&W used a peak-broadened and smoothed envelope of these three spectra as input to their NSSS loop model, thus, there is conservatism in these results. The fundamental frequency of the NSSS is approximately 9 Hz. Median damping is taken to be 5% of critical, thus, the spectral shape factor is evaluated as the ratio of the spectral acceleration at 9 Hz for the 5% damped envelope spectra to the spectral acceleration at 9 Hz for the 5% damped median soil case spectra.

Both rotational and translational spectra are used by B&W for the NSSS analysis. Inspection of the spectra and consideration of the height of the components indicates that the rocking and horizontal translation spectra dominate and contribute approximately equally to the component response. Thus, the spectral shape factor is taken as the average of the spectral shape factors obtained from these two spectra. From the 5% damped horizontal spectra at 9 Hz, S_a(envelope) = 0.34g and S_a(unbroadened, unsmoothed) = 0.26g indicating F_{SA} = 0.34/0.26 = 1.31. Similarly, from the rocking spectra S_a(envelope) = 0.0032g/ft and S_a(unbroadened, unsmoothed) = 0.0032g/ft indicating F_{SA} = 1.0. Thus,

$$\check{F}_{SA} = \frac{1}{2} \times (1.31 + 1.00) = 1.15$$

There is uncertainty in this evaluation since the relative contribution to the response for the rotational input with respect to the translational input is unknown. The value for this uncertainty may be evaluated by taking F_{SA} equal to unity as a -1.65β lower bound. There is additional uncertainty in the spectral shape due to uncertainties as-

sociated with the soil-structure interaction methodology employed. Based on results published by Kennedy, et al (1980), $\beta_{method} = 0.20$. The total spectral shape factor β's are, therefore:

$$\beta_{SA,R} = 0.00$$

$$\beta_{SA,U} = \left[\left\{\frac{1}{1.65} \ln (1.15)\right\}^2 + 0.20^2\right]^{1/2} = 0.22$$

2.4.4.2 Damping Factor, F_δ

B&W used 3% damping for components and 7% damping for reinforced concrete in their NSSS analysis. The reactor vessel and steam generator sit on the base mat and except for seismic load transmitted through their upper bumpers, their response is predominantly influenced by the base mat motion directly at an applicable median damping of 5%. A damping of 3-1/2% of critical is taken as the -1β level.

The reactor coolant pump is hung from above and all lateral load is transmitted through the horizontal snubbers. The governing failure mode occurs in the upper snubbers. Thus, the damping factor for the reactor coolant pump is evaluated using in-structure spectra for the upper snubber location developed by SMA (although these were not actually used by B&W in their analysis).

For the reactor vessel and steam generator, the horizontal spectral accelerations of interest at the 9 Hz NSSS frequency are $S_a(5\%) = 0.34g$, $S_a(3\text{-}1/2\%) = 0.39g$, and $S_a(3\%) = 0.41g$ indicating $F_\delta = 0.41/0.34 = 1.21$. Similarly, the rocking accelerations are $S_a(5\%) = 0.0032g/ft$, $S_a(3\text{-}1/2\%) = 0.0037g/ft$ and $S_a(3\%) = 0.0039g/ft$ indicating $F_\delta = 0.0039/0.0032 = 1.22$. Note that for this case both the horizontal and rocking spectra give essentially the same results, thus there is no additional β

to consider due to incomplete knowledge of the relative contributions of each component of motion to the response. For the reactor vessel and steam generator then

$$\check{F}_\delta = 1.21$$

is used.

$\beta_{\delta,U}$ is evaluated using the 5 and 3-1/2% damped spectral values. Weighting the rocking and horizontal spectra equally gives

$$\beta_{\delta,U} = \ln\left[\frac{1}{2}\left(\frac{0.39}{0.34} + \frac{0.0037}{0.0032}\right)\right] = \ln\left[\frac{1}{2}(1.15 + 1.16)\right]$$

$$= 0.14$$

$$\beta_{\delta,R} = 0.2 \times 0.14 = 0.03$$

For the reactor coolant pump, the in-structure response spectra are used, together with the damping factor evaluated at the reactor coolant pump frequency of 8.5 Hz. For the north-south direction, $S_a(5\%)$ = 0.58g, $S_a(3\text{-}1/2\%)$ = 0.68g and $S_a(3\%)$ = 0.75g indicating F_δ = 0.75/0.58 = 1.29. Similarly, for the east-west direction, $S_a(5\%)$ = 0.58g, $S_a(3\text{-}1/2\%)$ = 0.67g and $S_a(3\%)$ = 0.72g indicating F_δ = 0.72/0.58 = 1.24. Using a similar approach as for combining the two damping factors obtained from the rocking and translation spectra, $\check{F}_\delta$ for the reactor coolant pump is calculated as:

$$\check{F}_\delta = 1/2(1.29 + 1.24) = 1.27$$

and

$$\beta_{\delta,U} = \ln\left[\frac{1}{2}\left(\frac{0.68}{0.58} + \frac{0.67}{0.58}\right)\right] = \ln\left[\frac{1}{2}(1.17 + 1.16)\right]$$

$$= 0.15$$

$$\beta_{\delta,R} = 0.2 \times 0.15 = 0.03$$

2.4.4.3 Modeling Factor, F_M

The modeling employed by B&W is median-centered, therefore:

$$\check{F}_M = 1.0$$

The NSSS is a highly complex three-dimensional, multi-degree-of-freedom system, thus

$$\beta_{M,U} = 0.20$$

$$\beta_{M,R} = 0.00$$

2.4.4.4 Mode Combination Factor, F_{MC}

Modes were combined using the Regulatory Guide 1.92 SRSS rule with allowance for the absolute sum combination of closely spaced modes. This rule is considered to be median-centered, thus,

$$\check{F}_{MC} = 1.0$$

The NSSS is a complex multi-degree-of-freedom system, thus,

$\beta_{MC,R} = 0.15$

$\beta_{MC,U} = 0.00$

2.4.4.5 <u>Earthquake Component Combination Factor</u>, F_{EC}

The earthquake component combination methods used in the capacity evaluation are median-centered, thus

$$\check{F}_{EC} = 1.0$$

The seismically-induced loads are primarily due to input in the two horizontal directions with both contributing to failure, thus,

$\beta_{EC,R} = 0.06$

$\beta_{EC,U} = 0.00$

For the reactor vessel and steam generator, the total equipment response factor as shown in Table 2-2a is:

$$\check{F}_{RE} = 1.0 \times 1.15 \times 1.21 \times 1.0 \times 1.0 \times 1.0 = 1.39$$

$$\beta_{RE,R} = (0.03^2 + 0.06^2 + 0.15^2)^{1/2} = 0.16$$

$$\beta_{RE,U} = (0.22^2 + 0.14^2 + 0.20^2)^{1/2} = 0.33$$

For the reactor coolant pump, the total equipment response factor as shown in Table 2-2b is:

$$\check{F}_{RE} = 1.0 \times 1.15 \times 1.27 \times 1.0 \times 1.0 \times 1.0 = 1.46$$

$$\beta_{RE,R} = (0.03^2 + 0.06^2 + 0.15^2)^{1/2} = 0.16$$

$$\beta_{RE,U} = (0.22^2 + 0.15^2 + 0.20^2)^{1/2} = 0.33$$

2.4.5 Ground Acceleration Capacity, A_C

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

$$\check{A}_C = A_{SME} \cdot \check{F}_C \cdot \check{F}_{RS} \cdot \check{F}_{RE}$$

and the variability estimates as

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

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

Table 2-3 presents the fragility parameters for Midland NSSS 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 2-3) would result in a DEGB. By convolving the Midland seismic hazard curves (Figure 2-9) with the fragility curves of the equipment supports generated using Table 2-3, the probability of an indirect DEGB was calculated. The median probability of an indirect DEGB was calculated as 4.5×10^{-19} per reactor-year and the 10 to 90% subjective probability (interval) on P_{DEGB} was obtained as 0 per reactor-year to 9.8×10^{-14} per reactor-year. These extremely 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

(Figure 2-7) developed for the eastern and midwestern United States. The median probability of DEGB was obtained as 1.2×10^{-9} per reactor-year and the 10 to 90% subjective probability interval on P_{DEGB} was found to be 6.7×10^{-12} per reactor-year to 5.7×10^{-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 (1984) was 3.3×10^{-6} per reactor-year with the 10 to 90% subjective probability interval being 2.3×10^{-7} to 2.3×10^{-5} per reactor-year.

2.5 INDIRECT DEGB PROBABILITY CALCULATIONS FOR WPPSS-1

In the subsections which follow, the procedures used in calculating the probability of an indirect DEGB are further illustrated for the WPPSS-1 plant which was selected as the reference plant for the raised loop configuration. The methodology is, of course, very similar to that used in the evaluation of the Midland facility described in Section 2.4 and therefore only the significant details will be discussed at length. The major difference between the Midland and WPPSS-1 evaluations is that the derivation of the median ground acceleration capacities for the WPPSS-1 component supports is based upon the results of the design analysis for the Normal + SSE event while the Midland support capacities are based upon the results of the recently conducted Seismic Margin Review Study for the analysis of the Normal + SME (Seismic Margin Earthquake) event.

2.5.1 Support Arrangement

The WPPSS-1 Nuclear Steam Supply System (NSSS) consists of two loops which include the reactor vessel, two steam generators, four reactor coolant pumps, and a pressurizer. The reactor vessel is supported on nozzle pads which are part of the four cold leg inlet nozzles. The nozzles are free to move radially on lubrite plates to account for

thermal expansion of the vessel but are restrained against movement in the tangential and vertical directions. The once-through steam generators (OTSG) have two supports. The upper support frame provides lateral restraint at the top of the steam generator by restricting the horizontal movement of four trunnions which are attached to the shell. The lower support is set on a lubrite plate which allows for thermal expansion of the NSSS, is snubbed horizontally along the axis between the reactor vessel and the OTSG centerline, and is restrained horizontally perpendicular to the centerline axis. For the reactor coolant pumps, vertical deadweight and seismic support is provided by vertical support rods while, horizontal rods and snubbers attached to the pump case and to the motor provide lateral seismic support.

It is again assumed that seismic failure of the support of either the reactor vessel, a steam generator, or a reactor coolant pump would unconditionally result in a DEGB of the NSSS piping. It is again also assumed that the failure of both steam generators or all four coolant pumps is perfectly dependent and that the various possible failure modes for a component are dependent.

2.5.2 Capacity Factors

The calculation of the median capacity factors and their variability estimates is described below for each of three major NSSS component supports. The Capacity Factor, F_C, is based upon the determination of the support failure mode exhibiting the least capacity for each component.

2.5.2.1 Reactor Vessel Upper Shear Key

Failure of the RPV support occurs when any of the various components which make up the support is judged to fail. Babcock & Wilcox specified the normal and seismic stresses in a number of the support subcomponents. The four most critical items included the following (see Figure 2-11)

1) Cap Screw
2) Box Beam
3) Upper Shear Key
4) Lower Shear Key

From the calculations shown below, the failure of the upper shear key was found to govern the capacity of the RPV support.

The upper shear key is fabricated from A579 Grade 12a material which has a Code ultimate tensile strength of 145.0 ksi at the design temperature of 225°F. From a review of tensile test data, it is judged that the median tensile strength for high strength steel is approximately 1.1 times the Code specified value. The uncertainty, β_U, on the 1.1 factor is calculated to be 0.06. It is further judged that the ultimate shear strength is approximately 0.6 times the ultimate tensile strength with an uncertainty, β_U, of 0.11. Therefore, the strength value, S, of the shear key is:

$$S = 0.6(1.1)(145) = 95.7 \text{ ksi}$$

There is judged to be an uncertainty of 0.25 on using the ultimate strength to define failure and as a result β_U becomes:

$$\beta_U = (0.6^2 + .11^2 + .25^2)^{1/2} = 0.28$$

Babcock & Wilcox provided data showing that the calculated stresses in the upper shear key due to normal and SSE loads were 0.46 ksi and 15.73 ksi, respectively. Therefore, the Strength Factor, $\check{F}_S$, is:

$$\check{F}_S = \frac{95.7 - 0.46}{15.73} = 6.05$$

and using the second moment method, the variabilities on the calculated Strength Factor are:

$$\beta_{S,R} = 0.00$$

$$\beta_{S,U} = 95.7(0.28)/(95.7 - 0.46) = 0.28$$

Since ultimate shear strength was used to define the failure of the shear key, no credit is taken for ductility. Thus,

$$\check{F}_\mu = 1.00$$

$$\beta_{\mu,R} = 0.00$$

$$\beta_{\mu,U} = 0.00$$

and therefore, the RPV Capacity Factor, $\check{F}_C$, becomes:

$$\check{F}_C = \check{F}_S = 6.05$$

$$\beta_{C,R} = \beta_{S,R} = 0.00$$

$$\beta_{C,U} = \beta_{S,U} = 0.28$$

2.5.2.2 Steam Generator - Lateral Support Bracket

Failure of the steam generator support is assumed to occur when the most critical component of either the upper lateral support system or the lower lateral support and snubber systems reaches its failure capacity. Based upon calculated stress information provided by Babcock & Wilcox, the following OTSG support components were judged to be most critical.

1) Upper Support Truss Beam
2) Upper Support Truss Baseplate and Anchor Bolts
3) Upper Support Trunnion
4) Lower Lateral Support Bracket
5) OTSG Pedestal Wing Plate
6) OTSG Pedestal Support Studs
7) Snubber End Bracket
8) Snubber Rod and Rod End

Based upon the calculations shown below, the failure of the weld joining the Lower Lateral Support Bracket to the built-up beam (Figure 2-12), which together form the Lower Lateral Support Assembly, constitutes the failure mode which governs the capacity of the OTSG support.

The partial penetration groove weld is laid using an E60XX electrode which, based upon a review of material data, has a median ultimate strength of 43.4 ksi. The uncertainty on the ultimate strength value is estimated to be 0.10. The length of the weld is 80 inches with a load eccentricity of 20.5 inches. Therefore, using the weld analysis procedure for eccentrically loaded welds given in the AISC Code, the load capacity of the weld is calculated as

$$P_U = \ell \, C \, C_1 \, D \, (\sigma_u/\sigma_a)$$

where C is an empirical constant related to the load excentricity, C_1 is a constant related to the weld electrode used, and σ_a is the allowable working stress for the weld material on which the value of the constant C_1 is based. For the bracket/beam weld,

$$P_U = 80(1.027)(.857)(.90/.0625)(43.4/18.0) = 2377 \text{ kips}$$

The seismic load on the lateral bracket due to the SSE is 503.4 kips. There is no normal load on the Lateral Bracket. Therefore,

$$\check{F}_S = \frac{2377 - 0}{503.4} = 4.72$$

Again there is judged to be an uncertainty of 0.25 on using the ultimate strength to define failure. As a result the variabilities on the calculated Strength Factor are:

$$\beta_{S,R} = 0.00$$
$$\beta_{S,U} = (.10^2 + .25^2)^{1/2} = 0.27$$

No credit is taken for ductility since ultimate strength is used to define failure. Thus, the steam generator support Capacity Factor is:

$$\check{F}_C = \check{F}_S = 4.72$$
$$\beta_{C,R} = \beta_{S,R} = 0.00$$
$$\beta_{C,U} = \beta_{S,U} = 0.27$$

2.5.2.3 Reactor Coolant Pump - Vertical Leg Pin

Similar to the evaluation of the reactor vessel and steam generator supports, failure of the reactor coolant pump supports is assumed to occur when the most critical component of either the upper or lower snubber assembly, the upper or lower horizontal support assembly, or the vertical support rod assembly reaches its failure capacity. From the stress data provided, the following items were found to be most critical.

1) Lower Snubber Rod and Rod End
2) Vertical Support Leg Pin
3) Vertical Support Leg Bolt Connector
4) Vertical Support Leg Bracket
5) Lower Horizontal Link Bracket
6) Lower Horizontal Link Bolt Connector

Based upon the calculations shown below, the failure of the Vertical Support Leg Pin in combined shear and bending constitutes the failure mode which governs the capacity of the Reactor Coolant Pump support.

The 4.50" diameter pin (Item 11 as shown in Figure 2-13) is fabricated from A471 - Class 8 material which has a Code ultimate tensile strength of 135.9 ksi at the design temperature of 650°F. A review of material data indicates that the median ultimate strength for high strength steels is approximately 1.1 times the Code specified minimum value with an uncertainty variability, β_U, of 0.06. Thus, the tensile capacity, S, of the pin is:

$S = 1.1(135.9) = 149.5$ ksi

$\beta_R = 0.00, \quad \beta_U = 0.06$

From the loads and stress data provided, the direct bending and shear stresses in the pin due to deadweight and other normal loads are 5.95 ksi and 1.73 ksi, respectively. Similarly, the bending and shear stresses in the pin due to the SSE are 12.98 ksi and 3.78 ksi, respectively. The shear and bending interaction equation for the pin can then be written as

$$\frac{[1.73 + F_S(3.78)]^2}{[.6(149.5)]^2} + \frac{[5.95 + F_S(12.98)]^2}{[1.0(149.5)]^2} = 1.0$$

Solving the resulting quadratic equation for $\check{F}_S$, the value of the Strength Factor becomes:

$\check{F}_S = 9.91$

Again there are uncertainties concerning the use of a 0.6 factor to define the ratio of shear ultimate strength to tensile ultimate strength and concerning the use of ultimate strength to define failure. The uncertainty variabilities for these two issues are 0.10 and 0.25, respectively. Combining these with the 0.06 uncertainty variability on the calculation of tensile capacity, the randomness and uncertainty variabilities on the Strength Factor, $\check{F}_S$, are calculated to be

$\beta_{S,R} = 0.00$

$\beta_{S,U} = 0.27$

Since ultimate strength is used to define failure, no credit is again taken for ductility. Thus, the Reactor Coolant Pump support Capacity Factor is:

$$\check{F}_C = \check{F}_S = 9.91$$

$$\beta_{C,R} = \beta_{S,R} = 0.00$$

$$\beta_{C,U} = \beta_{S,U} = 0.27$$

2.5.3 Structural Response Factor

As discussed in Section 2.4.3 the Structural Response Factor, F_{RS}, is made up of several factors determined on the basis of the methods and procedures used in the design evaluation of the containment structure. The calculation of the various factors which affect the assessment of structural response for the WPPSS-1 facility were accomplished in a manner similar to that described for Midland and therefore, will not be repeated here. However, the following important points related to the evaluation of the WPPSS-1 Reactor Coolant Loop are noted which influenced the resultant Structural Response Factors shown in Tables 2-4a through 2-4c.

- 7% damping used in design of containment structure for SSE.
- 7% damped median statistical spectrum for alluvium from WASH 1255 was selected to define the median ground spectrum.
- Component fundamental frequencies are:

  Reactor Vessel - f_N = 15 Hz

  Steam Generator - f_N = 12 Hz

  Reactor Coolant Pump - f_N = 10 Hz

- Average embeddment depth of the containment structure is 73 feet. $F_{SD} = 1.10$.

2.5.4 Equipment Response Factor

Again the Equipment Response Factor, F_{RE}, for the major WPPSS-1 Reactor Coolant Loop component supports were calculated in a manner similar to that described for the Midland facility in Section 2.4.4. However, it was found that the WPPSS-1 evaluation was based on median centered values for each of the contributing factors. As can be seen in Tables 2-4a through 2-4c the Equipment Response Factor, F_{RE}, as well as each of the contributing factors have a value of 1.00. The following important points related to the evaluation of the Equipment Response Factor for the WPPSS-1 Reactor Coolant Loop are noted.

- 7% damping used in design of bolted structures for SSE.
- 7% damping considered median for bolted components.
- A coupled response spectrum design analysis (structure included in model) was the basis for the NSSS evaluation.

2.5.5 Ground Acceleration Capacity

The median ground acceleration capacity, $\check{A}_C$, of each equipment support was calculated from the formula:

$$\check{A}_C = \check{F}_C \cdot \check{F}_{RS} \cdot \check{F}_{RE} \cdot A_{SSE}$$

with the random and uncertainty variabilities on the median acceleration capacity being calculated as

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

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

The resulting fragility parameters for the WPPSS-1 NSSS equipment supports is presented in Table 2-5.

2.5.6 Probability of Indirect DEGB

By convolving the WPPSS-1 seismic hazard curves (Figure 2-10) with the fragility curves of the equipment supports generated using Table 2-5, the probability of indirect DEGB was calculated. The median probability of indirect DEGB was calculated as 1.8×10^{-7} per reactor-year with the 10 to 90% subjective probability (interval) on P_{DEGB} being 3.6×10^{-9} per reactor-year to 4.4×10^{-6} per reactor-year. The sensitivity of the results to the seismic hazard prediction was examined by convolving the above fragility curves with the generic seismic hazard curves (Figure 2-7) developed for the eastern and midwestern United States. The resulting median probability of DEGB was determined to be 2.1×10^{-7} per reactor-year and the 10 and 90% subjective probability interval on P_{DEGB} was found to be 4.2×10^{-9} per reactor-year to 3.3×10^{-6} 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 (1984) was 3.3×10^{-6} per reactor-year with the 10 to 90% subjective probability interval being 2.3×10^{-7} to 2.3×10^{-5} per reactor-year.

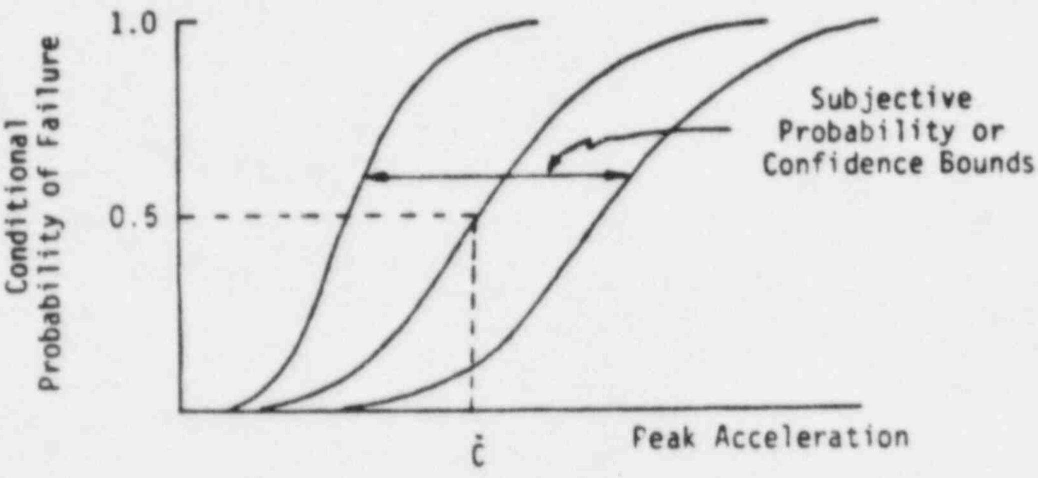

FIGURE 2-1. FRAGILITY OF STRUCTURE OR EQUIPMENT

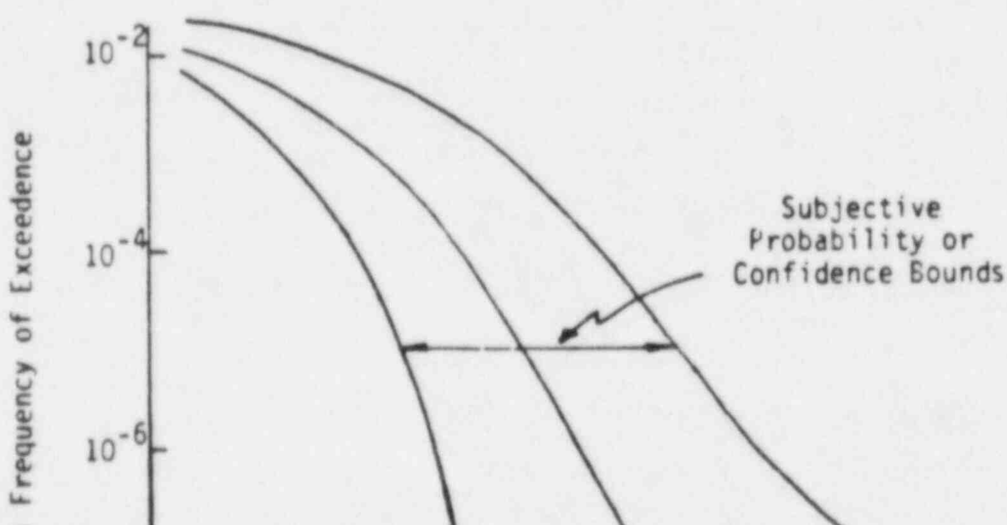

FIGURE 2-2. SEISMIC HAZARD CURVES

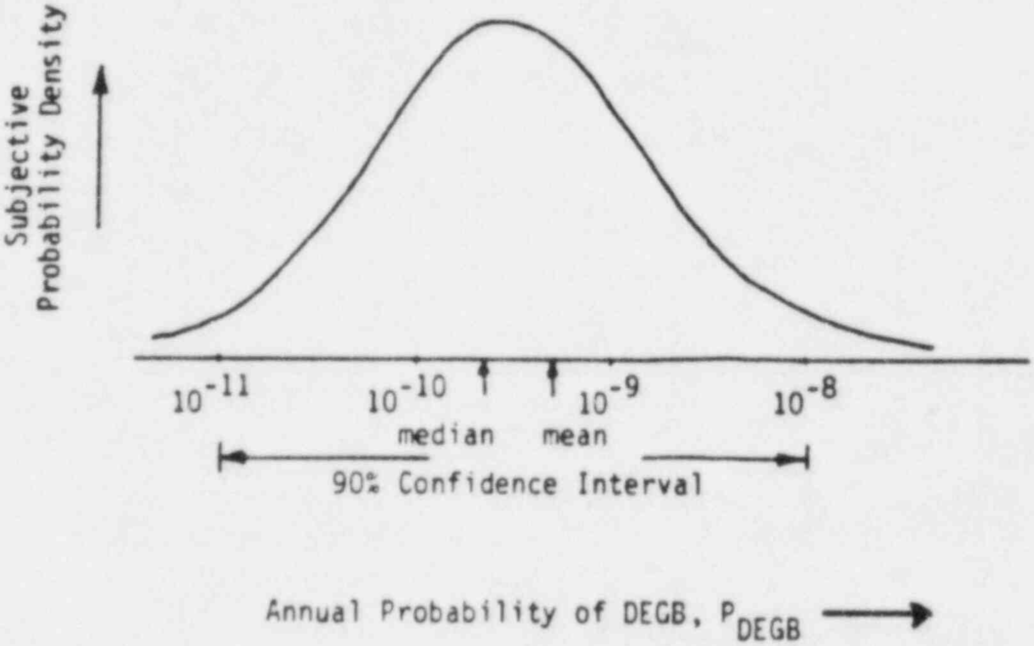

FIGURE 2-3. DISTRIBUTION OF THE PROBABILITY OF INDIRECTLY-INDUCED DEGB

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

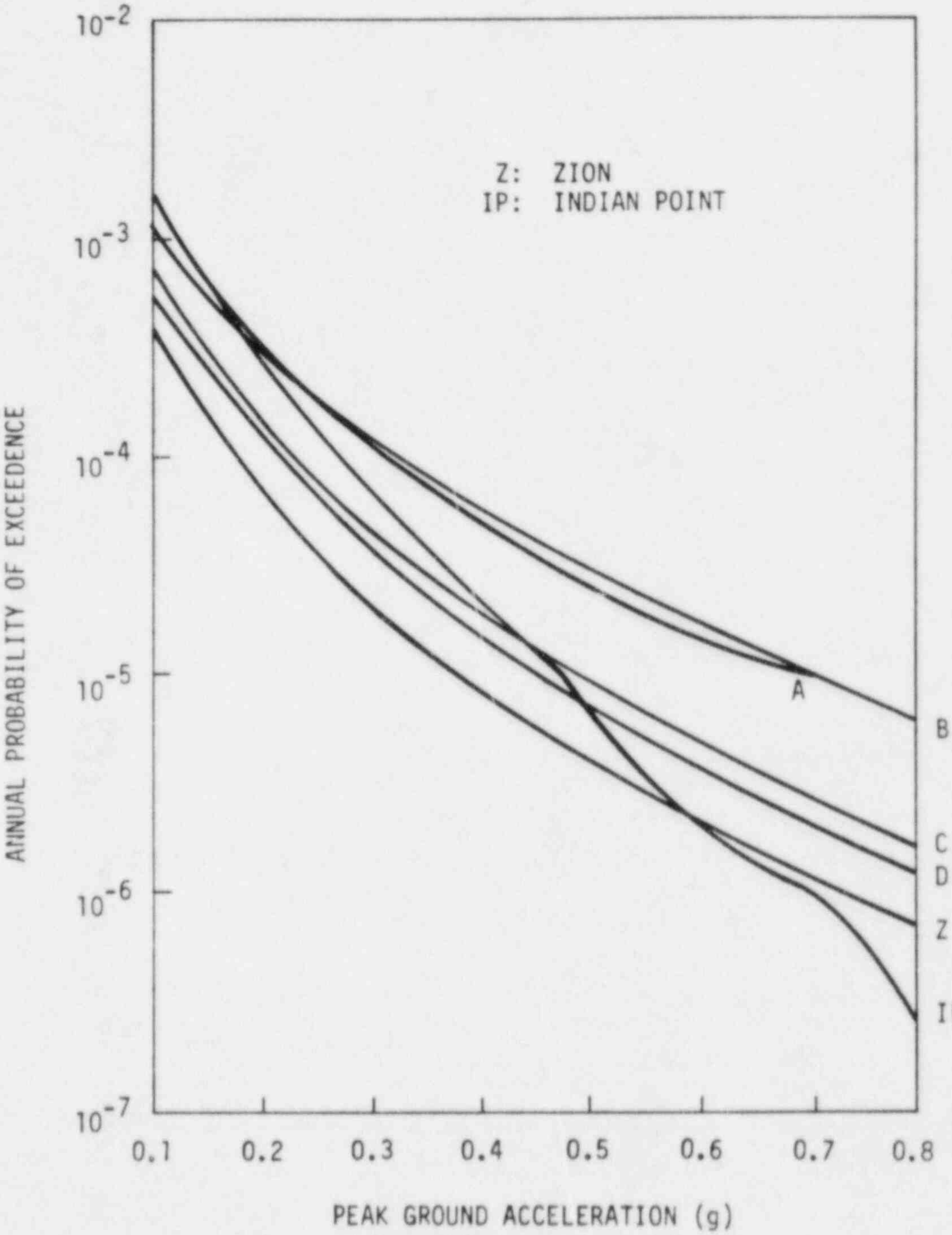

FIGURE 2-5. MEAN SEISMIC HAZARD CURVES

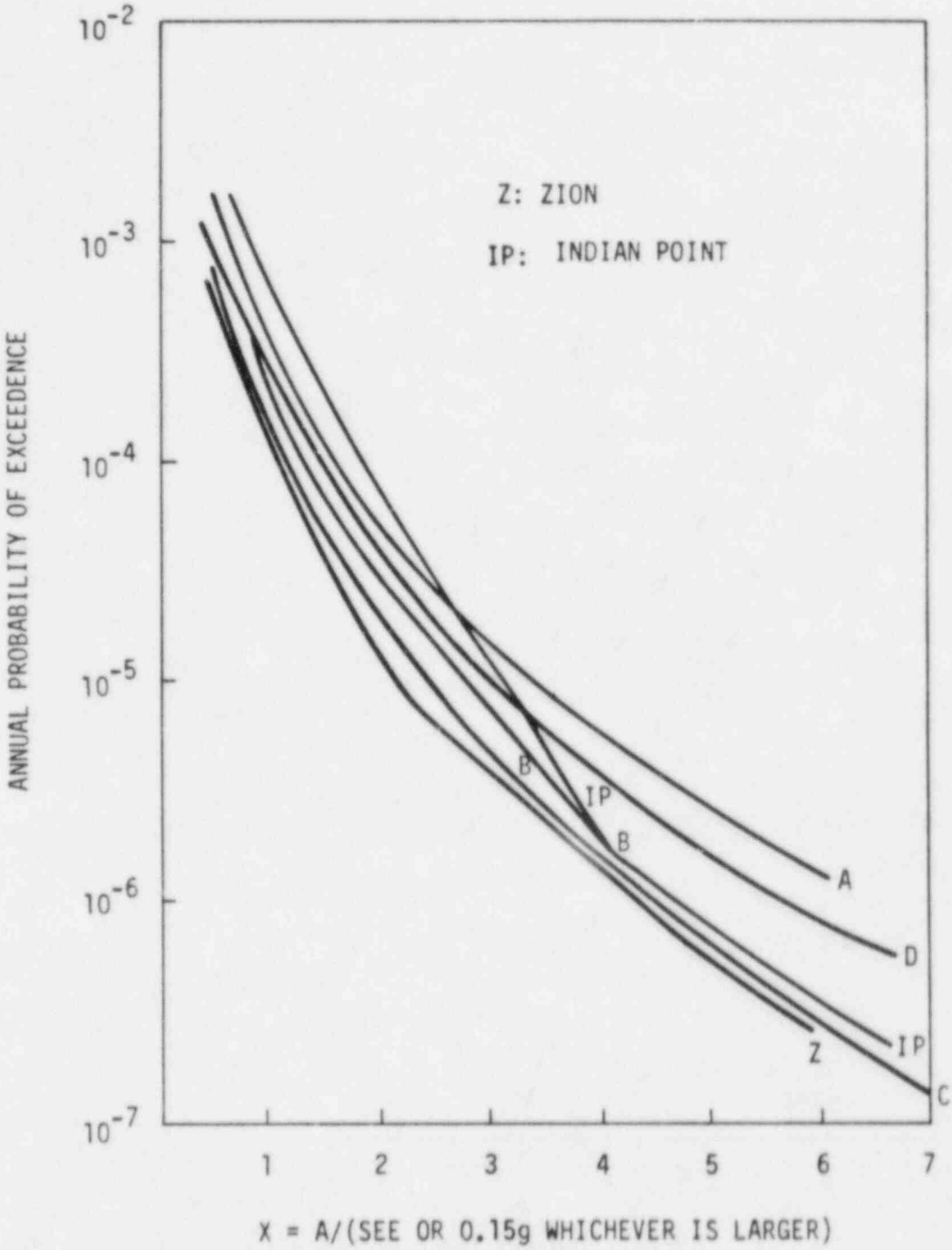

FIGURE 2-6. NORMALIZED MEAN HAZARD CURVES

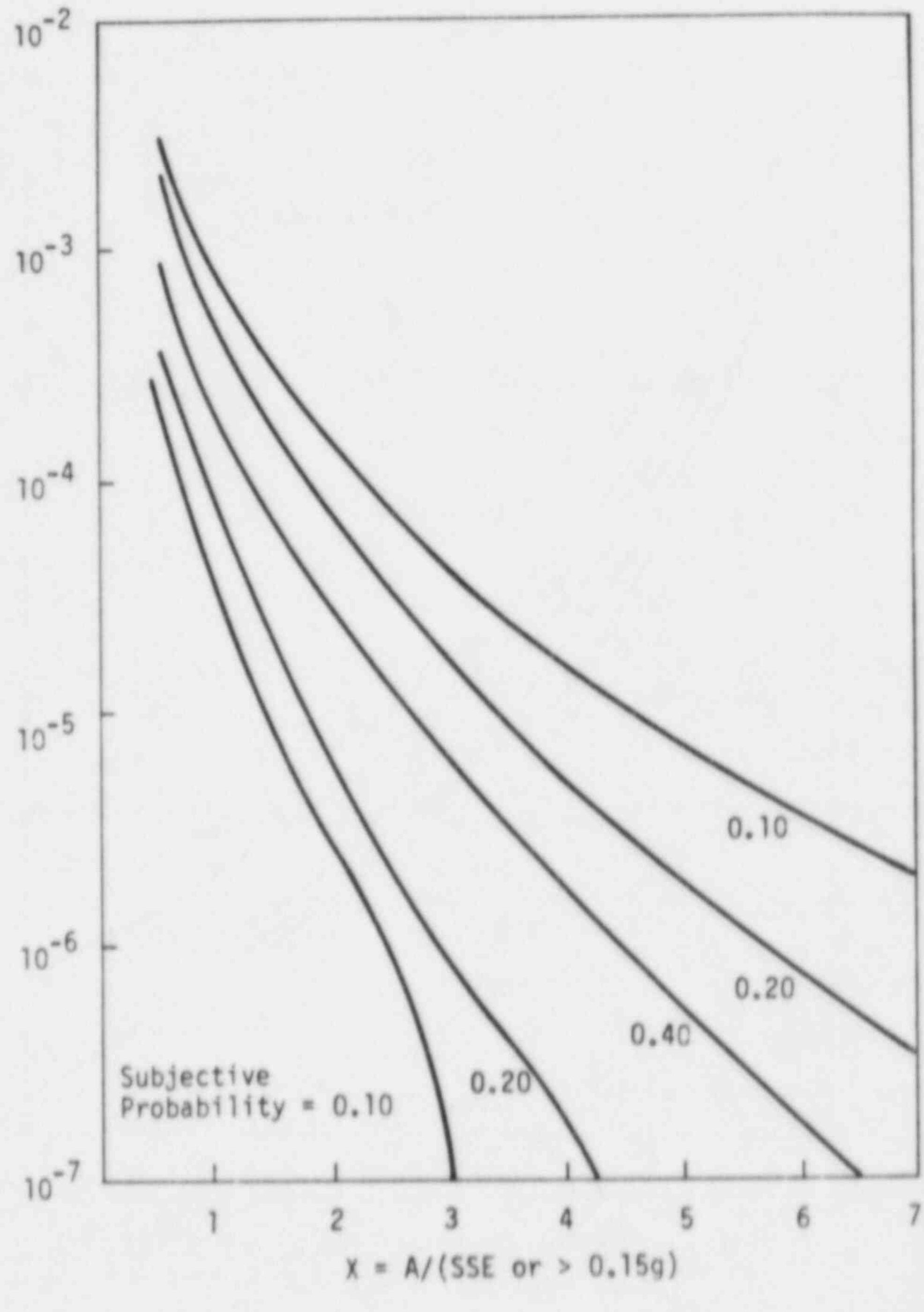

FIGURE 2-7. GENERIC SEISMIC HAZARD CURVES

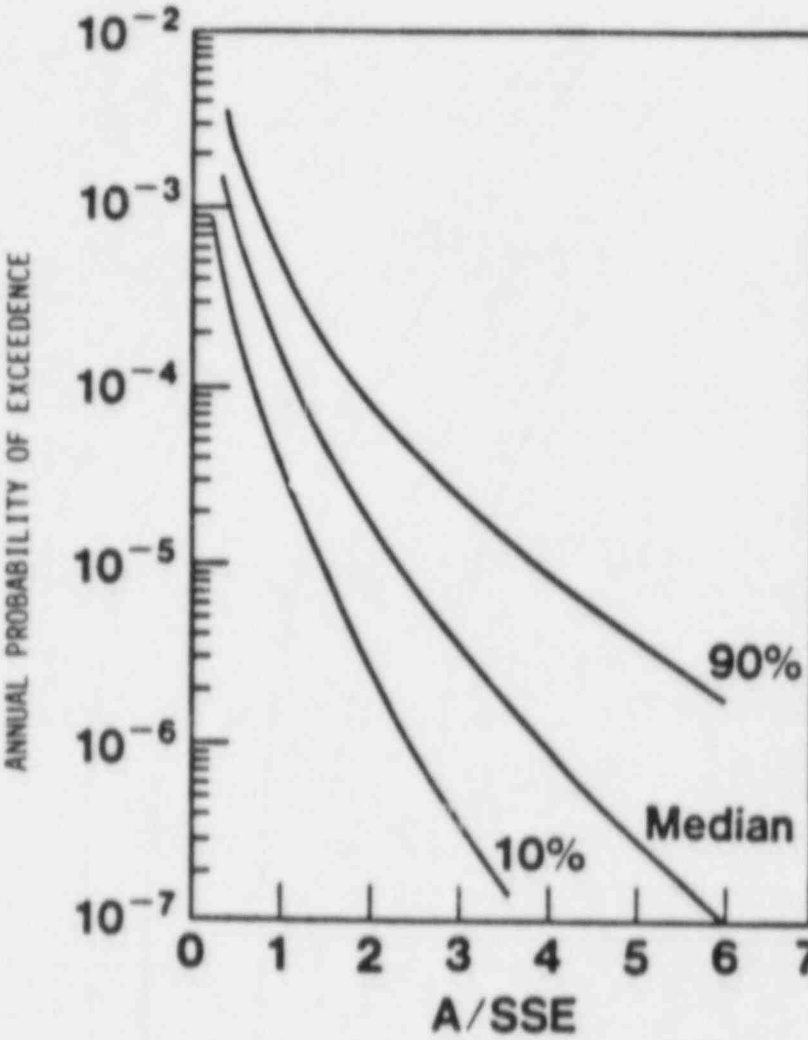

FIGURE 2-8 GENERIC SEISMIC HAZARD CURVES - MEDIAN AND 10% AND 90% CURVES

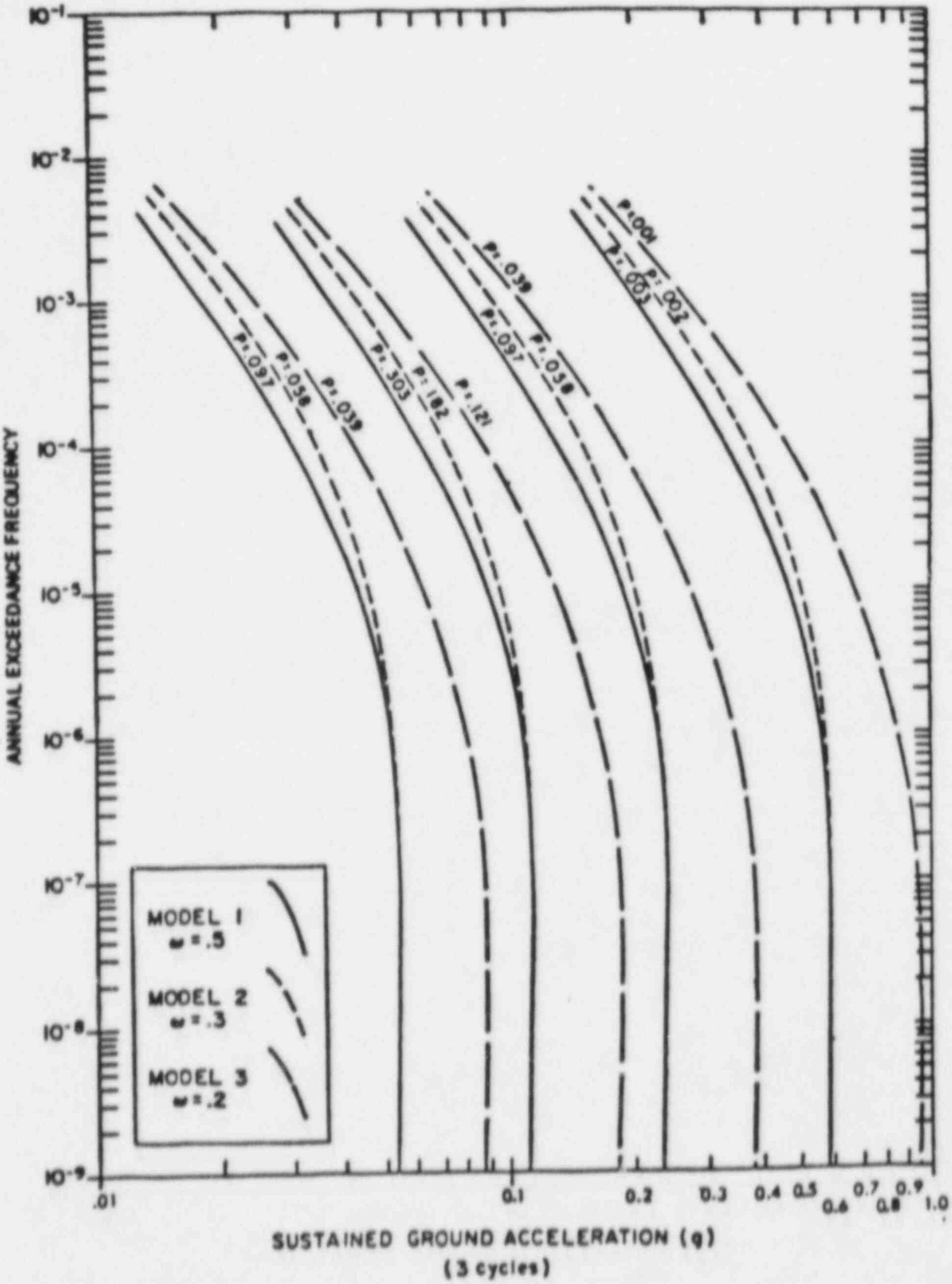

FIGURE 2-9. FAMILY OF SEISMIC HAZARD CURVES FOR THE MIDLAND SITE

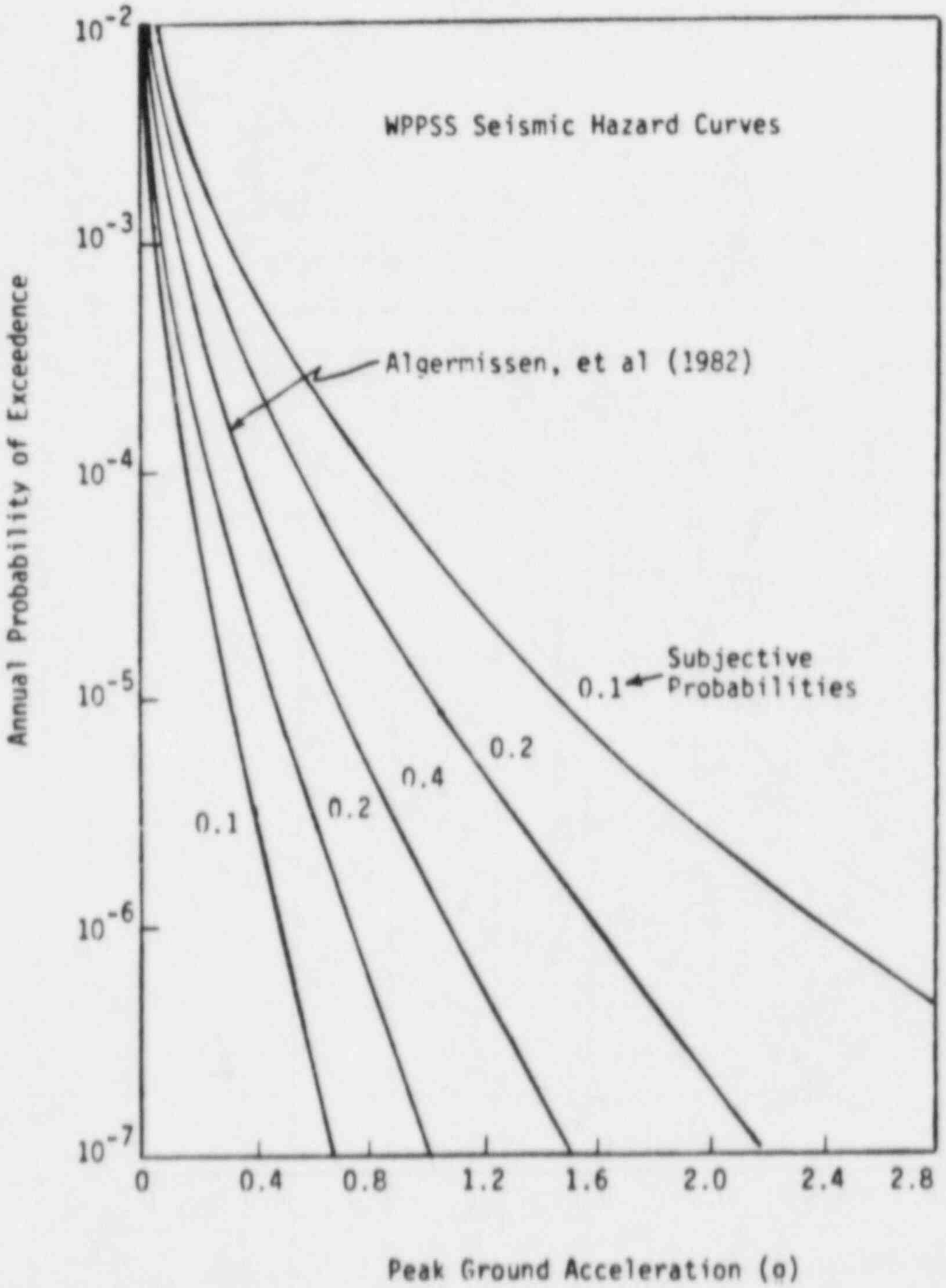

FIGURE 2-10. WPPSS SEISMIC HAZARD CURVES

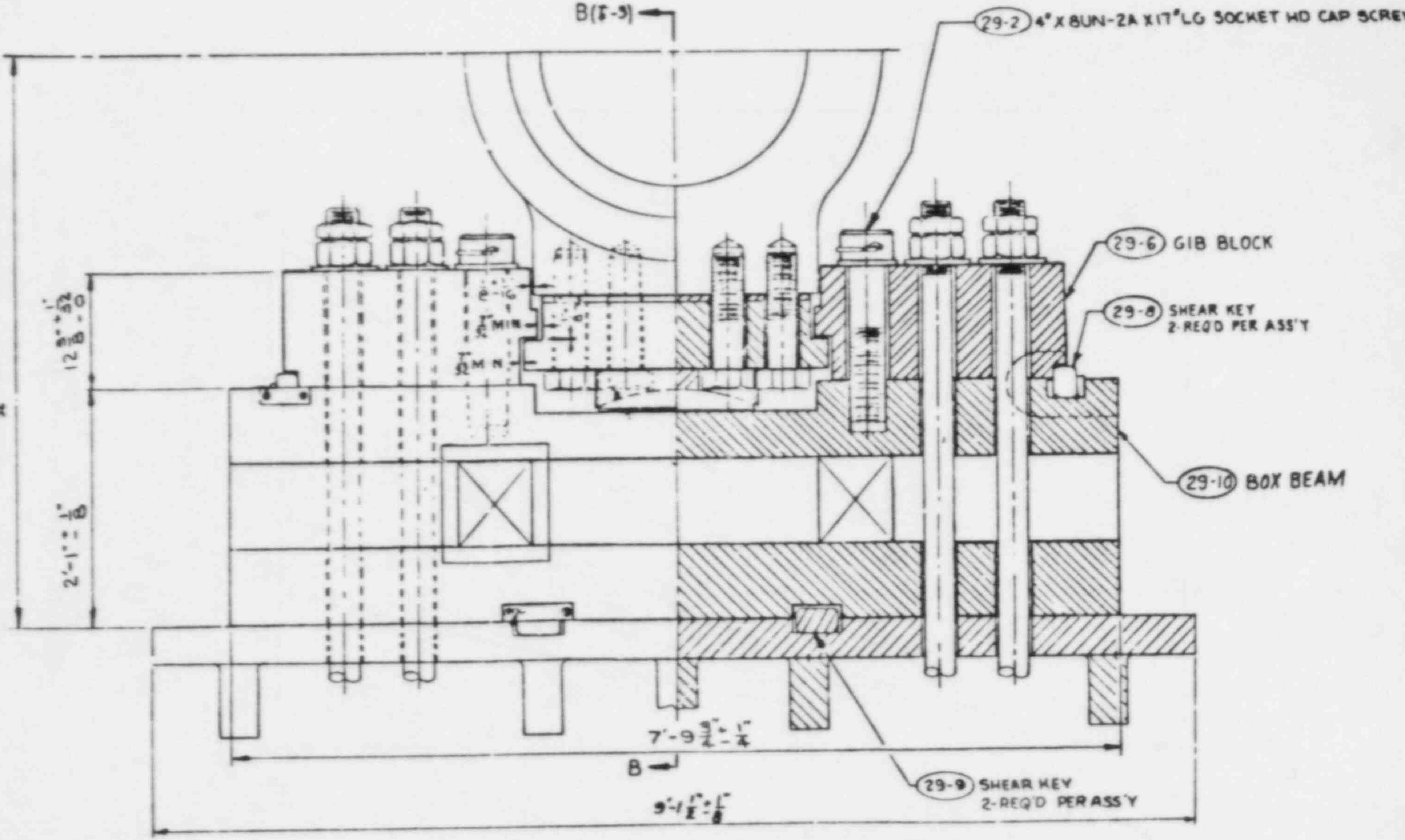

FIGURE 2-11. WPPSS-1 REACTOR VESSEL SUPPORT CROSS SECTION

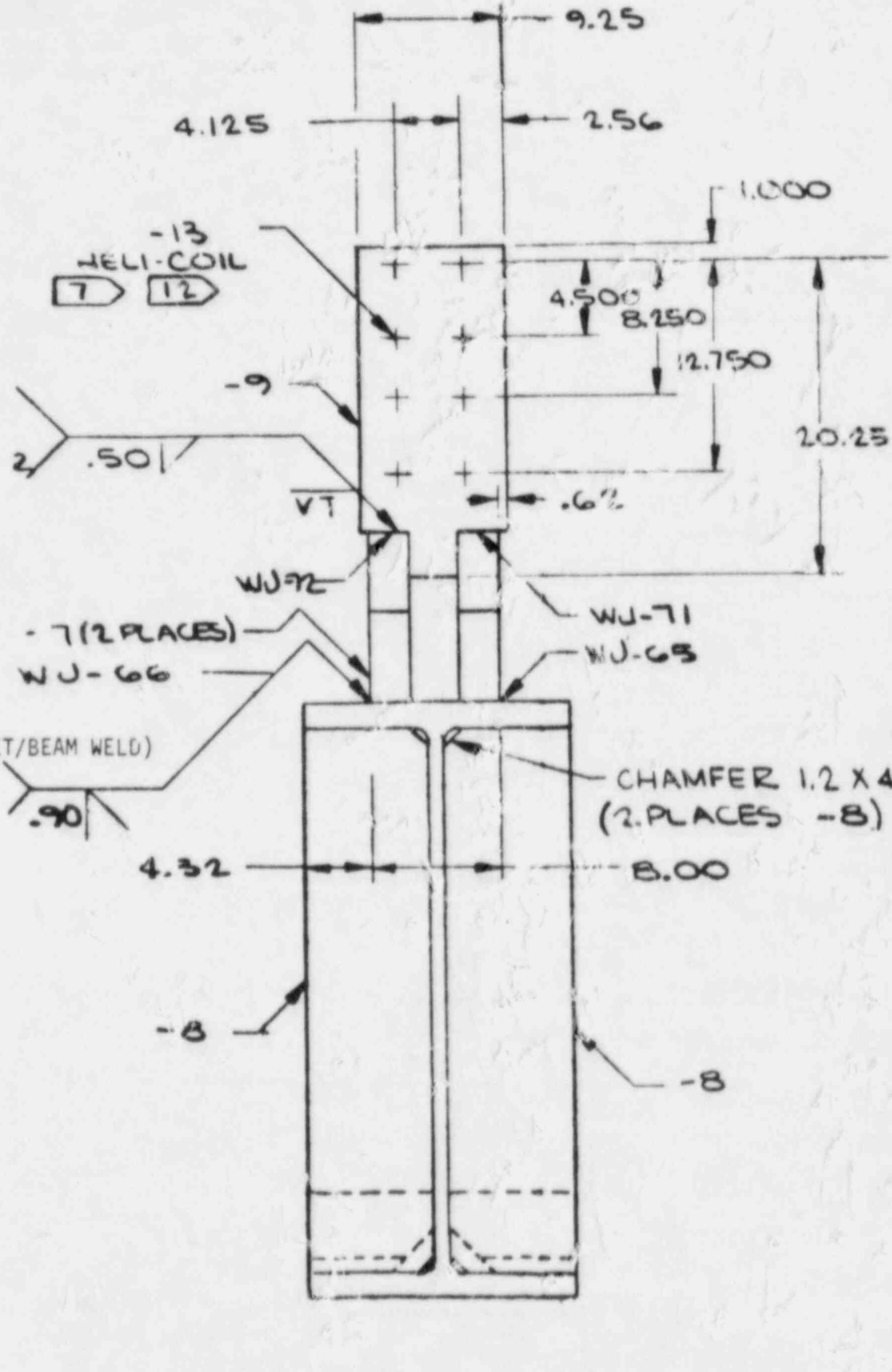

FIGURE 2-12. WPPSS-1 STEAM GENERATOR LOWER LATERAL SUPPORT ASSEMBLY

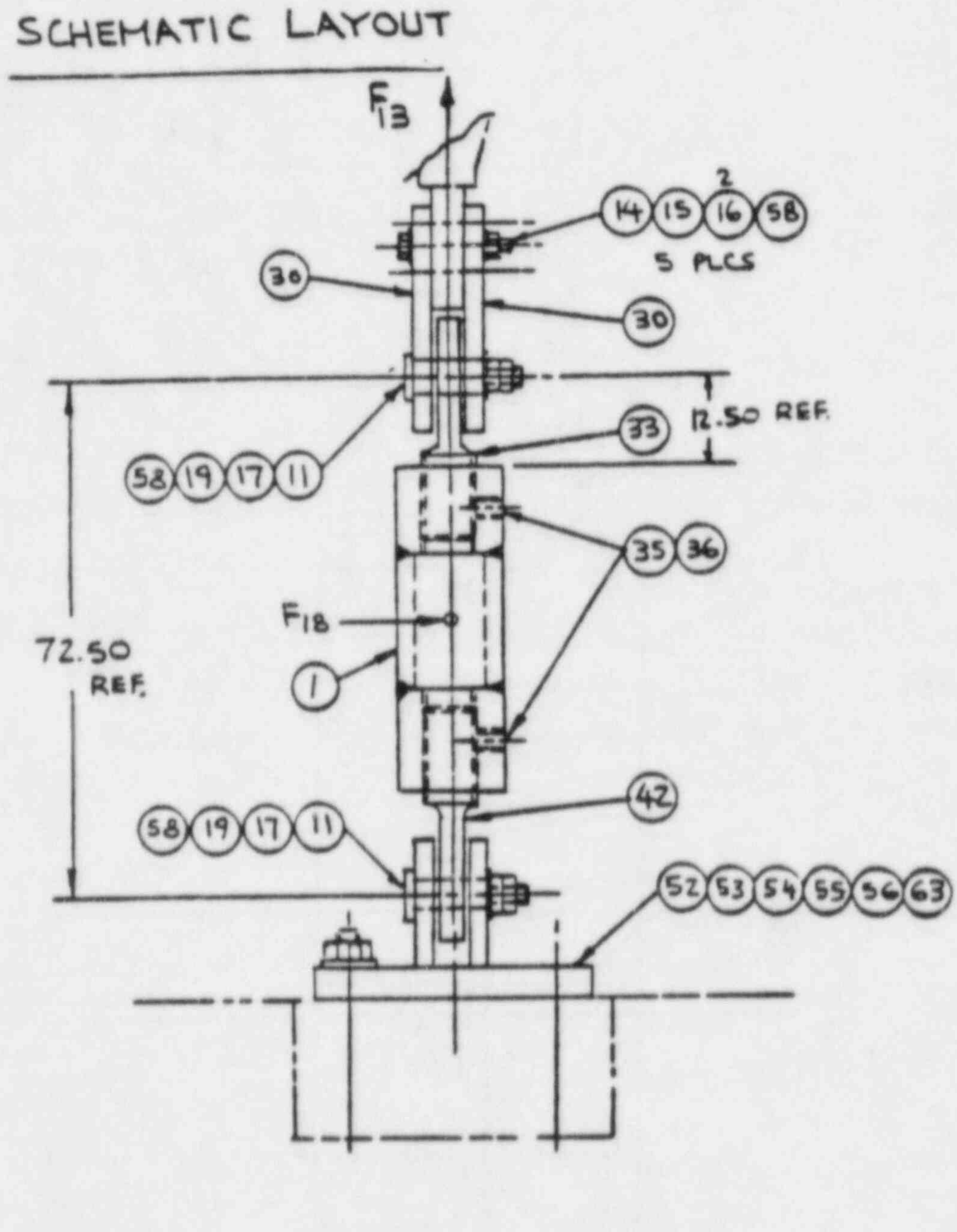

FIGURE 2-13. WPPSS-1 REACTOR COOLANT PUMP VERTICAL SUPPORT LEG ASSEMBLY

TABLE 2-1. MAXIMUM EARTHQUAKE MAGNITUDES

Province	m_b maximum
Model 1	
Michigan Basin	5.3
Cincinnati, Findlay, Kankakee Arch Structure	6.0
New Madrid	7.5
St. Francois-Wabash Valley	6.5
Western Quebec Seismic Zone	6.5
Model 2	
Anna, Ohio Tectonic Structure	6.0
Attica, New York Tectonic Structure	6.0
Niagara Peninsula	5.3
Central Province Background	5.3
New Madrid	7.5
St. Francois-Wabash Valley	6.5
Western Quebec Seismic Zone	6.5
Model 3	
Central Province Background	6.0
New Madrid	7.5
St. Francois-Wabash Valley	6.5
Western Quebec Seismic Zone	6.5

TABLE 2-2a

RESPONSE FACTORS FOR MIDLAND REACTOR VESSEL AND STEAM GENERATOR SUPPORTS

Factor	Median Factor Safety	β_R	β_U
Structure Response			
o Spectral Shape	1.00	0.20	0.07
o Damping	1.08	0.03	0.13
o Modeling	1.00	0	0.25
o Soil-Structure Interaction	1.05	0	0.03
F_{RS}	1.13	0.20	0.29
Equipment Response			
o Spectral Shape	1.15	0	0.22
o Damping	1.21	0.03	0.14
o Modeling	1.00	0	0.20
o Mode Combination	1.00	0.15	0
o Earthquake Component Combination	1.00	0.06	0
F_{RE}	1.39	0.16	0.33
Overall Response Factor	1.57	0.26	0.44

TABLE 2-2b.

RESPONSE FACTORS FOR MIDLAND REACTOR COOLANT PUMP SUPPORT

Factor	Median Factor Safety	β_R	β_U
Structure Response			
o Spectral Shape	1.00	0.20	0.07
o Damping	1.08	0.03	0.13
o Modeling	1.00	0	0.25
o Soil-Structure Interaction	1.05	0	0.03
F_{RS}	1.13	0.20	0.29
Equipment Response			
o Spectral Shape	1.15	0	0.22
o Damping	1.27	0.03	0.15
o Modeling	1.00	0	0.20
o Mode Combination	1.00	0.15	0
o Earthquake Component Combination	1.00	0.06	0
F_{RE}	1.46	0.16	0.33
Overall Response Factor	1.65	0.26	0.44

TABLE 2-3.

MIDLAND NSSS 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	Failure of Upper Bumpers	31.1	4.1	0.26	0.53
2.	Steam Generator	Failure of Upper Bumpers	51.2	6.8	0.26	0.52
3.	Reactor Coolant Pump	Snubber Assembly Failure	18.5	2.5	0.26	0.45

TABLE 2-4a

RESPONSE FACTORS FOR WPPSS-1 REACTOR VESSEL SUPPORTS

f_N = 15 Hertz

FACTOR	MEDIAN SAFETY FACTOR	β_R	β_U
STRUCTURE RESPONSE			
• SPECTRAL SHAPE	1.56	0.22	0.17
• DAMPING	1.00	0.10	0.10
• MODELING	1.00	0.00	0.15
• SOIL-STRUCTURE INTERACTION	1.10	0.00	0.20
F_{RS}	1.71	0.24	0.32
EQUIPMENT RESPONSE			
• QUAL METHOD	1.00	0.00	0.00
• SPECTRAL SHAPE	1.00	0.00	0.00
• DAMPING	1.00	0.02	0.10
• MODELING	1.00	0.00	0.20
• MODE COMBINATION	1.00	0.10	0.00
• EARTHQUAKE COMPONENT COMBINATION	1.00	0.06	0.00
F_{RE}	1.00	0.12	0.22
OVERALL RESPONSE FACTOR	1.71	0.27	0.39

TABLE 2-4b

RESPONSE FACTORS FOR WPPSS-1 STEAM GENERATOR SUPPORTS

$f_N = 12$ Hertz

FACTOR	MEDIAN SAFETY FACTOR	β_R	β_U
STRUCTURE RESPONSE			
• SPECTRAL SHAPE	1.65	0.22	0.17
• DAMPING	1.00	0.10	0.10
• MODELING	1.00	0.00	0.15
• SOIL-STRUCTURE INTERACTION	1.10	0.00	0.20
F_{RS}	1.81	0.24	0.32
EQUIPMENT RESPONSE			
• QUAL METHOD	1.00	0.00	0.00
• SPECTRAL SHAPE	1.00	0.00	0.00
• DAMPING	1.00	0.02	0.10
• MODELING	1.00	0.00	0.20
• MODE COMBINATION	1.00	0.10	0.00
• EARTHQUAKE COMPONENT COMBINATION	1.00	0.06	0.00
F_{RE}	1.00	0.12	0.22
OVERALL RESPONSE FACTOR	1.81	0.27	0.39

TABLE 2-4c

RESPONSE FACTORS FOR WPPSS-1 REACTOR COOLANT PUMP SUPPORTS

$f_N \approx 10$ Hertz

FACTOR	MEDIAN SAFETY FACTOR	β_R	β_U
STRUCTURE RESPONSE			
• SPECTRAL SHAPE	1.75	0.22	0.17
• DAMPING	1.00	0.10	0.10
• MODELING	1.00	0.00	0.15
• SOIL-STRUCTURE INTERACTION	1.10	0.00	0.20
F_{RS}	1.92	0.24	0.32
EQUIPMENT RESPONSE			
• QUAL METHOD	1.00	0.00	0.00
• SPECTRAL SHAPE	1.00	0.00	0.00
• DAMPING	1.00	0.02	0.10
• MODELING	1.00	0.00	0.20
• MODE COMBINATION	1.00	0.10	0.00
• EARTHQUAKE COMPONENT COMBINATION	1.00	0.06	0.00
F_{RE}	1.00	0.12	0.22
OVERALL RESPONSE FACTOR	1.92	0.27	0.39

TABLE 2-5

WPPSS-1 NSSS EQUIPMENT SUPPORT FRAGILITY PARAMETERS

EQUIPMENT SUPPORT	FAILURE MODE	MEDIAN SAFETY FACTOR	GROUND ACCELERATION CAPACITY		
			$\check{A}$	β_R	β_U
REACTOR VESSEL	FAILURE OF UPPER SHEAR KEY	10.35	2.59	0.27	0.48
STEAM GENERATOR	FAILURE OF LATERAL SUPPORT BRACKET WELD	8.54	2.14	0.27	0.47
REACTOR COOLANT PUMP	FAILURE OF VERTICAL SUPPORT LEG PIN	19.03	4.76	0.27	0.47

CHAPTER 3

RESULTS AND CONCLUSIONS

The following sections present the results and conclusions drawn from the study of the probability of an indirect DEGB occurring in the RCL piping of nuclear power plants for which Babcock & Wilcox was the NSSS supplier. The limitations concerning the use of these results in light of certain unresolved issues is also discussed.

3.1 RESULTS

3.1.1 Probability of an Indirect DEGB

The probability for the occurrence of an indirect DEGB has been computed for a total of ten nuclear power facilities for which Babcock & Wilcox was the NSSS contractor. Eight of the plants represented the "lowered loop" configuration with the Midland Energy Center Project serving as the reference plant. The remaining two plants represented the "raised loop" configuration with the Washington Public Power Supply System Unit 1 (WPPSS-1) facility serving as the reference plant. The evaluation of the two reference plants was comprehensive and included all three major NSSS components (reactor vessel, steam generator, and reactor coolant pump). The evaluation of the remaining plants was based on selected information, and was intended only to offer insight as to the applicability of the results to all B&W plants. As a result, the evaluation of the remaining plants was limited to the information available from B&W pertaining only to the reactor vessel and steam generator. No information was obtained from the Architect/Engineer firms for the non-reference facilities.

The results of the indirect DEGB calculations for B&W plants are shown in Table 3-1 and indicate the acceleration capacity, $\check{A}$, for the most critical component support for each plant, the hazard curve used in the calculation of the per year value of P_{DEGB}, and the median, mean, and 10% and 90% subjective per year probability values for the occurrence of a DEGB.

For the lowered loop plants, the evaluation of the reference Midland facility using site-specific seismic hazard curves resulted in a median value of P_{DEGB} of 1.5×10^{-17} per reactor-year with a 10% to 90% subjective probability interval on P_{DEGB} of 1.3×10^{-21} to 1.8×10^{-12} per reactor-year. These probabilities are extremely low and reflect the character of the Midland site-specific hazard curves, which are judged to be quite optimistic. The Midland plants and the other six lowered loop plants were evaluated using the generic seismic hazard curves, which are generally considered to be conservative for sites located in the eastern and midwestern United States. For these cases the median P_{DEGB} value varies from 1.1×10^{-7} to 6.1×10^{-11} per reactor-year (Midland=9.5×10^{-10}). The 90% confidence (subjective probability) value of P_{DEGB} ranges from 1.1×10^{-5} to 8.1×10^{-9} per reactor year (Midland=4.6×10^{-8}). Rancho Seco is the plant exhibiting the highest calculated values for P_{DEGB}. The lower seismic capacity is the result of the high seismic stresses which occur in the steam generator support skirt at the primary pipe and manway opening leading to a potential buckling mode failure of the OTSG skirt. Based upon the information provided, B&W defined skirt buckling to be the most probable failure mode for both the RPV and the OTSG for each of the lowered loop configuration plants. As a result, the lowest median acceleration capacity, $\check{A}$, for each of the non-reference lowered loop plants shown in Table 3-1 as well as the median Capacity Factors, $\check{F}_C$, shown in Table 3-2 for the RPV and OTSG are based upon a skirt buckling failure mode.

Since site-specific seismic hazard curves are not available for the Rancho Seco plant, the generic hazard curves were used in the evaluation. Although the generic hazard curves are conservative for eastern and midwestern plants, it is unclear whether the generic curves (coupled with a 0.25g SSE) are a conservative or unconservative representation of the seismic hazard for the Rancho Seco plant, which is located north of Sacramento, California.

For the raised loop plants, the evaluation of the reference WPPSS-1 facility using pseudo-plant-specific hazard curves derived from seismic hazard data for the region resulted in a median value for P_{DEGB} of 1.8×10^{-7} per reactor-year and a 10% to 90% subjective probability interval on P_{DEGB} of 3.6×10^{-9} to 4.4×10^{-6} per reactor-year. It can be seen from Table 3-1 that the evaluation of the WPPSS-1 plant using the generic hazard curves gave almost identical results. Davis-Besse, the other raised loop plant evaluated, was found to have median, 10% confidence, and 90% confidence values for P_{DEGB} of 8.6×10^{-8}, 1.5×10^{-9}, and 1.4×10^{-6}, respectively based upon the failure of a gusset weld associated with the RPV horizontal support. These appear to be consistent with the reference plant results. For the purposes of comparison, the median probability for the occurrence of an indirect DEGB in a B&W facility, whether raised or lowered loop configuration, is generally less than that for the lowest capacity Westinghouse plant.

3.1.2 Response Factors

An additional benefit of this study is the determination of the overall response factor of safety, F_R, for the equipment supports in each plant. The response factor is described in terms of a median value, $\check{F}_R$, and randomness and uncertainty variability measures, $\beta_{R,R}$ and $\beta_{R,U}$ and has

been estimated by assessing the conservatism or unconservatism present in different stages of the structure/equipment response analysis as described in Chapter 2. Table 3-2 shows the equipment Capacity Factor $\check{F}_C$ and the overall Response Factor ($\check{F}_R$) and their variabilities for the plants evaluated in this study. Since much more information was available for the evaluation of the reference plants (Midland and WPPSS), the Capacity and Response Factors are judged to be representative of a comprehensive evaluation of these plants. However, care should be taken in the use of the factors computed for the non-reference plants which are based upon limited information and are intended only to offer insight on the applicability of the results of the reference plant analyses to all B&W plants.

3.2 COMPARISON WITH PREVIOUS STUDIES

In a previous comprehensive study evaluating the probability of an indirect DEGB in the RCL piping of Westinghouse reactors east of the Rocky Mountains, the median probability of an indirect DEGB was calculated to be 3.3×10^{-6} per reactor-year with the 10% and 90% confidence bounds ranging from 2.0×10^{-7} to 2.0×10^{-5} per reactor-year. These probability values represent the plant exhibiting the lowest seismic capacity for the RCL equipment supports among all Westinghouse reactors. Table 3-1 shows that all B&W reactors have a lower median P_{DEGB} value than the above W plant, and that all B&W plants except Rancho Seco exhibit a lower 90% confidence probability.

The lowest capacity plant in the Westinghouse study was a modern plant which was designed using more sophisticated analytical techniques (i.e., time history analysis of coupled reactor coolant system and containment building model) such that the median overall response factor, F_R, was calculated to be 1.52. From Table 3-2 it can be seen that the overall

response factors for the B&W reactors range from 1.57 to 3.38 indicating that there is a greater conservatism introduced in the analysis methods used for the design of the B&W reactors evaluated in this study.

Similarly, the capacity factor, $\check{F}_C$, for the lowest capacity W plant was calculated to be 3.1. Table 3-2 shows that the capacity factors for the major component supports for the B&W plants evaluated ranged from 2.57 to 118.8 with the value generally being between 3 and 20. Again all B&W plant component supports except the Rancho Seco OTSG support exhibit a higher capacity factor of safety than the lowest capacity Westinghouse plant. The net result of the generally higher capacity and response factors of safety is that the probability of an indirect DEGB occurring in the RCL piping of a B&W reactor is generally lower than that for the lowest capacity W plant.

3.3 SENSITIVITY OF THE RESULTS

In the subsections which follow, the sensitivity of the results of this study to variation in important parameters such as the seismic hazard and gross errors are discussed.

3.3.1 Seismic Hazard

An important variable influencing the calculated P_{DEGB} value is the seismic hazard at the site. Plant-specific seismic hazard curves were available for the two reference plants (Midland and WPPSS-1) and were used in estimating P_{DEGB}. Since the results of the seismic hazard study for Midland appeared to be quite optimistic when compared to the seismic hazard study results for other eastern and midwestern United States plant sites, an alternative set of generic seismic hazard curves were also used to calculate P_{DEGB}. Although the calculated value of P_{DEGB}

was found to be sensitive to the seismic hazard curve used (compare Midland results shown in Table 3-1), even the values of P_{DEGB} calculated using the more conservative generic curves were found to be significantly lower than the P_{DEGB} value for the lowest capacity W plant.

The generic seismic hazard curves were utilized for the evaluation of the remaining non-reference plants. The wide spread of the uncertainty in these generic hazard curves is expected to cover all of the sites in the eastern and midwestern U.S. and is seen in Table 3-1 to give nearly identical results for the WPPSS-1 facility located in south-eastern Washington. It is generally expected that the calculated P_{DEGB} would be lower than that reported in Table 3-1 if site-specific hazard curves were used in the evaluation of a given plant. The possible exception is Rancho Seco, for which it is uncertain whether or not the generic hazard curves provide a conservative representation of the site.

3.3.2 Design and Construction Errors

The calculation of the probability of an indirect DEGB in this study has been based upon a comparison of the B&W computed normal and seismic stresses and the stress levels judged to result in ultimate failure of the equipment supports for the most critical failure modes. This approach assumes that there are no undetected or uncorrected 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, fabricated, and installed under the careful supervision and Quality Assurance procedures of the reactor vendor. However, the topic of design and construction errors (DCE) in nuclear power plants has been broached on many different occasions. The concern is that potential gross DCE's may reduce the safety margins well below the calculated values and that the probability of an indirect DEGB

may be significantly higher. This possibility was examined in depth in the Load Combination Program study of the Westinghouse reactors (Ravindra, et al , 1984). 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 calculated P_{DEGB} values.

In the present study, all of the construction Non-Conformance Reports (NCRS's) written for the Midland Reactor Coolant System between March, 1978 and July, 1984 were reviewed and identified as to their ability to affect the seismic capacity of the RCL equipment supports. A sampling of the types of construction or installation errors discovered included the following:

1) Reactor vessel anchor bolts broken (2)
2) Steam generator support misalignment
3) Reactor coolant pump mislocation (1-1/4")
4) Uncontrolled weld filler used
5) Component position tolerances violated
6) Unqualified welder
7) Gouges in vessel surfaces
8) Required contact between base & sole plates violated
9) Air bubble located in snubber tubing
10) Oil leak in snubber fitting
11) Cracked welds in hold down bolts
12) Anchor bolt bent beyond tolerance
13) Anchor studs detensioned improperly
14) Weld root pass inspection by-passed
15) Unauthorized field weld made
16) Torque wrench calibration certification unacceptable

Based upon a careful evaluation of the above types of construction errors, it is judged that any such error occurring alone or even any two occurring simultaneously, if undetected, would not likely result in a 30% reduction of the equipment support failure capacity and would certainly not result in a 50% reduction in capacity. Furthermore, based upon the comprehensive nature and longstanding activities of the Babcock & Wilcox Quality Assurance Program as it relates to design, fabrication, and installation tasks, it is highly unlikely that any error which could significantly affect the capacity of the equipment supports would go undetected.

However, in order to assess the potential for the increased probability of an indirect DEGB as the result of a gross error, the P_{DEGB} values were recalculated for the WPPSS-1 facility assuming that the failure capacity of the most critical equipment support (OTSG) was reduced by 30% and 50%. Table 3-3 presents the resulting values for P_{DEGB}. It can be seen that a 30% reduction in capacity increases P_{DEGB} by about a factor of 4 over the "error-free" case while a 50% reduction in capacity increases P_{DEGB} by about a factor of 13.5 or just over one order of magnitude. In view of the Quality Assurance and Quality Control procedures adopted for the reactor coolant system of B&W reactors and the lack of sensitivity of P_{DEGB} to gross errors of plausible magnitude make the issue of gross design and construction errors appears to be relatively unimportant relative for purposes of using the results of this study.

3.3.3 Low Fracture Toughness

The Nuclear Regulatory Commission funded a study related to the potential for low fracture toughness and lamellar tearing of component supports in nuclear power facilities. The results of the study were published in

NUREG-0577 and identified structural materials which were potentially susceptible to low fracture toughness and therefore brittle failure. Nuclear plants were ranked into three groups based upon whether or not such materials were used in the fabrication of major component supports. Five B&W plants were placed in Group I (highest susceptibility to low fracture toughness) based upon the use of SA515 steel. The Group I plants have been targeted for additional study to ascertain the fracture toughness of several steam generator and reactor coolant pump support materials. To the best of our knowledge these additional studies have not been completed and therefore the potential effects of low fracture toughness on support capacities have not been included in the evaluation of the probability of an indirect DEGB for B&W plants.

3.4 SUMMARY AND CONCLUSIONS

In this study, the probability of an indirectly-induced DEGB of the RCL piping in B&W reactors has been calculated. Two groups of B&W reactors have been studied. The first included eight plants representing the "lowered loop" configuration while the second included two plants representing the "raised loop" configuration. The seismic margins to ultimate failure of the major equipment supports (reactor vessel, steam generator, and reactor coolant pump) were calculated using design information provided by Babcock & Wilcox. Site-specific and/or generic seismic hazard curves were used along with the calculated seismic margins to compute the indirect DEGB probabilities.

Based upon insight gained and the results of this study, the following conclusions can be drawn:

1. The probability of an indirectly-induced DEGB in the RCL piping due to earthquakes is very low for B&W reactors. Using

very conservative assumptions, the 90% confidence value of P_{DEGB} is found to be less than 1.4×10^{-6} for plants located in the eastern and midwestern United States and 1.1×10^{-5} for west coast plants.

2. Sensitivity studies have shown that even when using conservative generic hazard curves, the resulting P_{DEGB} values are very low and that only very unlikely design and construction errors of implausible magnitude could substantially increase the P_{DEGB} values calculated in this study.

TABLE 3-1

PROBABILITIES OF INDIRECT DEGB IN BABCOCK & WILCOX PLANTS*

PLANT NAME	LOWEST $\check{A}$(g)	HAZARD CURVES	P_{DEGB}/YR MEDIAN	10% PROB.	90% PROB.	REMARKS
MIDLAND (SSE = 0.13g)	2.50 (RCP)	PLANT SPECIFIC	1.5×10^{-17}	1.3×10^{-21}	1.8×10^{-12}	OPTIMISTIC HAZARD CURVES
		GENERIC	9.5×10^{-10}	4.9×10^{-12}	4.6×10^{-8}	
OCONEE (SSE = 0.15g)	2.38 (RPV)	GENERIC	1.5×10^{-9}	1.3×10^{-12}	1.1×10^{-7}	
RANCHO SECO (SSE = 0.25g)	1.39 (OTSG)	GENERIC	1.1×10^{-7}	3.5×10^{-8}	1.1×10^{-5}	APPLICABILITY OF GENERIC CURVES IS QUESTIONABLE
CRYSTAL RIVER (SSE = 0.10g)	2.38 (RPV)	GENERIC	6.1×10^{-11}	2.9×10^{-14}	8.1×10^{-9}	LOW SEISMIC REGION
ARKANSAS 1 (SSE = 0.20g)	4.74 (OTSG)	GENERIC	5.8×10^{-10}	2.4×10^{-12}	3.4×10^{-8}	
WPPSS-1 (SSE = 0.25g)	2.14 (OTSG)	PLANT SPECIFIC	1.8×10^{-7}	3.6×10^{-9}	4.4×10^{-6}	DERIVED FROM SEISMIC HAZARD DATA FOR REGION
		GENERIC	2.1×10^{-7}	4.2×10^{-9}	3.3×10^{-6}	
DAVIS-BESSE (SSE = 0.15g)	1.46 (RPV)	GENERIC	8.6×10^{-8}	1.5×10^{-9}	1.4×10^{-6}	
WESTINGHOUSE LOWEST CAPACITY PLANT		GENERIC	3.3×10^{-6}	2.3×10^{-7}	2.3×10^{-5}	

*Reactor coolant loop piping only.

TABLE 3-2

CAPACITY AND RESPONSE FACTORS OF SAFETY FOR BABCOCK & WILCOX PLANTS

PLANT NAME (SSE)	$\check{F}_C$	β_R	β_U	$\check{F}_R$	β_R	β_U
MIDLAND (0.13g)						
RPV	19.80		0.29	1.57		
OTSG	32.60	0.00	0.28	1.57	0.26	0.44
RCP	11.20		0.11	1.65		
OCONEE (0.15g)						
RPV	5.60	0.00	0.17	2.80	0.29	0.45
OTSG	29.03			2.80		
RANCHO SECO (0.25g)						
RPV	4.37	0.00	0.17	2.16	0.29	0.41
OTSG	2.57			2.16		
CRYSTAL RIVER (0.10g)						
RPV	5.56	0.00	0.17	2.80	0.29	0.45
OTSG	118.8			2.80		
ARKANSAS NUCLEAR 1 (0.20g)						
RPV	11.25	0.00	0.19	2.27	0.29	0.41
OTSG	10.44		0.17	2.27		
WPPSS-1 (0.25g)						
RPV	6.05		0.28	1.71		
OTSG	4.72	0.00	0.27	1.81	0.27	0.39
RCP	9.91		0.27	1.92		
DAVIS-BESSE (0.15g)						
RPV	3.76	0.00	0.28	2.60	0.29	0.41
OTSG	9.48	0.26	0.27	3.38		

TABLE 3-3

SENSITIVITY OF P_{DEGB} TO GROSS DESIGN & CONSTRUCTION ERRORS

WPPSS-1 FACILITY (PLANT-SPECIFIC HAZARD CURVE)

CASE	LOWEST $\check{A}$ (g)	P_{DEGB}/YR			
		MEDIAN	MEAN	10% PROB.	90% PROB.
"ERROR FREE"	2.14	1.8×10^{-7}	1.3×10^{-6}	3.6×10^{-9}	4.4×10^{-6}
30% REDUCTION	1.50	6.8×10^{-7}	4.5×10^{-6}	1.5×10^{-8}	1.9×10^{-5}
50% REDUCTION	1.07	2.5×10^{-6}	1.5×10^{-5}	7.5×10^{-8}	5.9×10^{-5}

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16. Snaider, R.P., et al., "Potential for Low Fracture Toughness and Lamellar Tearing on Component Supports," NRC, NUREG-0577, Draft, October, 1980.

17. Structural Mechanics Associates, Seismic Margin Review Midland Energy Center Project, Vol. VIII, NSSS Equipment and Piping, SMA 13701.05-R003, July, 1983.

18. Transmittal of Design Data for B&W Designed Reactors, Letter from J.F. Walters (B&W) to G.S. Holman (LLNL) including Attachments 1 through 6, dated July 19, 1984.

19. Weston Geophysical, Site-Specific Response Spectra, Midland Plant - Units 1 and 2, February, 1981.

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.
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.
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}).

NOMENCLATURE (Continued)

Symbol	Definition
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.
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; (distance from the site to the earthquake source).
S	Strength of structural element for the particular failure mode.
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.
$\sigma_{\ell na}$	Logarithmic standard deviation of the uncertainty in the peak ground acceleration.

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.

GLOSSARY (Continued)

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.

APPENDIX A

DESIGN INFORMATION FOR WASHINGTON PUBLIC POWER SUPPLY SYSTEM - UNIT 1

SUPPLY SYSTEM NSS-23

1. Spectra — Ref (16) based upon R.G. 1.60 figure 3.7-2

2. ZPA — Ref (16) .167 g's vertical SSE
   .25 g's horizontal SSE
   .125 g's horizontal OBE
   .125 g's vertical OBE

3. 2D/3D — 3D ref (16)

4. Damping — FSAR ref (16)

	OBE	SSE
Eqpt, pip > 12"	2	3
Small piping	1	2
Welded steel	2	4
Bolted	4	7
Prestressed concrete	2	5
Reinforced concrete	4	7

5. Site Type — Soil, see ref (16) 3.7.2.1.1

6. Analysis Method — Response spectrum method ref (16)

7. Model Type — Coupled ref (16)

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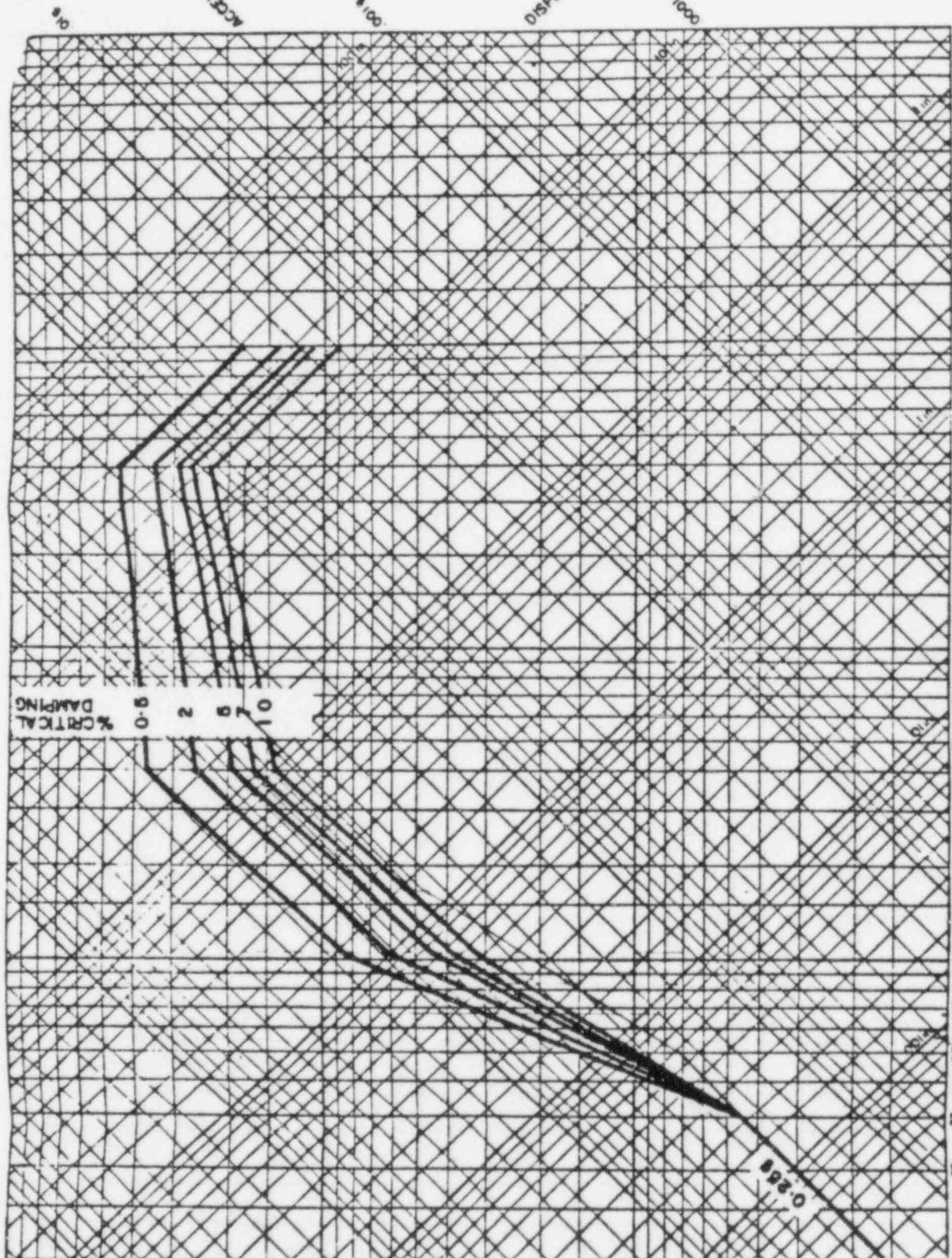
NONCONTRIBUTORY
Djra 6/6/94



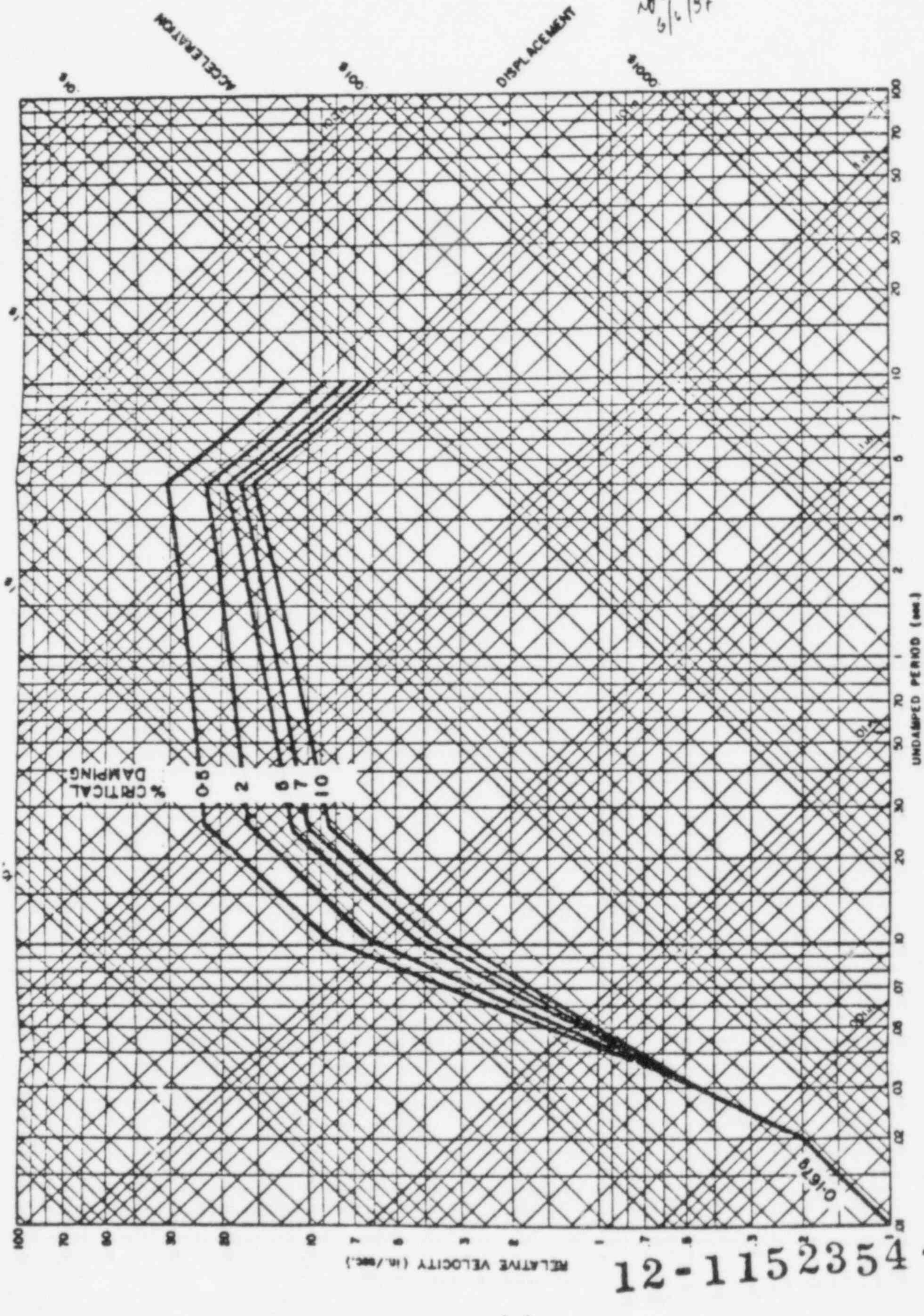
WASHINGTON PUBLIC POWER SUPPLY SYSTEM
NUCLEAR PROJECTS NOS. 1 & 4
FINAL SAFETY ANALYSIS REPORT

DESIGN RESPONSE SPECTRA
HORIZONTAL MOTIONS - SSE

FIGURE 3.7-1

12-1152354-00

NONCONTRIBUTORY



WASHINGTON PUBLIC POWER SUPPLY SYSTEM
NUCLEAR PROJECTS NOS. 1 & 4
FINAL SAFETY ANALYSIS REPORT

DESIGN RESPONSE SPECTRA
VERTICAL MOTIONS - SSE

FIGURE 3.7-2

12-1152354-00

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12-1153039-00

SUPPLY SYSTEM RV SUPPORTS (BOX BEAM)

TO SATISFY DESIGN & EMERGENCY CONDITIONS
EMERGENCY LOADS ANALYZED TO DESIGN ALLOWABLES

LOADS	F_y (KIPS)	F_s (KIPS)
DW	633.4	1.7
TE	322.0	25.7
SSE	± 485.6	± 826.2
SSH	25.2	- 3.4
TOTAL	1466.2	850.2

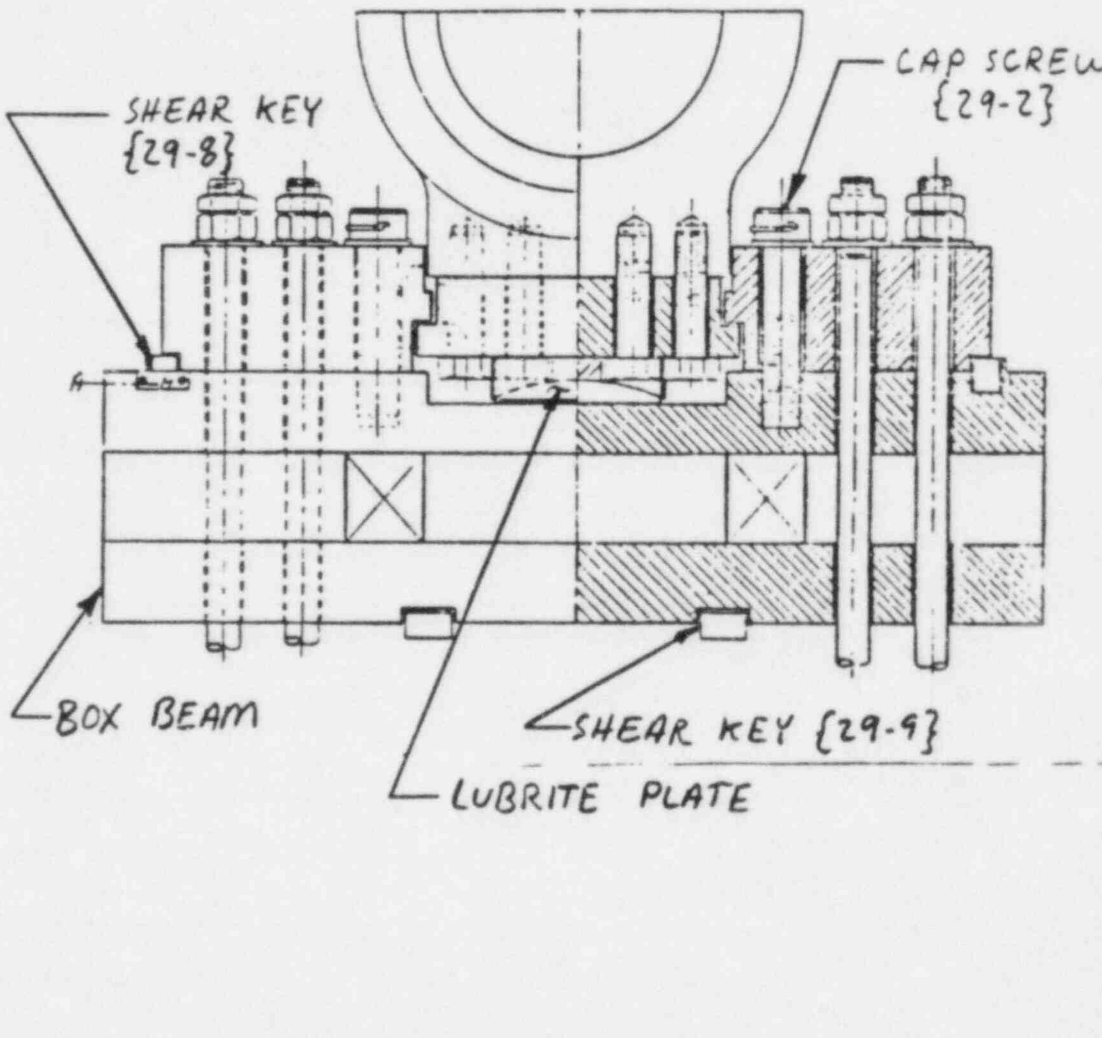
	COMPONENT		CALC STRESS	ALLOW STRESS	
	LUBRITE PLATE		8.71 KSI	10.0 KSI	ASTM-B22 E863
Shear resulting in moment	CAP SCREW {MK 29-2}	(threads)	thread shear 4.79 KSI	19.8 KSI	SA540-B23 Class 1
		(shank)	Tensile σ 11.46 KSI	54.1 KSI	
	Box BEAM (SHEAR)*		8.73 KSI	26.0 KSI	SA533-Gr B Class 2
T σy σu 25° 136 45	SHEAR KEY {mk 29-9}	(BEARING)	27.83 KSI	58.5 KSI	A579 Gr 12a**
		(SHEAR)	8.52 KSI	54.5 KSI	
	SHEAR KEY {mk 29-8}	(BEARING)	20.42 KSI	58.5 KSI	A579 Gr 12a**
		(SHEAR)	16.19 KSI	54.5 KSI	

*SHEAR STRESS ON THE BOX BEAM IS AT PLANE A-A (DUE TO LOADS FROM SHEAR KEY {29-8})

** Code Case 1644, Rev 3, 1974

TITLE		DOC. NO.
PREPARED BY YVP DATE 7-19-84	REVIEWED BY DATE	PAGE NO. 15

SUPPLY SYSTEM RV SUPPORTS
SCHEMATIC



SUPPLY SYSTEMS - NSS 23

SLIDING SUPPORT ASSEMBLY

OTSG SUPPORT SKIRT LOADS & STRESSES:

SUPPORT LOADS: (REF. 13, PAGE M-2.2.2)

LOAD TYPE	LOAD KIPS)	
	F_Y	F_Z
THERMAL EXP.	276.5 (UPWARD)	210.0
DEAD WEIGHT	1475.2 (DOWNWARD)	-
± OBE	±576.6 (UP OR DOWN)	-
± SSE	±1147.6 (UP OR DOWN)	-
SSH	486.7 (UPWARD)	-

LOADING COMBINATION:

CASE I: DESIGN, NORMAL & UPSET CONDITIONS

DOWNWARD THRUST:

TOTAL LOAD = 1475.2 + 576.6 = 2051.8 KIPS

UPWARD THRUST:

TOTAL LOAD = -1475.2 + 576.6 + 486.7 + 276.5

= -135.4 KIPS

THUS, NO UPWARD THRUST EXISTS FOR THIS CONDITION

HORIZONTAL LOADS:

TOTAL LOAD = 210.0 KIPS

SUPPORT STRESSES

CASE III: ~~DESIGN, NORMAL & UPSET~~ FAULTED CONDITIONS

REF.: PAGES M-1.3.1 & M-1.3.2 OF REF. 13.

SUPPORT FORGING:

MAXIMUM BEARING STRESS = 2.4 KSI

ALLOWABLE BEARING STRESS = 28.89 KSI

FASTENERS:

— SUPPORT FORGING TO PEDESTAL

TENSILE STRESS = 101.8 KSI

ALLOWABLE TENSILE STRESS = 107.65 KSI

SHEAR STRESS = 3.03 KSI

ALLOWABLE SHEAR STRESS = 34.95 KSI

— RETAINER RING TO BASE PLATE

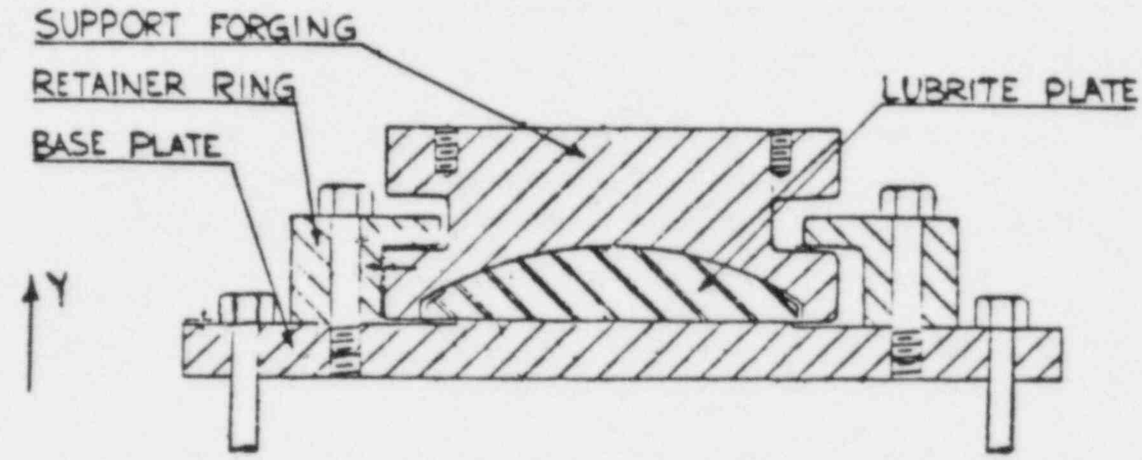
TENSILE STRESS = 103.4 KSI

ALLOWABLE TENSILE STRESS = 107.65 KSI

SHEAR STRESS = 1.05 KSI

ALLOWABLE SHEAR STRESS = 34.95 KSI

OTSG SLIDING SUPPORT ASSEMBLY — NSS 23



Pedestal/Support Forging

(10) 3"-UNC 4 Bolts, A = 5.97 in², Material = SA540-B23, Class 1

Retainer Ring/Base Plate

(6) 4"-8N Bolts, A = 11.81 in², Material = SA540-B23, Class 1

SUPPLY SYSTEM - NSS 23 - PEDESTAL SUPPORT FOR OTSG

DESIGN CONDITION LOADS AND STRESSES FOR PEDESTAL SUPPORT ARE TABULATED.

PEDESTAL SUPPORT LOADS:

REF. : PAGE L-2.6.2 OF REF. 12

LOAD TYPE	FORCES (KIPS) FX	FY	FZ
DEAD WT.	0.0	1387.8	0.0
OBE SEISMIC	±531.	±435.	±507.1
SSE	961	829	938
SS HYDRAULICS	0.	−351.6	0.

Mmax = 13.215"

LOAD TYPE	MOMENTS (FT-KIPS) MX	MY	MZ
DEAD WEIGHT	0.	0.	0.
OBE SEISMIC	±558.6	±177.	±584.9
SSE Seismic	1033	319.	1059
SS HYDRAULICS	3.0	0.3	0.

LOADING COMBINATION:

DW + OBE + SS HYDRAULICS

REF. : PAGE L-2.8.3 OF REF. 12

$F_A = F_y = 1471.2$ KIPS

$F_S = (F_x^2 + F_z^2)^{1/2} = 734.2$ KIPS

$M_T = M_y = 2127.6$ IN-KIPS

$M_0 = (M_x^2 + M_z^2)^{1/2} = 9730.4$ IN-KIPS

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12-1153039-00
GENERAL CALCULATIONS

FROM PAGE L-2.9.11,

MAXIMUM PRIMARY STRESS,

$\sigma_m = 5,473$ PSI $< S_m = 26,700$ PSI

THE MAX. $P_L + P_b$ STRESS OCCURS IN WING PLATE.

WING PLATE LOADS: (DESIGN CONDITION)

OBE

$F_N = -41.619$ KIPS
$F_S = 147.229$ KIPS
$M_o = 2242.08$ IN-KIPS

(FROM PAGE L-2.8.19)

FROM PAGE L-2.9.13,

MAX. $P_L + P_b = \underline{13.047 \text{ KSI}} < S_m = 40.05$ KSI

OBE $F_x = 139.25$ F_s $\theta = 8.70°$
F_z 63.388

SSE F_x 251.7
F_z 117.225

Loads in each wing plate at minimum section

Each Lateral Support = 2(251.7) = 503.4 kips (F_x)
Each Snubber = 2(117.225) = 234.45 kips (F_z)

$M_{o_{SSE}} = 1.8284(2242.08) = 4099.5$ in-kips

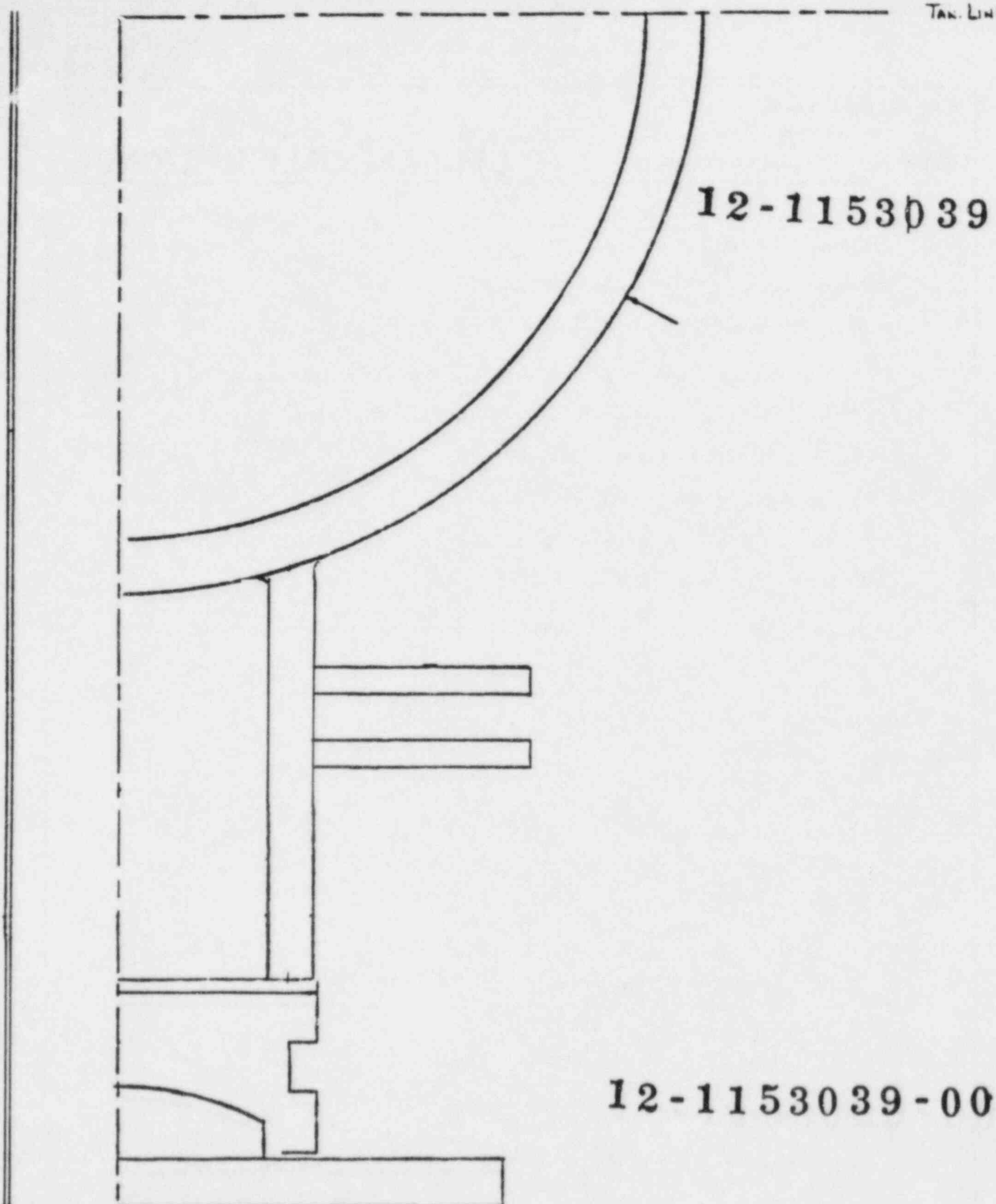
$F_N = (-117.225)\cos 8.7° + (251.7)\sin 8.7° = -77.81$

$F_S = (251.7)\cos 8.7° + (117.225)\sin 8.7° = 266.54$

$\sigma_{SSE} \approx 23.86$ ksi

TITLE	DOC. NO.

PREPARED BY	DATE	REVIEWED BY	DATE	PAGE NO.
YVP	7-18-84	BFB	7/18/84	25



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~~DEPARTMENT~~ OTSG DATE 7-18-84 BY YVP
PEDESTAL SUPPORT-NSS 23 CHECKED DATE BY
JOB NO.

REVISION
OTSG
SHEET 26

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GENERAL CALCULATIONS

Supply System - Answers to Ref (A)

III. A. 3. 2. per 32-3089-02 S.S. RCS Seismic and Static Analysis Ref (7)

(1) The Supply System NSSS piping model is coupled with the concrete structure

(2) The analysis used the response spectrum method.

(3) Damping values used in the analysis

	OBE	SSE
EQPT, PIP > 12"	2	3
Small piping	1	2
Welded steel	2	4
Bolted	4	7
Prestressed concrete	2	5
Reinforced concrete	4	7

(4) The fundamental frequencies of the S.S. loop model can be found in the attached microfiche.

(1) ACMQOAX, ML Jarrell 3/30/84
(2) ACMQCUF, ML Jarrell 4/2/84

The former contains frequencies associated with the X-Y earthquake, while the latter contains Z frequencies. These runs are seperate because of the different boundary conditions employed.

TITLE

DOC. NO.

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GENERAL CALCULATIONS

SUPPLY SYSTEM

III. A. 3. 6.

(1) HORIZONTAL / VERTICAL SPECTRA RESULTS ARE COMBINED USING THE (SRSS) SQUARE ROOT OF THE SUM OF THE SQUARES

(2) ROTATIONAL OR ROCKING SPECTRA WAS Included in The Analysis

(3) THE ABSOLUTE VALUE OF THE ROTATIONAL SPECTRA RESULTS ARE COMBINED DIRECTLY (MODE-BY MODE) WITH THE ABSOLUTE VALUE OF THE HORIZONTAL SPECTRA RESULTS

ie X + ROT (Z)
and Z + ROT (X)

(4) MODAL RESULTS ARE COMBINED USING THE "CLOSELY SPACED" METHOD. THE ROSENBLEUTH SUM SOLUTION TECHNIQUE IS EMPLOYED.

III. A. 4. 2.

THIS MATERIAL IS CONTAINED IN 12-1152354-00 OR REV 00 of this document

- Model description, spectrum, damping, SSE level etc.

INTERFACE SPECTRA AND FUNDAMENTAL FREQUENCY ARE NOT REPORTED HERE SINCE THE MODEL IS COUPLED WITH THE CONCRETE STRUCTURE

TITLE | DOC. NO.

SUPPLY SYSTEM
THE FOLLOWING QUESTIONS REGARD THE HANDLING OF SOIL-STRUCTURE INTERACTION.

III. A. 4. 6

Per SS. FSAR 3.7.2.4.1

"FINITE ELEMENT ANALYSIS WAS USED TO INVESTIGATE THE INTERACTION BETWEEN THE SOIL AND THE CONTAINMENT" SEE FSAR FOR FURTHER DETAIL. See pages 48 to 49.

CONSUMERS POWER CO

SEE ATTACHMENT 2 PAGES 86 to 89

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GENERAL CALCULATIONS

SUPPLY SYSTEM

THE FOLLOWING SECTION CONTAINS A BRIEF SEISMIC LOAD TABULATION OF INTERFACE POINTS TO ALL NON- B/W Supplied SUPPORTS & RESTRAINTS

(IE)

(1) RC PUMP SNUBBERS AND PIN-HIN. SRCS.

(2) OTSG UPPER LATERAL SUPPORT

(3) OTSG LOWER SNUBBERS

B/W DOES NOT SUPPLY THIS HARDWARE, NOR DOES IT QUALIFY THESE RESTRAINTS. THESE LOADS ARE SENT TO THE ASSOCIATED A/E AS INPUT TO THEIR STRESS ANALYSIS

THE SS COORDINATE SYSTEM IS SHOWN IN THE ANSWER TO QUESTION II A 3 B

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GENERAL CALCULATIONS

SUPPLY SYSTEM

PUMPS & MOTORS

		FORCE (KIPS)		
		F_x	F_y	F_z
(1) P1A2 VERTICAL SUPPORT JT 210 FIG 6D	OBE	–	92.4	–
	SSE	–	161.0	–
(2) P1A2 VERTICAL SUPPORT SW JT 209 FIG 6D	OBE	–	101.3	–
	SSE	–	177.1	–
(3) P1A2 VERTICAL SUPPORT NE JT 211 FIG 6D	OBE	–	90.0	–
	SSE	–	157.8	–
(4) P1A1 VERTICAL SUPPORT SE JOINT 22 FIG 6D	OBE	–	84.4	–
	SSE	–	156.8	–
(5) P1A1 VERTICAL SUPPORT NW JT 20 FIG 6D	OBE	–	100.6	–
	SSE	–	175.9	–
(6) P1A1 VERTICAL SUPPORT NE JT 21 FIG 6D	OBE	–	91.9	–
	SSE	–	160.1	–

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GENERAL CALCULATIONS

SUPPLY SYSTEM
PUMPS / MOTORS FORCE (KIPS)

		F_X	F_Y	F_Z
(7) P1A2 LOWER HORZ PIN BAR JT 154 FIG 6C	OBE	28.2	–	22.8
	SSE	50.8	–	41.1
(8) P1A2 LOWER HOR. PIN BAR JT 141 FIG 6C	OBE	28.2	–	22.8
	SSE	50.8	–	41.1
(9) P1A2 LOWER HOR SNUBBER JT 139 FIG 6C	OBE	27.1	–	53.4
	SSE	50.3	–	62.1
(10) P1A1 LOWER HOR. PIN BAR JT 16 FIG 6C	OBE	27.5	–	22.3
	SSE	49.6	–	40.1
(11) P1A1 LOWER HOR. PIN BAR JT 18 FIG. 6C	OBE	27.5	–	22.3
	SSE	49.6	–	40.1
(12) P1A1 LOWER HOR. SNUBBER JT 17 FIG 6C	OBE	27.2	–	53.6
	SSE	50.4	–	62.2
(13) P1A2 UPPER HOR. PIN BAR JT 179 FIG 6B	OBE	47.2	–	38.2
	SSE	92.7	–	75.1
(14) P1A2 UPPER HOR. PIN BAR JT 180 FIG 6B	OBE	48.2	–	39.0
	SSE	96.4	–	78.1

TITLE

DOC. NO.

GENERAL CALCULATIONS

SUPPLY SYSTEM
PUMPS & MOTORS

		FORCE (KIPS) F_X	F_Y	F_Z
(15) P1A2 UPPER HOR SNUBBER JT 194 FIG 6B	OBE	28.5	—	35.2
	SSE	50.2	—	62.0
(16) P1A1 UPPER HOR. PIN BAR JT 59 FIG 6B	OBE	45.3	—	36.6
	SSE	88.8	—	71.9
(17) P1A1 UPPER HOR. PIN BAR JT 67 FIG 6B	OBE	46.4	—	37.5
	SSE	92.8	—	75.1
(18) P1A1 UPPER HOR SNUBBER JT 68 FIG 6B	OBE	27.9	—	34.5
	SSE	49.7	—	61.3

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GENERAL CALCULATIONS

SUPPLY SYSTEM

STEAM GENERATOR LOADS

		FORCE (KIPS)		
		F_X	F_Y	F_Z
(1) LOWER SUPPORTS (4 SNUBBERS TOTAL)	OBE	—	—	132.9
JT 150 FIG 3A	SSE	—	—	23[illegible].4
(2) UPPER SUPPORT NORTH TRUNNION	OBE	—	—	141.3
JT 265 FIG 3B	SSE	—	—	256.9
(3) UPPER SUPPORT EAST TRUNNION	OBE	172.6	—	—
JT 266 FIG 3B	SSE	251.0	—	—
(4) UPPER SUPPORT SOUTH TRUNNION	OBE	—	—	141.6
JT 270 FIG 3B	SSE	—	—	256.6
(5) UPPER SUPPORT WEST TRUNNION	OBE	144.2	—	—
JT 275 FIG 3B	SSE	213.3	—	—

TITLE

DOC. NO.

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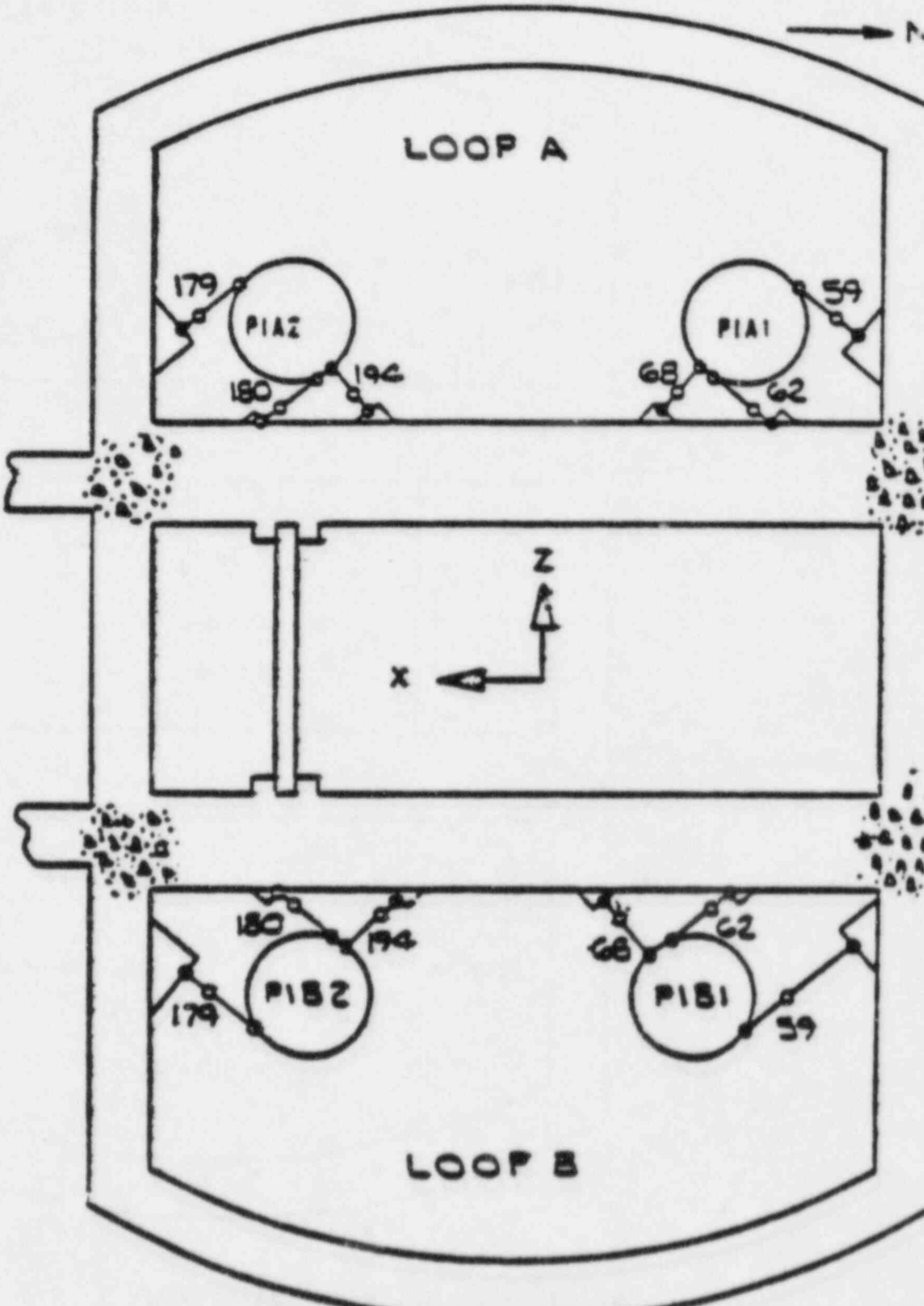

REACTOR COOLANT PUMP HORIZONTAL
SUPPORTS AT ELEVATION 458-2 1/4
FIGURE 68

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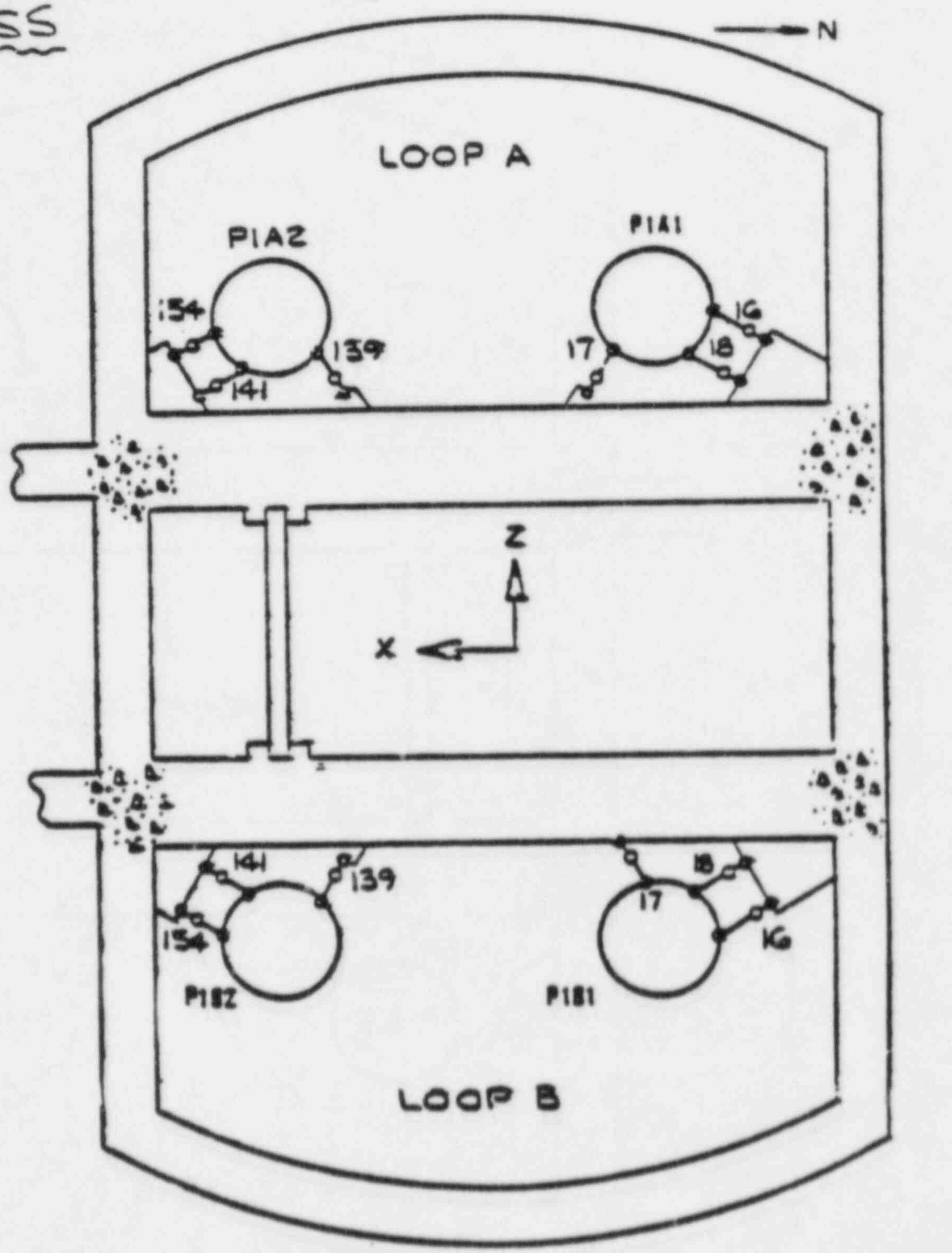
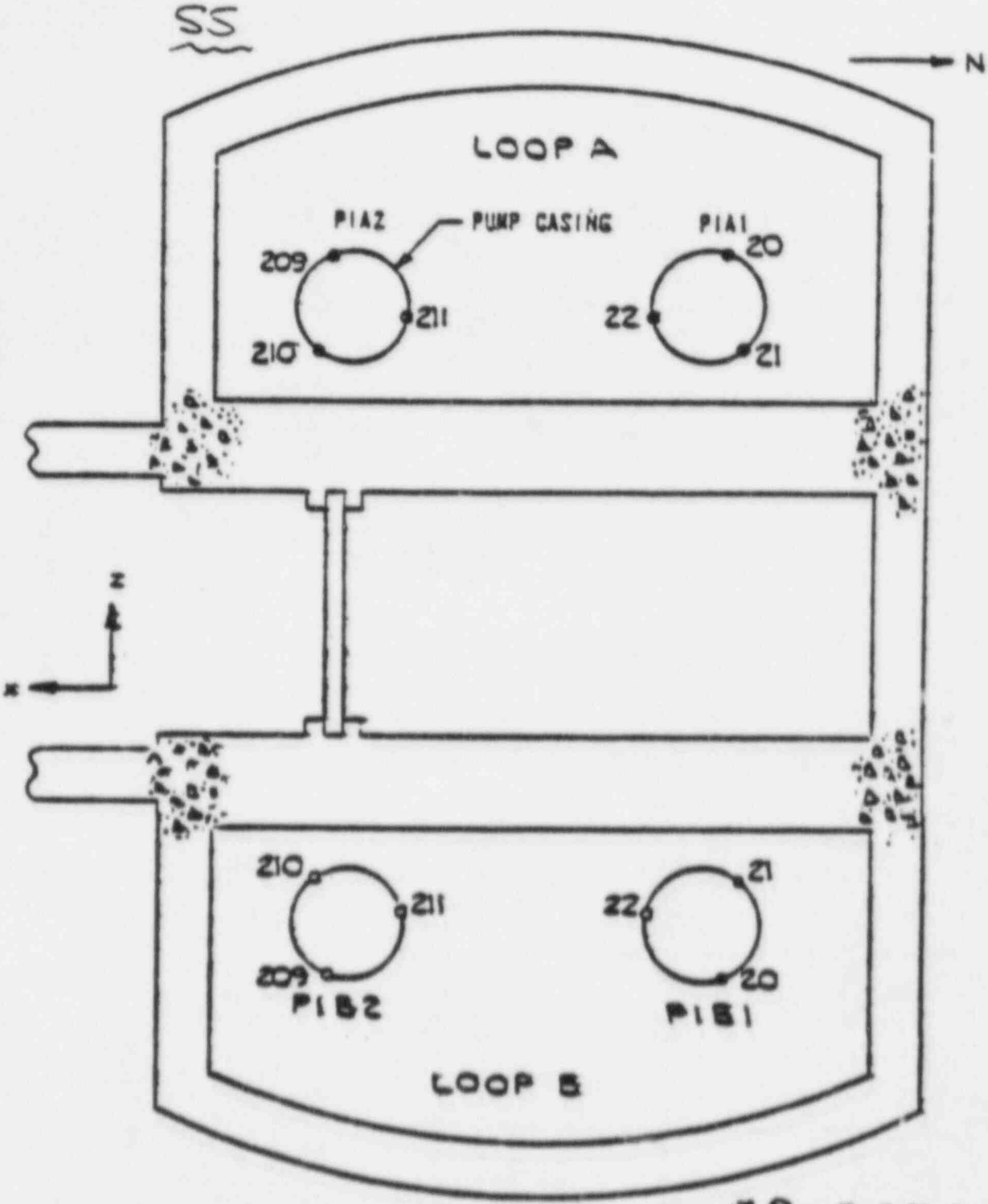


REACTOR COOLANT PUMP HORIZONTAL SUPPORTS AT ELEVATION 442'-11 $\frac{1}{8}$"

FIGURE 6C



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REACTOR COOLANT PUMP VERTICAL
SUPPORT @ EL 443'-1½"
FIGURE 6D

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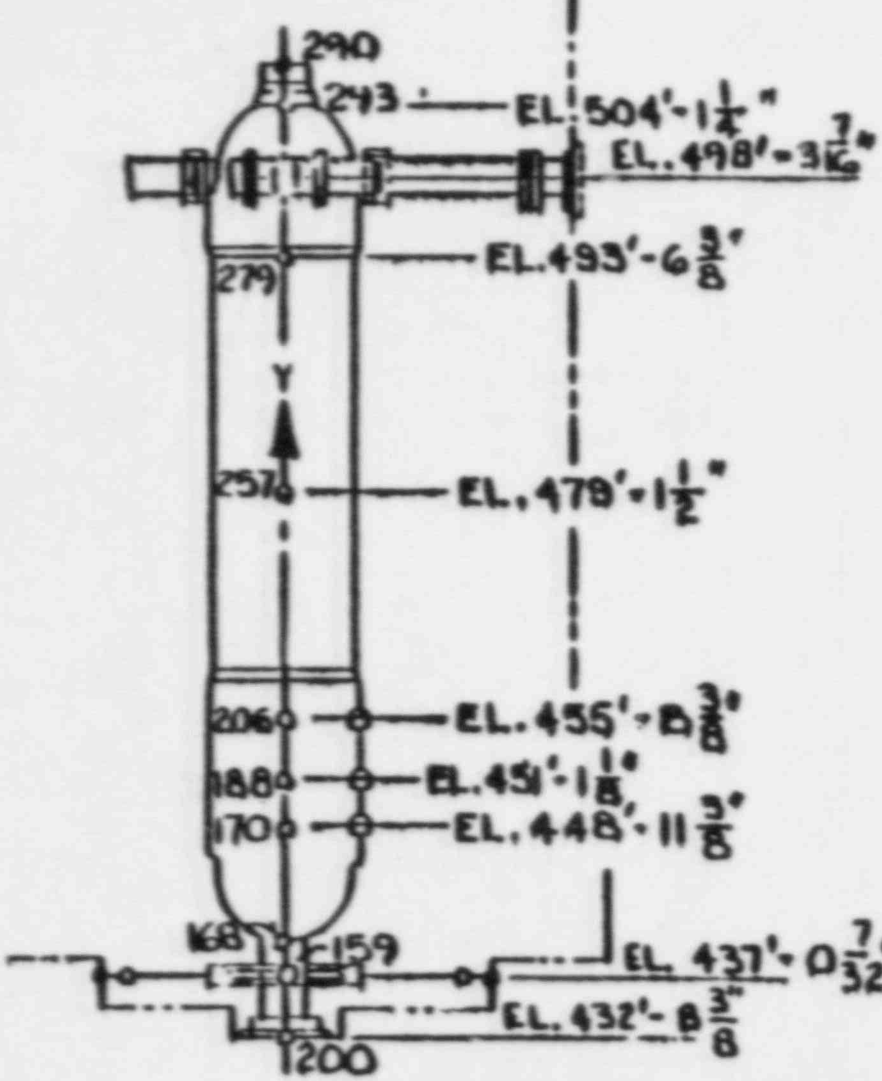
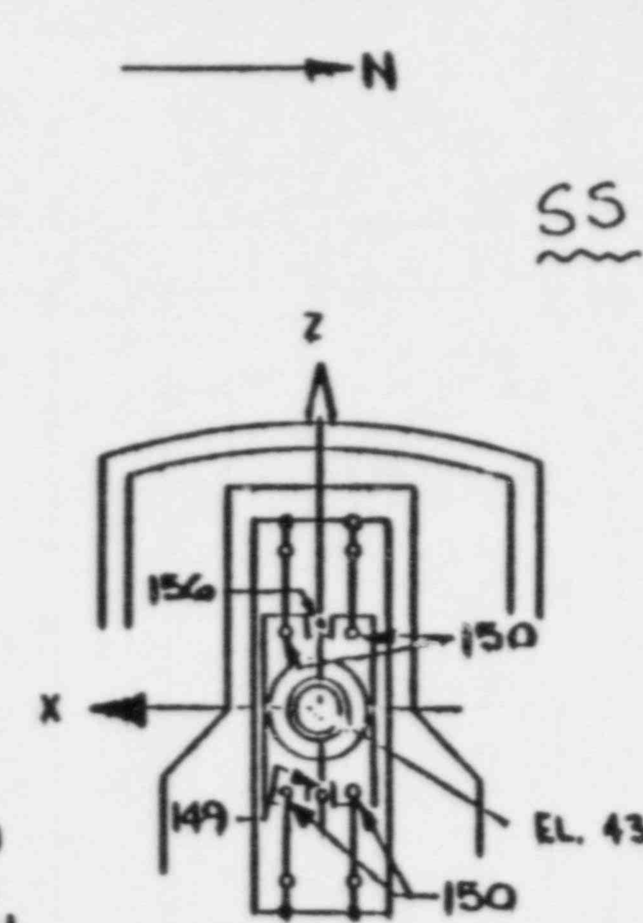


ELEVATION VIEW



PLAN @ EL., 437'-0 $\frac{7}{32}$"

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STEAM GENERATOR SUPPORTS

FIGURE 3A

SS

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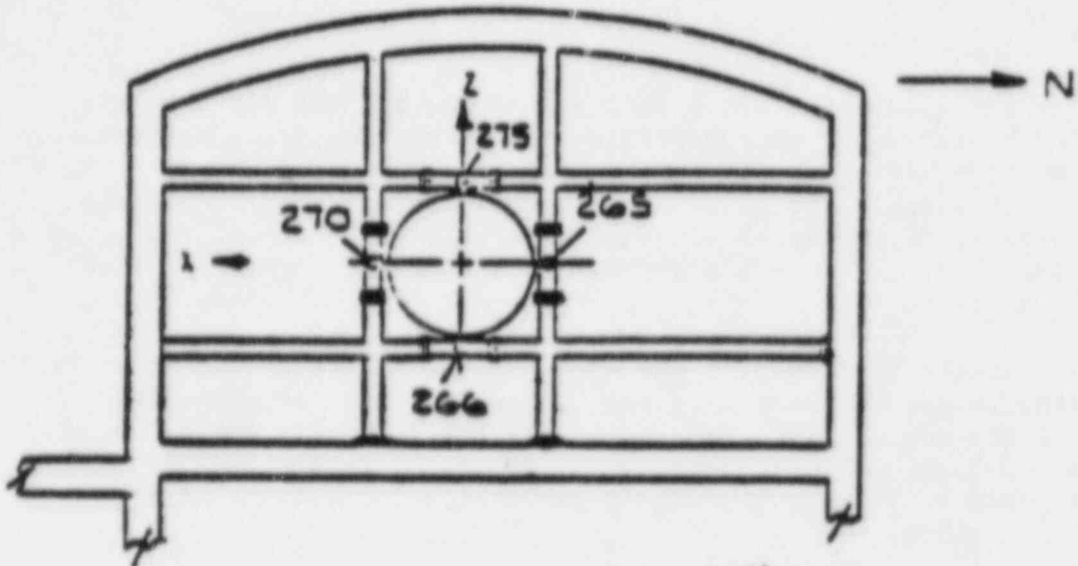

PLAN VIEW AT EL. 498'-3 7/16"

STEAM GENERATOR UPPER SUPPORTS
FIGURE 3B

3.7 SEISMIC DESIGN

3.7.1 Seismic Input

This subsection contains a discussion of the input criteria used for seismic design of the plant. Items included in the discussion are design response spectra and the basis for their selection, earthquake time-motion records and the basis for their selection, response spectra obtained from time-motion records, percentages of critical damping used for seismic analysis, and the description of soil supporting Category I Structures.

3.7.1.1 Design Response Spectra

Figure 3.7-1 gives the free-field Design Response Spectra for the Safe Shutdown Earthquake which is applicable to the two horizontal motions at right angles to one another and having peak horizontal ground acceleration of 0.25g. The corresponding free-field vertical response spectra appear in Figure 3.7-2 and also have peak ground acceleration of 0.25g. These spectra follow the provisions of NRC Regulatory Guide 1.60, Revision 1.

The design response spectra, for the horizontal and vertical motions corresponding to the OBE were obtained by dividing the corresponding ordinates of the design response spectra for the SSE by 2, and are shown in Figures 3.7-3 and 3.7-4 respectively. The response spectra for the SSE and OBE were applied at the elevations of the foundations of the Category I Structures.

There were no actual strong motion earthquake records for the site. Hence, the spectra were based upon the general recommendation of the NRC Regulatory Guide 1.60. If an earthquake corresponding to these spectra were to occur, its expected durations of strong shaking was considered to be about 22 seconds[4].

3.7.1.2 Design Time History

Artificial time-history records, corresponding to two horizontal and one vertical motion due to the Safe Shutdown Earthquake (SSE) are shown in Figures 3.7-5, 3.7-6, and 3.7-7 respectively. The artificial time histories corresponding to OBE for horizontal and vertical motions were obtained by dividing the ordinates of the respective Figures (3.7-5, 3.7-6 and 3.7-7) by 2.

Response spectra obtained from the earthquake time history motions are shown in Figures 3.7-8 through 3.7-25 for different damping values. Included on each figure is the corresponding free-field design response spectra. The response spectra obtained from the time histories enveloped the design spectra for the period range of 0.03 seconds to 3.0 seconds. The response spectra were computed using a method based on the exact solution of the governing differential equation for a single degree of freedom oscillator with viscous damping. To insure that the

response spectra were sufficiently accurate, they were calculated at a set of discrete values of period, T, forming a geometric progression, i.e.,

T_0, $T_0 r$, $T_0 r^2$......$T_0 r^{n-1}$

Ratio r = 1.03

Initial Period T_0 = 0.03 seconds

This ratio corresponds to a period interval varying from 0.0009 seconds at a period of 0.03 seconds to a period interval of 0.015 seconds at a period of 0.50 seconds. The procedure utilized to develop the time-history for use at the base of the soil-structure interaction system is described in Subsection 3.7.2.

3.7.1.3 Critical Damping Values

3.7.1.3.1 Critical Damping Values for Structures, Systems and Components Other Than NSSS Components

The damping values determined from laboratory tests for the sand, clay and silt soils under the site are given in Figures 2.5.4-13 and 2.5.4-14 as a function of shear strain. The range of damping ratios proposed by Seed and Idriss[3] for similar soils are also shown by the curves drawn on these figures. The laboratory data were noted to be near or above the upper ranges proposed by Seed and Idriss. While these values of damping were reasonably consistent with the Seed and Idriss curves at the higher levels of strain, (particularly in the sand), there was greater disparity at the lower levels of strain. For this site, it was considered that the average range of damping ratios given by Seed and Idris would provide a satisfactorily conservative design. On this basis, for the maximum shear strains of 10^{-2} to 10^{-1} percent computed for this site under the SSE, the equivalent viscous damping values for the sands range from 6 to 15 percent of critical damping. For the silts and clays, the range is lower, being approximately 5 to 10 percent.

The very dense gravel soil below elevation 380 to 390 feet (MSL) is rock-like in character, as noted elsewhere (See Subsection 2.5.4.1). It has very high wave velocities and shear moduli, comparable to those exhibited by rock. Therefore the gravel at the site was given damping values that are typical of rock as shown in Figure 3.7-84.

The percentage of critical viscous damping used for the seismic analysis of Category I Structures, systems and components other than NSSS components and soil were based on recommendations presented in NRC Regulatory Guide 1.61 including the deviations as discussed in Section 1.8 (Regulatory Guide 1.61). These percentages, which account for stress level as well as type of construction or fabrication, are summarized in Table 3.7-1. Table 3.7-2 provides categorization of NSSS components for the purpose of assigning damping values.

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3.7.1.3.2 Critical Damping Values For NSSS Components

The percentages of critical damping values for the spectral analysis of components are given in Table 3.7-1. These values are compatible with the recommendations of NRC Regulatory Guide 1.61, October 1973. The categorization of components for purposes of assigning damping values is given in Table 3.7-2.

When piping and components are included in the same dynamic model, the response spectra curves for all critical damping values are used in conjunction with the composite modal weighted damping. For a given eigenvalue or frequency, the composite modal weighted damping is computed in equation (1) and the required acceleration is used from the response spectra curves.

Alternative methods for calculating composite damping are presented in equations (2) and (3).[1],[2]

$$\beta_k = \frac{\sum_{i=1}^{N} |\phi_{ik}| \beta_i}{\sum_{i=1}^{N} |\phi_{ik}|} \tag{1}$$

$$\beta_k = \frac{\{\phi\}_k^T [\beta] \cdot [M] \{\phi\}_k}{\{\phi\}_k^T [M] \{\phi\}_k} \tag{2}$$

$$\beta_k = \frac{\{\phi\}_k^T [\beta] [K] \{\phi\}_k}{\{\phi\}_k^T [K] \{\phi\}_k} \tag{3}$$

where N = Number of components

$\{\}^T$ = Transpose of $\{\}$

β_k = Composite damping for kth mode

β_i = Percentage critical damping associated with component i

$[\beta]$ = System damping matrix for N components

$\phi ik = \{\phi\}_k$ = mode shape vector for kth mode,

ϕik = absolute value of kth mode shape,

$[M]$ = system mass matrix,

$[K]$ = system stiffness matrix.

Seismic analyses have been performed on a suitable structural model to verify that composite damping values calculated according to equation (1) yield more conservative results than those calculated from equations (2) and (3). The structural model used was a typical RCS including the reactor vessel, pressurizer, steam generators, reactor coolant pumps, and reactor coolant piping. In this model, the minimum component damping used was 2 percent and the maximum was 48 percent. The results of the comparison are presented in Table 3.7-3. Note that the results for equations (2) and (3) are the same. This is due to the fact that

$$w^2 \{\phi\}^T [M] \{\phi\} = \{\phi\}^T [K] \{\phi\}$$

where w is the natural frequency. The data in Table 3.7-3 clearly illustrates that the overall results from equation (1), used by B&W for WNP-1/4 are conservative in comparison with those from equations (2) and (3).

3.7.1.4 Supporting Media for Seismic Category I Structures

Table 3.7-4 describes the foundation embedment, width of the structural foundation, total structural height, ratio of soil embedment depth to least foundation dimension for the Category I Structures.

The depth of soil from finished grade to bedrock is approximately 476 feet at WNP-1 and 445 feet at WNP-4. The soil layering characteristics, shear wave velocity, shear modulus, and soil density are discussed in Subsection 2.5.4.2.

3.7.2 Seismic System Analysis

This subsection contains a discussion of the seismic analyses performed for Category I structures and systems. Included in the discussion are the methods of seismic analysis used, the criteria used for mathematically modeling the structures and systems, the assumptions made in the analyses, and the effects considered.

3.7.2.1 Seismic Analysis Methods

The seismic response of Category I structures, components, and systems has been determined from a suitable elastic dynamic analysis. The results of these analyses are used for the design of Category I structures, components, and systems. These analyses also furnished input for use in subsequent subsystem dynamic analysis.

The combined effect of the two horizontal and one vertical motions was taken into account to obtain the design parameters. The three seismic responses or effects at a particular point caused by each of the three orthogonal

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components of seismic motion were combined by taking the square root of the sum of the squares of the particular effect or response at that point in accordance with NRC Regulatory Guide 1.92.

The seismic system analyses were performed using time-history modal analysis method.

3.7.2.1.1 Seismic Analysis Methods for Category I Structures

The time history method was used to determine the dynamic response of the structures and to obtain in-structure response spectra. The mathematical models used for the seismic analysis of Category I structures typically consisted of lumped masses connected by linear elastic springs. In this manner, each structure was described by a finite number of degrees-of-freedom chosen to represent the principal behavior of the system. The mass of the structure was assumed to be concentrated at particular locations on the model. These locations were considered at floor levels, and at other levels where the dynamic response was required for input to a subsequent subsystem analysis. Since the structure was idealized as a three-dimensional model, each mass point could be assigned up to six degrees-of-freedom; two horizontal translational degrees-of-freedom, two rotational degrees-of-freedom; a vertical degree-of-freedom, and a torsional degree-of-freedom. The corresponding spring stiffnesses accounted for the shear, flexural, torsional, and axial properties of the structure. The mass properties with appropriate eccentricity and mass moment of inertia properties were assigned at each mass point. An adequate number of masses or degrees-of-freedom were included in the analysis by considering total degrees-of-freedom to exceed twice the number of modes with frequencies less than 33 cps. The effects due to inertial characteristics of the fluid confined within the structural component were considered in the analysis. A sufficient number of modes were included in the analysis to account for the participation of all significant modes.

The nodal acceleration and displacements obtained from dynamic analysis of structures were used for the design of structures, systems and components. The displacements were used to compute additional stresses in interconnected components as discussed in Subsection 3.7.3.8.

Soil-structure interaction was included in the analysis as discussed in Subsection 3.7.2.4. The lumped mass mathematical models of the Containment and the GSB used for horizontal and vertical seismic analyses are shown in Figure 3.7-27 through 3.7-31. Figures 3.7-107 and 3.7-108 show the physical structure and the lumped mass soil spring mathematical model of the addition to the Air Intake Structures. The procedure used for modeling these structures is discussed in Subsection 3.7.2.3.

3.7.2.1.2 Seismic Analysis Methods for NSSS

Table 3.7-24 gives the methods of seismic analysis used for B&W Seismic Category I equipment.

A three dimensional seismic analysis was performed on the reactor coolant loop components to determine piping and component loading as outlined in topical report BAW-10131, "Reactor Coolant System Structural Loading Analysis" using rotational as well as translational response spectra to represent the characteristics of the soft-soil site properly. An idealized mathematical model was used. It represents a single loop that includes the reactor, one steam generator, the pressurizer, two coolant pumps with associated piping, and the secondary shield wall with attached linear elastic restraints or supports (see Figures 3.7-32 through 3.7-45). The modeling technique involves the use of lumped masses connected by elastic members. These lumped masses include the weight of the component, contained fluid, internal structures, if any, and any external attachments to the component. In order to represent each component with a sufficient number of masses, each component was initially modeled on an isolated basis (e.g., reactor vessel, OTSG, hot leg, etc.). For these models sufficient degrees of freedom were included to yield very accurate frequencies and mode shapes for the seismically excitable frequencies, i.e., less than 33 Hz. The number of degres of freedom for each model was then successively reduced to a point where the reduced model's mode shapes and lower frequencies differed by less than 10 percent from those of the more complex model. The reduced model that retained accuracy within the 10 percent error range was then used in the total system analysis.

The graphs of Figure 3.7-46 through 3.7-49 represent the results of a typical study done on the hot leg. Note that the modes presented cover a frequency range from 5.5 to 12.5 Hz. Also note that if 6 or more masses are used, the frequency variation is within the required 10% error range. The Supply System model used 9 mass joints in the hot leg. After the number of masses was determined and the model complete, the elastic properties of the piping and components were used to calculate a reduced flexibility matrix. All flexibility calculations included the effects of torsional, shearing, bending, and axial deformations as well as changes in flexibility due to curved members and internal pressure. Flexibility factors were calculated in accordance with ASME III, NB-3687. These flexibility factors were then used to calculate the frequencies and mode shapes of the system. Seismic response spectra, providing excitation input at the base mat, was then used to find the effective inertial forces and moments resulting from an earthquake excitation. Finally, modal forces and moments were calculated with the use of the inertial forces and moments. Regarding modal participation, all components participate at whatever frequency of the system that was considered; however, the degree of participation was different for various components. For each frequency, the normalized mode shapes (deflection of each mass point with respect to the maximum deflection) and participation factors were computed. The analysis included 50 modes (approximately 45 Hz) which was sufficient to assure participation of all significant modes. The inclusion of additional modes above the 50 basic ones results in a negligible increase in combined resultant loadings. The modal combination was in accordance with Regulatory Guide 1.92.

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3.7.2.4 Soil-Structure Interaction

3.7.2.4.1 Finite Element Analysis

Finite element analysis was used to investigate the interaction between the soil and the Containment and GSB. The computer program LUSH, described in Section 3.8, Appendix 3.8A was used for this analysis.

The coupled mathematical models were constructed using plane strain quadrilateral and triangular elements for the structures and the soil. The equivalent plane strain models for the structures were obtained to adequately represent the vibrational characteristics of the actual structures. Important parameters such as cut-off frequency, soil element size and number of frequency interpolations in the complex frequency response method were determined by parametric studies of the soil column.

The soil model also consisted of a quadrilateral and triangular finite element mesh in which the elements were given strain dependent properties (modulus and damping) that were typical for each of the various soil layers within the subsurface profile at the site. The base of soil layers was established at a depth of 139 feet below the foundation level. The control motion (the time history of ground acceleration), equivalent to 0.25g for SSE and 0.125g for OBE, was applied at the foundation level of the structures. The motions below 139 feet are not affected by the presence of the structures when the control motion is specified at a level considerably above the bottom boundary of the combined soil-structure model. The top of soil profile reflects ten feet of fill that was placed above the existing soil to bring the area up to the plant grade. The side boundaries of the soil model were considered vertical and were located far away from the sides of the structures in order to eliminate errors in the seismic wave propagation. The free field control elevation response spectra at the side boundaries were found to compare reasonably well with the free field input motion spectra as shown in Figures 3.7-82 and 3.7-83. The strain dependent dynamic soil properties are shown in Figures 3.7-84 and 3.7-85. The variation of soil properties was explicitly included in the analysis. Non-linear soil-structure interaction analysis was performed using an equivalent linear method and complex frequency response procedure described in LUSH.

The general method of approach consisted of the specification of the control motion at a depth in the free field. The depth was chosen to correspond to the lowest grade of the foundation mat of the structures being analyzed. As a first step, it was necessary to determine the motions which would have to develop in an underlying rock-like formation in order to produce the specified motions at the control point. This was accomplished by means of a wave propagation analysis of soil column in the free field using appropriate computer program such as SHAKE, described in Section 3.8, Appendix 3.8A. The generalized soil deposit model used for the deconvolution analysis to derive base motion time histories is shown in Figure 3.7-86. The response spectra at the grade elevation and the response spectra of the deconvolved motion at the soil base for SSE are shown in Figure 3.7-83.

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Several soil-structure models were analyzed to approximately account for multi-directional earthquake and structure-structure interaction between significant structures. The locations of various sections considered for the analysis are shown in Figure 3.7-87. A typical model of the soil-structure system in north-south direction is shown in Figures 3.7-88 and 3.7-89.

The base motion obtained from SHAKE analysis of soil column was subsequently used as an excitation for a two-dimensional analysis (plane strain) of the soil-structure system, leading to an evaluation of the motions at any selected point such as the base of a structure, the operating floor of a structure, etc. The interface motions at the base of structures were obtained for the Safe Shutdown and Operating Basis Earthquakes for two horizontal orthogonal directions (north-south and east-west) and a vertical direction.

A comparison of the free-field peak acceleration profile derived by SHAKE and LUSH analysis for SSE conditions is shown in Figure 3.7-90.

The detailed fixed-base lumped mass models of structures were then subjected to respective interface motions to determine structural responses significant for the structure and component design. Since the structure models, included in the two-dimensional soil-structure interaction analysis (plane strain) were dynamically equivalent to the detailed lumped-mass models of the structures, the response at critical locations obtained from both the analyses provided a check on the results of the analyses. The dynamic equivalence of the structures was considered established when the same dynamic characteristics such as frequencies, mode shapes and response were obtained from both analyses. The peak acceleration values for the Containment, crane wall, GSB and soil are shown in Figures 3.7-91 and 3.7-92. The response spectra envelope at selected elevations in the buildings are shown in Figures 3.7-54 through 3.7-81.

3.7.2.4.2 <u>Lumped Mass Soil Spring Analysis</u>

Lumped mass discrete spring models were used to determine the structural response and amplified response spectra of (1) addition to Air Intake Structure and (2) Borated and Demineralized Water Tank Enclosure. The soil spring constants were determined by using the expressions derived from the theory of a rigid base resting on the surface of an elastic half space.(10,11) Variations in the shear modulus, as determined by the site investigation and subsequent soil tests were taken into account. The seismic analysis was performed using upper and lower bound values of soil moduli. The magnitude of the soil modulus was obtained from Subsection 2.5.4, Figure 2.5-4-11A and are shown in Table 3.7-31. The floor acceleration response spectra calculated using these upper and lower bound values were smoothed so that the response curve was an upper bound envelope of all spectral acceleration points. Thus, the smooth response spectra were conservative and considered all possible variations in the soil upper and lower bound values. The peak acceleration values and the forces for the addition to Air Intake Structures are shown in

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TABLE 3.7-1

DAMPING VALUES

Item, Equipment, or Structure	Percent of Critical Damping: Operating Basis Earthquake[3]	Percent of Critical Damping: Safe Shutdown Earthquake
Equipment and large diameter piping systems,[1] pipe and diameter greater than 12 in.	2	3
Small diameter piping systems,[2] diameter less than or equal to 12 in.	1	2
Welded steel structures	2	4
Bolted steel structures	4	7
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7
Soil[4]	10	10

Notes:

1. Includes both material and structural damping. If the piping system comprises only one or two spans, with little structural damping, value for small diameter piping, are used.

2. Assumed damping is composed primarily of material damping with negligible system damping.

3. These values are also used for SSE in the dynamic analysis of active components as defined in NRC Regulatory Guide 1.48.

4. Assumed when soil - spring models were used for the analysis. Varies in use of a finite element model. (Strain dependant damping is used for finite element analysis as discussed in Section 3.7.1.3.1).

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The requirements to design nuclear power plants for the effects of an instantaneous double-ended guillotine break (DEGB) of the reactor coolant loop (RCL) piping have led to excessive design costs, interference with 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 demonstrating that 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 estimates the probability of indirect DEGB in RCL piping as a consequence of seismic-induced structural failures within the containment of Babcock & Wilcox supplied pressurized water reactor nuclear power plants in the United States. The median probability of indirect DEGB was estimated to range between 6×10^{-11} and 1×10^{-7} per year. Using very conservative assumptions, the 90% subjective probability value (confidence) of P_{DEGB} was found to be less than 1×10^{-5} per year.

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