APPENDIX B

REVIEW OF THE
MILLSTONE UNIT 3 PROBABILISTIC SAFETY STUDY
SEISMIC FRAGILITY, WIND, AND EXTERNAL FLOODING

by

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1.0 INTRODUCTION

Jack R. Benjamin and Associates, Inc. (JBA) was retained by Lawrence Livermore National Laboratories (LLNL) to perform a review of the Millstone Unit 3 Probabilistic Safety Study (referred to as the Millstone PSS), which was prepared by Northeast Utilities Service Company (NUSCO), August 1983. The areas reviewed included seismic fragility, wind, and external flooding. The scope of the review is discussed in the next section. The final section in this chapter discusses the overall methodology used to develop the seismic fragility data and the bases for excluding wind and external flooding. Chapters 2, 3, and 4 present the review of seismic fragility, wind, and external flooding, respectively. Finally, Chapter 5 gives conclusions and recommendations based on the review.

1.1 SCOPE

The review of the Millstone PSS focused on the following report sections which document the seismic fragility, wind, and external flooding analyses:

- Section 1.2.3 External Flooding
- Section 1.2.5 Wind
- Section 2.5.1 External Event Analysis
- Appendix 2-I Millstone Unit 3 Seismic Analysis--Structures and Equipment
- Appendix 2-J Millstone Unit 3 Probabilistic Analysis of Structural and Component Fragilities

Jack R. Benjamin and Associates, Inc. has performed similar reviews of the Indian Point Probabilistic Safety Study (IPPSS) (Ref. 1) and the Zion Probabilistic Safety Study (ZPSS) (Ref. 2). (See Reference 3 for the IPPSS review. The Zion review has not been published.) Based on experience gained from the IPPSS and ZPSS reviews, the review of the Millstone PSS was conducted in a short time period in order to quickly evaluate the adequacy and accuracy of the results and to make recommendations based on the findings. In contrast to the previous reviews which consisted of an in-depth evaluation of each section and subsection of the PRA report, this review focused on critical areas which impact the results.

Dr. John W. Reed performed the review of the Millstone PSS. Dr. Martin W. McCann, Jr. assisted in the review of the external flooding hazard. One man-month of effort was devoted to the review with approximately two days each spent on the wind and flood sections with the remaining time devoted to the seismic fragility analysis.

During the review, a meeting was held with NUSCO to discuss the findings. It was learned that a reanalysis of the seismic hazard and fragility parts of the Millstone PSS is currently being conducted. The reanalysis has not been reviewed and comments in this report are made only for the information given in the Millstone PSS which was submitted in August 1983. A tour of the plant site was conducted at the end of the review. In comparison with other plants, the Millstone 3 structures and components appear to be properly constructed and supported from a structural viewpoint. The construction details and appearance of the plant, support the comments and conclusions made in this report.

It was assumed in the review that LLNL would be responsible for evaluating the seismic hazard analysis and the systems analysis (i.e., event trees, fault trees, and hazard/fragility integration). Since the overall seismic analysis was not reviewed, the impact of the findings are discussed in terms of the various component seismic fragilities. Note that it was concluded in the Millstone PSS that wind and external flooding are not significant hazards, hence no formal probabilistic analysis was conducted.

1.2 OVERALL METHODOLOGY

The overall methodology used to develop the seismic fragility data and the bases for excluding wind and external flooding are discussed below.

1.2.1 Seismic Fragility

The methodology used in the Millstone PSS to develop seismic fragility data is appropriate and adequate to obtain a rational measure of the probability distribution on the frequency of failure. However, the application of the methodology is a concern. It is stated in the calculations and Appendix 2-I that References 4 and 5 were used as the basis for the seismic fragility analysis. The approach used in the Millstone PSS is referred to in the PRA Procedures Guide as the "Zion" method (Ref. 6). However, the method used was not named or justified as appropriate in the Millstone PSS.

The Zion method has two important features. First, the methodology is based on a double lognormal distribution model. Both the distribution on the median and the random variation of frequency of failure are assumed to be lognormal. Secondly, the probabilistic analyses use the results from the original design analysis as the basis for the seismic fragility estimate. The median fragility values are obtained using the responses and capacities from the design analyses which are scaled to eliminate conservatisms and variabilities (i.e., randomness and uncertainty) and are estimated based on some data, but mostly on engineering judgment.

It is interesting to note that nowhere in the Millstone PSS report sections pertaining to the seismic fragility analysis was the word "lognormal" used. Thus, no defense is given why the lognormal model is appropriate. As discussed in Chapter 2 of this report, there is considerable confusion since the logarithmic standard deviations reported in Appendix 2-I (note they are referred to as randomness and uncertainty variabilities or beta values) for structures and equipment are incorrectly interpreted as standard deviations. Also, the lognormal distribution is converted to a Weibull distribution with a lower bound cutoff at 0.17g. This is philosophically inconsistent since the fragility analysis incorrectly assumed that the variabilities of the various capacity and response parameters are lognormal. It is also not clear that 0.17g is the proper cutoff point. Since potential design and construction discrepancies were not considered in the analysis, the use of a lower-bound cutoff is difficult to defend.

As discussed in Chapter 2, many errors in the seismic fragility analysis were found. In general, the assumptions which were made are on the conservative side. The final conclusion is that the analysis is not rational and the frequencies of failure are too conservative. This is part of the reason why the mean frequency of core melt is 9.4×10^{-5}, which is high relative to the results from other PRAs.

From Section 7.5 (i.e., Table 7.5.1-2) of the Millstone PSS, the mean systems fragility values for core melt were obtained and listed in Table 1-1. As can be seen from Table 1-1, the frequency of failure is very high with a median value approximately equal to 0.3g. Even close to the SSE value of 0.17g, the frequency of failure is on the order of 1 in 10. This is not reasonable since Millstone is a newer plant, which has been designed to comply with more recent regulations.

In contrast to other PRAs submitted to the USNRC to date, the Millstone PSS reflects new response analyses based on simple single-degree-of-freedom (SDOF) models for the buildings. This was done to rationally evaluate the conservatisms in the floor response spectrum used in the design analyses. This is commendable; however, it is not clear from the calculations whether the effects of the higher frequency building modes have been properly accounted for in the SDOF analyses.

A troublesome concern is the question of secondary nonsafety-related components failing and falling on safety-related equipment.

This issue is not addressed. Also, it is not clear why the Main Steam Valve building was not considered in the fragility analysis, since it is a safety-related structure.

The same general philosophical concerns from past PRA studies, which were based on the Zion method, also apply to the Millstone PSS. Reference 3 discusses these issues in depth based on the review of the IPPSS. The reader is directed to Section 2 of Appendix A of Reference 3 for a general discussion of these issues.

1.2.2 Wind

It is concluded in the Millstone PSS that wind effects do not contribute significantly to the risk. As discussed in Chapter 3.0, this conclusion is reasonable. The argument for excluding wind is based on the hazard at the site and the protection provided by the two-foot-thick concrete walls and roof elements which form the safety-related structures. No fragility curves were developed and a systems analysis was not performed.

1.2.3 External Flooding

Similar to wind, it is argued in the Millstone PSS that external flooding is not a significant hazard to the risk at Millstone. As discussed in Chapter 4, the arguments given do not provide a rational basis for excluding flood. Since external flooding was eliminated, no fragility curves were developed and a systems analysis was not performed.

Table 1-1. Mean Core Melt Systems Fragility Values

Ground Acceleration	Mean Frequency of Failure
0.185	0.087
0.25	0.354
0.35	0.706
0.45	0.886
0.55	0.958
0.65	0.993
0.75	0.999
0.80	1.000

2.0 SEISMIC FRAGILITY

The review of the seismic PRA analysis focused on the following sections of the Millstone PSS report:

Section 2.5.1	Seismic Risk Analysis
Appendix 2-I	Millstone Unit 3 Seismic Analysis--Structures and Equipment
Appendix 2-J	Millstone Unit 3 Probabilistic Analysis of Structural and Component Fragilities

In addition to the review of these sections, the calculations for selected structures and components which are significant contributors to the frequency of core melt were evaluated. The steam generator was also included in the review since it was given as a representative example in Appendix 2-I.

Section 2.1 presents comments on specific sections of the report pertaining to the seismic fragility analysis. Section 2.2 gives the results of the review of the fragility calculations for selected items. Finally, Section 2.3 closes the seismic review and gives an estimate of the general level of conservatism which is contained in the Millstone PSS fragility analysis.

2.1 REPORT SECTIONS

The following comments are directed to specific sections of the PSS. Page numbers precede each comment to help the reader locate the area of concern. Specific comments on the calculations of structure or component fragility data are given in the next section.

Section 2.5.1 Seismic Risk Analysis

Page 2.5-2: It is stated that failure is assumed to occur if allowable load limits established by design codes or functional tests are exceeded. This is an overly conservative assumption; however, the fragility calculations in Appendix 2-I did not adhere to this constraint, but rather included the energy-absorption capacity beyond the yield limit, if appropriate.

Page 2.5-3: β_R and β_U are described as being standard deviations. In fact, they are logarithmic standard deviations. This philosophical error is discussed in more detail below (see comments on Appendix 2-J).

Page 2.5-5: It is assumed that the safety-related components designed to the SSE level will not fail (i.e., probably equal to 1.0) for accelerations below 0.17g. As concluded below, the fragility curves are conservative, and the equivalent median frequency of the safety-related systems, which is between 0.25g and 0.35g, should be roughly a factor of two to three higher. For higher capacities, this cutoff assumption is not critical. However, for the analyses documented in the PSS, accelerations below 0.17g will contribute noticeably to the mean frequency of core melt. As discussed in Section 1.2, the lower bound cutoff value of 0.17g has not been justified.

Page 2.5-6: Components above 1.11g were excluded from the fault tree since their capacity is sufficiently high such that they do not contribute significantly to the results. However, if higher capacities are justified as discussed below, then some of the excluded components may become significant contributors.

Page 2.5-10. Terminating the analysis at 0.8g is inconsequential for the low structural capacities developed in the PSS since the mean system fragility curve corresponds to a frequency of failure of 1.0 at 0.8g (see Table 1-1 in Chapter 1). However, if higher capacities are used as discussed below, then the upper bound acceleration cutoff should be justified on the basis of the maximum earthquake intensities which can occur.

Page 2.5-11: The assumption that incipient sliding leads directly to failure of interconnecting piping is very conservative.

Page 2.5-12: The basis for the comment that structural sliding does not affect interconnecting piping and cabling is not true. However, the incipient sliding condition assumed in the PRA is very conservative (see comment directly above).

Appendix 2-I - Millstone Unit 3 Seismic Analysis--Structures and Equipment

Page 2-I-6: It is not clear why the variabilities were reduced on a selected basic at the end of the project.

Page 2-I-24: It is stated that as part of the inelastic energy absorption capacity, the redistribution of forces among structural elements was considered. For the component calculations reviewed (see Section 2.2), no allowance for force redistribution was found.

Page 2-I-26: The definition of seismic fragility as ". . .level of effective median peak ground acceleration at which the structure would cease (fail) . . ." is incorrect. The median value is a parameter of the lognormal distribution on capacity, not the entire fragility curve of a structure.

Page 2-I-26: The sliding capacity as calculated is the incipient sliding capacity and is conservative. As discussed below (see Section 2.2), the amount of sliding displacement and its effects on interconnecting equipment is the critical issue, not the level of ground motion at which sliding begins.

Page 2-I-28: FCSM should be the ratio of the strength, computed with the actual material properties, divided by the strength, computed with the specified material properties.

Page 2-I-30: The simple inelastic energy model used in the PRA analysis is applicable to structures which can be modeled as single-degree-of-freedom systems. This model has not been shown to apply to complex structures. At best the uncertainty is a function of the complexity of the system being analyzed.

Page 2-I-30: The average factor of safety due to earthquake combination (i.e., 1.3) where the responses from the three directions were combined using an absolute sum, is a gross approximation. A specific value should be determined for each structure. For example, using data from the FSAR it can be shown that values of 1.2 and 1.4 are more appropriate for the interior concrete and exterior shell of the containment building, respectively. However, it was later determined that the seismic forces were actually combined by the SRSS method in the PRA calculations for the Containment internal structure. See discussion in Section 2.2 for the Containment crane wall failure.

Page 2-I-35: The use of different soil-structure interaction safety factors for the EGEB (i.e., for sliding and strength failure modes) is inconsistent. The values should be the same.

Page 2-I-36: The variability in the ultimate strength prediction is also due to uncertainty in the simple models used in the PRA analysis.

Page 2-I-37: The variability for the design capacity factor should be based on the strength equation which generally is different for each structure. A range of ±20 percent (COV) is a gross approximation which should be determined specifically for each structure.

Page 2-I-37: The material strength factor variability is not equal to the variability in material strength. It is equal to the variability of the strength model due to variability in the parameters in the model such as material properties.

Page 2-I-39: The statement: "The randomness also was considered to contain the uncertainty of the mean" is philosophically wrong. By definition, randomness and uncertainty are mutually exclusive.

Page 2-I-39: The modeling factor variability should be based on variability of the modal frequencies and mode shapes. The frequency effect should be obtained using the median ground response spectrum at the median damping value. Variability in frequency should be transformed using the response spectrum to variability in response. The use of ±15 percent (COV) is a gross approximation and should be determined specifically for each structure.

Page 2-I-40: The basis for establishing the earthquake combination factor variability is not rational. Some simple calculations for the Limerick PRA (Ref. 7) show that the randomness logarithmic standard deviation varies from 0 to 0.16 depending on the relative responses from the three components and the coupling between the responses. Thus the value of 0.22 is on the high side.

Pages 2-I-41 to 44: See Section 2.2 for a discussion of the calculations performed for selected structures.

Page 2-I-58: It is assumed that electrical relay unrecoverable chatter is a failure. It is not clear how this was used in the PRA analysis. Also, using incipient sliding of the connecting buildings as a failure mode for buried piping is very conservative.

Page 2-I-61: Single-degree-of-freedom models were used to obtain the modified ARS for median damping values (i.e., 10 percent for structure and 5 percent for equipment). It is not clear whether the effect of higher frequency modes was properly included in the modified floor response spectra.

Page 2-I-63: The report states that the variability for the equipment design capacity factor for piping was based on the ratio of design pressure to operating pressure calculated for each representative pipe element. This is not rational. The variability should be calculated from the strength equation using the variabilities for the model parameters (i.e., material properties).

Page 2-I-64: Equation 4-4 is for a lognormal distribution, not a normal distribution as stated. This is an example of the misunderstanding of what was calculated and reported in Appendix 2-I.

Page 2-I-65: The generic damping factor variability value of 0.09 may be low for equipment with natural frequencies close to the lower frequencies of the supporting structure. When the frequencies of the equipment and structure coincide, small differences in equipment damping can cause large differences in response.

Page 2-I-65: Modeling factor variability includes contributions from mode shape and frequency. The latter effect is sensitive to the proximity of the equipment frequency value to the peak of the floor response spectra. Component-specific values for this parameter should be developed.

Page 2-I-65: The earthquake component combination variability is a function of the relative contributions from the three earthquake components and the degree of coupling which exists. The frequency of the equipment (i.e., flexible versus rigid) is not a significant variable for this type of variability. The logarithmic standard deviation values are typically between 0 and 0.16.

Page 2-I-66: It is assumed that the earthquake component combination factor variability is the same as the structural response factor variability for the structure in which the component is located. This is not rational. This factor should be based on the characteristics of the equipment, not the structure as discussed above.

Page 2-I-72: The modal combination factor, FRSC, should not be included in the structure response factor when determining the fragility for an equipment item, since the floor response spectra were obtained from a time history analysis (i.e., since the phasing information is properly incorporated). Also, the earthquake combination factor, FRSE, should not be included since the effects of random phasing between the earthquake components should only be included in the equipment response factor.

Page 2-I-76: The illustrative example given in Appendix 2-I, Section 5.2 is discussed as part of the review of the calculations for significant components in Section 2.2, below.

Page 2-I(A)-2: As discussed above for page 2-I-61, it is not clear that the higher frequency modes of the buildings were properly reflected in the modified ARS development.

From the calculations it appears that the equipment spectral shape factor was calculated properly. The safety factor for equipment damping was obtained from a ratio of the relative floor responses (i.e., the median level case vs. the design level case) where the relative floor responses for each case were obtained as the spectral ordinate divided by the zero period acceleration value. This approach is reasonable; however, it is not clear that the effect of higher frequency modes of the structures was properly incorporated.

Also, as discussed below in Section 2.2, the approach used for scaling the structure response factor with height is incorrect.

Appendix 2-J - Millstone Unit 3 Probabilistic Analysis of Structural and Component Fragilities

Page 2-J-2: It is obvious from the use of the fragility parameters produced in Appendix 2-J that the authors of this appendix did not realize that a median value and two logarithmic standard deviations were developed for eacn structure or component in Appendix 2-I. It was erroneously assumed that β_R and β_U are standard deviations (in fact, they are logarithmic standard deviations). The conversion from a lognormal distribution to a Weibull distribution is incorrectly performed. By assuming that the betas from Appendix 2-I are standard deviations the variabilities are increased substantially. The final result is that the fragility curves used in the analysis are overly conservative.

2.2 CALCULATIONS

The calculations for a select group of structures and components were reviewed. The following items were chosen because they are significant contributions to the frequency of core melt. The steam generator was also reviewed since it is the single illustrative example that is presented in Appendix 2-I of the Millstone PSS report.

Structure/Component	Median Capacity (g)*
Steam Generator	2.28
Auxiliary Building (collapse)	0.55
125 VDC Distribution Panel	0.64
Demineralized Water Storage Tank	0.68
Reactor Core Geometry	0.68
Service Water Piping	0.74
ESF Building (sliding)	0.74
Emergency Generator Enclosure (sliding)	0.75
Containment (crane wall failure)	0.87

*Listed in Table 2.5.1-3 of the Millstone PSS report.

A discussion of the calculations for each item is given below.

Steam Generator (Millstone PSS Report Section 5.2 - ILLUSTRATIVE EXAMPLE)

Numerous discrepancies were found in the illustrative example of the steam generator. The following is a discussion of the problems which were encountered.

The structure earthquake combination factor, FRSE, should be 1.0, not 1.3, since the equipment was designed using the SRSS procedure. Also, the structural response factor should not be scaled to different elevations, since as first approximation all points in the building above the base are equally affected by changes in the structural response parameters (i.e., spectral shape, damping, modeling, etc.). Hence the structural response factor should be just 1.5.

In developing the variability for the spectral shape safety factor, it was assumed that the uncertainty component was zero, which is similar to what was done in past seismic PRA studies. As discussed in Reference 3, it is believed that if this were true, there would be no motivation to ever conduct site studies to develop site-specific spectra.

The variability for modeling is assumed to be all uncertainty with a logarithmic standard deviation of 0.15. This value is consistent with other PRA studies; however, it is more rational to compute this parameter for each building considering variability in the mode shape and fundamental frequency. In the Zion and Indian Point PRA studies, the logarithm of the ratio of the spectral ordinates of the median spectrum at the median and median minus one standard deviation frequency was used to compute the uncertainty value.

The variability for structure modal combination (i.e., 0.17) should not be included since the floor response spectra were obtained using a time history analysis where all significant modes and the phasing between the modes were retained. The variability for structure earthquake component combination (i.e., 0.22) should be eliminated for the structural response factor and a smaller value included in the equipment response factor. Based on a study conducted as part of the Limerick PRA (Ref. 7), the randomness due to earthquake components is a function of the number of components which contribute to the response and the degree of coupling between the components. Values for the logarithmic standard deviations varied in the study from 0 to 0.16. Also, a small randomness component for damping would be more reasonable than assuming zero.

The safety factor for equipment response, FRE, was assumed to be 1.0. The spectral shape factor, FRES, was inappropriately assumed to be 1.0. The value for this factor is actually included as part of the capacity factor (i.e., third equipment factor equal to 1.5). However, this factor was not computed correctly since it is based on the ratio of the design ZPA value to the median ZPA value. Since the steam generators are flexible, the factor should be based on the ratio of spectral ordinates corresponding to the fundamental frequency of the component.

The variability for equipment spectral shape factor was assumed equal to the same value as for the structure spectral shape factor. The variability should have been based on the floor response spectra from the median analysis and split into randomness and uncertainty components corresponding to single mode variability and higher mode contributions, respectively. Similarly, the equipment damping variability should have been based on floor response spectra from the median analysis. The value of 0.09 in the report is equal to the variability in damping. It should be equal to the variability in response due to the variability in damping. Also, the equipment damping variability should be split into randomness and uncertainty components.

The equipment modeling variability should reflect variability in mode shape and frequency. Both of these factors are affected by uncertainty in boundary conditions and randomness of material properties. The frequency variability can be used to directly obtain response variability using the median floor response spectrum at the location of the steam generators.

The randomness of the capacity factor due to material property variability is equal to 0.14. According to the calculations, this was obtained by the following expression:

$$\ln (1.25)/1.65 = 0.14$$

where 1.25 is the lower bound capacity factor and 1.65 corresponds to the 95 percent probability level. This is incorrect. The logarithmic standard deviation should be based on the randomness in design capacity due to the randomness in the material properties. Although the value used (i.e., 0.14) is reasonable, the method for obtaining this number is not rational.

The variability in the inelastic energy absorption factor was incorrectly set equal to the variability in the ductility ratio. A more correct value is obtained by the following expression:

$$\ln \sqrt{\frac{2(3) - 1}{2(1.5) - 1}} / 2 = 0.23$$

In addition to the uncertainty in the ductility ratio, there is uncertainty in the inelastic energy absorption model, which should be included. Again, the value of 0.35 used is reasonable, but the method for obtaining this value is not rational.

In conclusion, the median acceleration value of 2.28g may be reasonable. The structural response factor should be 1.5, not 2.51, but the spectral shape factor (called the third design capacity factor in the report) should be based on the design versus the median spectral ordinates at the natural frequency of the steam generator. It is likely that the correct safety factor for spectral shape is greater than the 1.5 value used in the analysis. Thus, the two discrepancies tend to offset each other.

The total variability logarithmic standard deviation value of 0.62 tends to be on the high side. It is likely that a more careful analysis would reduce this value.

<u>Auxiliary Building</u> (Collapse)

The design capacity safety factory, FCSD, equal to 1.6 for the Auxiliary building collapse, is based on the strength of columns in the building. It is assumed that there is no significant margin beyond the design load; thus FCSD is equal to 1.1/0.7, where 1.1 is the SSE load factor and 0.7 is the code capacity reduction factor for tied columns. It is not clear why columns are controlling the capacity for seismic events since the Auxiliary building is a shear wall structure. In typical shear wall buildings the walls resist the lateral forces and the columns support vertical dead and live loads. For cases where lower story columns support upper story walls, axial forces due to seismically-induced overturning moments at the bottom of walls will be predicted by elastic analyses; however, if the columns begin to yield the lateral loads will redistribute through the floor diaphragms to walls which extend to the foundation. This safety factor is an important contribution to the median capacity and is low compared to similar factors for shear wall structures at other nuclear power plants.

The material strength safety factor, FCSM, equal to 1.35 is based on the ratio of the column axial strength using actual material properties to the strength using design properties. This is appropriate if column axial strength is the dominate strength contribution. Because an axial column failure is a brittle failure no inelastic energy absorption was assumed. As discussed above it may be unrealistically conservative to assume that the Auxiliary building will fail based on overstressing the building columns. Possible redistribution of forces should be considered.

The spectral shape factor, FRSS, was based on the ratio between the spectral ordinates of the design spectra and the median rock spectra from WASH-1255 (Ref. 8). The approach used is appropriate and is consistent with other PRA studies.

The damping safety factor was incorporated in the development of the spectral shape safety factor in that the design spectrum at 5 percent damping and the median rock spectrum at 10 percent damping were used. This is a consistent approach.

The modeling safety factor, FRSM, was assumed to be 1.0. It is stated in the calculations that modeling was accounted for in the spectral shape safety factor. Since the calculated frequency was assumed to be the median frequency, this is correct.

The SRSS combination of modal response was used; hence, the modal combination safety factor was assumed to be 1.0. This is reasonable.

The earthquake components were evidently combined using an SRSS combination for the Auxiliary building. This contrasts to other Millstone building analyses where the absolute sum of all three components was supposedly used. For the Auxiliary building, a safety of 1.0 is appropriate for this safety factor.

Since the Auxiliary building is on rock, the safety factor for soil-structure interaction is 1.0.

In conclusion, the total safety factor of 3.24 appears to be low. The assumption that the interior columns control the capacity of the Auxiliary building should be reexamined.

The uncertainty logarithmic standard deviation for strength failure of structures was assumed to be 0.20 for all buildings. It was not based on the uncertainty of the different strength prediction models, but was assumed based on values used in the Zion and Oyster Creek PRAs. Because of the possibility of load redistribution, this value is on the low side.

The randomness logarithmic standard deviation for material strength of the Auxiliary building was based on the variability of individual construction materials (i.e., steel and concrete). However, the variability of the strength equation should have been used rather than the variability of the individual material strengths. The value of 0.10 which was assumed is reasonable compared to results from other PRAs.

Because no inelastic energy absorption was assumed for the Auxiliary building, no variability was assumed. This is reasonable based on this premise; however, the assumption that there is no inelastic capacity is very conservative.

In developing the variability for the spectral shape safety factor, it was assumed that the uncertainty component is zero, which is similar to what was done in past seismic PRA studies. As discussed in Reference 3, it is believed that if this were true, there would be no motivation to ever conduct site studies to develop site-specific spectra. The approach used to develop the randomness component was based on the logarithm of the response spectral ordinates from the WASH-1255 spectra at the median and the median plus one standard deviation curves, which is a reasonable approach.

The variability for damping effects was assumed to be all uncertainty. The logarithmic standard deviation was based on the logarithm of the response spectral ordinate from the median curves at 7 percent and 10 percent, where it is assumed that the 7 percent value is at the minus one standard deviation level. The approach used is reasonable. However, there should be a small randomness component.

The variability for modeling is assumed to be all uncertainty with a logarithmic standard deviation of 0.15. This value is consistent with other PRA studies; however, it is more rational to compute this parameter for each building, considering variability in the mode shape and fundamental frequency. In the Zion and Indian Point studies, the logarithm of the ratio of the spectral ordinates of the median spectrum at the median and median minus one standard deviation frequencies was used to compute the uncertainty value.

The variability for modal combinations was assumed to be all randomness, and the logarithmic standard deviation value of 0.17 was assumed based on what was used in the Zion PRA. This is consistent with other PRA studies.

For the combination of earthquake components, the variability was assumed to be all randomness, which is appropriate. However, a value of 0.17 was assumed to be equal to the same value as used for modeling (see discussion above). The basis for this value is not rational and is probably on the high side. The results of a study conducted for the Limerick PRA (Ref. 7) were based on considering the possible response extremes to be at ±3 standard deviations. Different response coupling and phase relationships (i.e., in-phase and out-of-phase) were considered. The values for the logarithmic standard deviations ranged from 0 to 0.16. Although the assumption of ±3 standard deviation range is debatable, based on this study the 0.17 value seems high.

Since the Auxiliary building is on rock, there will be no significant soil structure interaction, hence the variability is essentially zero.

In conclusion, the logarithmic standard deviation for combined variability is 0.43, which is consistent with other studies. However, the approach used to obtain the value is not entirely rational as discussed above, and the final value of 0.43 is probably on the low side.

125 VDC Distribution Panel

The 125 VDC Distribution Panel contains switchgear and is located on the base mat of the Control building. The PRA calculations for this component are confusing, and it is difficult to systematically account for all the discrepancies that have occurred in the calculations. It is assumed in the analysis that electrical malfunctions will occur at a higher acceleration than failure of the external cabinet. Consequently, a static force computer analysis was performed using forces from transmissibility data obtained from dynamic tests of the cabinet.

In the development of the design capacity factor, the computer analysis was conservatively biased. Both horizontal direction earthquake loadings were applied simultaneously (which produced an absolute sum combination in the critical column). The maximum stress appears to be high for the applied loading. The energy absorption safety factor was based on a median ductility value of 1.5, which is low.

The randomness value of 0.14 assumed for the effect of material strength on design capacity is not rational. This is the same incorrect value as used for the steam generator (see discussion above). The variability for inelastic energy absorption is low. The median and median plus one standard deviation values for ductility were assumed to be 1.5 and 1.8, respectively. This is not a realistic range. Also, no uncertainty was assumed for the inelastic energy absorption model.

The equipment response safety factor was assumed to be 1.0 with a variability logarithmic standard deviation of 0.24. The latter value is different from the calculations (i.e., 0.15) and may be low because the variability value consists only of contributions from instrumentation and control errors, and modeling effects. The effect of response spectrum, frequency, damping, mode shape, mode combination, earthquake combination, general test input, and static model factors should also be systematically incorporated into the analysis.

The structural response safety factor also was assumed to be 1.0 with a variability of zero. This assumption was made because the equipment is supported on the base mat which was poured against the ground. According to the FSAR, there is some fill beneath the Control building, and a soil-structure interaction model was developed in the design of the building. The base mat floor response spectrum should be used in developing the equipment response factor and the ground response spectrum used to develop the structural response factor with the appropriate properties of the building (i.e., frequency, damping, mode shape, etc.). Even if the building were supported directly on rock, the equipment response factor should reflect the difference between the design and median ground response spectra.

Note that the analysis assumed a 0.25g peak base acceleration value rather than the SSE value of 0.17g. This discrepancy was encountered at several places in the calculations, which suggests that some of the analysts believed that the plant had been originally designed for an SSE ground acceleration value of 0.25g.

In summary, the median capacity of 0.64g and the combined logarithmic standard deviation value of 0.30 are both low. These values are considerably lower than values for similar components in the Zion and Indian Point PRAs.

<u>Demineralization Water Storage Tank</u> (sliding)

The design capacity factor, FCSD, for sliding is equal to 2.0, which is based on the incipient sliding friction failure between the bottom of the 10-foot thick base mat and the fill concrete placed against the rock foundation. The calculation is based on a sliding coefficient of friction of 1.0 and resistance provided by the compacted earth fill against the base mat. The value of 1.0 is conservative even for incipient sliding. Apparently the applied force due to the SSE earthquake is an absolute sum combination of the two horizontal components. A traditional-type incipient sliding analysis was performed; however, it is unlikely that the tank foundation will slide very far, even for an earthquake with accelerations greater than the median capacity.

The variability logarithmic standard deviation value of 0.1 for capacity is low. It is based on the assumption that a coefficient of friction value of 1.0 is 1.65 standard deviation below the mean value in the logarithmic domain (i.e., median equals 1.2). This is apparently an error because FCSD was based on a median value of 1.0. A more realistic value could be obtained from multiple nonlinear sliding analyses where uncertainty in the dynamic coefficient of friction is used to determine the uncertainty in response. The randomness component was included in the spectral shape randomness logarithmic standard deviation.

The uncertainty for the spectral shape was assumed to be zero. As discussed above for the Auxiliary building, this is not reasonable.

If the absolute sum of the two horizontal earthquake components was used as stated, then the earthquake combination safety factor appears to be correct. The logarithmic standard deviation value for randomness equal to 0.22 appears to be high as discussed above for the Auxiliary building.

In summary, the median capacity value of 0.68g is low, because the design capacity factor was based on a conservative incipient sliding analysis. A more realistic definition of failure should be used to correspond to actual failure conditions of interconnecting equipment. The combined variability logarithmic standard deviation value of 0.38 is low. Because of the uncertainties which are present, a higher value would be more reasonable.

Reactor Core Geometry

The seismic PRA analysis for the reactor core geometry was based on the Westinghouse stress analysis of the upper and lower reactor pressure vessel internals. The development of the equipment capacity safety factor, FCE, is conceptually correct; however, the Westinghouse stress analysis results were not available and the interpretation of these results and use in the seismic PRA analysis were not reviewed. However, the assumed median ductility value of 1.5 used in the analysis appears to be on the conservative side.

The reported equipment capacity factor variability value is 0.36. The corresponding value given in the calculations is 0.20, which does not agree with the report. The difference is probably due to the variability in ductility (0.35 was used for other components) which is not included in the calculations, but somehow was included in the report (however, a value closer to 0.40 in the report, rather than 0.36, would be more consistent with this explanation). The randomness values for material strength and ductility are incorrectly calculated (see discussion above for the steam generator example). The approach for determining the equipment capacity factor variability should be based on the variability in the models, material properties, and ductility ratios and their effect on the equipment capacity safety factor. The first item contributes to uncertainty and the last two to both uncertainty and randomness.

The equipment response factor, FRE, is assumed to be 1.0, since the final stresses were scaled to plant-specific spectral values in the Westinghouse calculation. The value of 1.0 is not correct. The value should be larger due to the difference between the envelope floor response spectra and assumed equipment damping values, and the corresponding median properties. It is likely that FRE is much larger than 1.0 because of these effects.

The reported equipment response factor variability is 0.41. The corresponding value given in the calculations is 0.26, which does not agree with the value of 0.41 given in the report. The uncertainty component was derived generically to include 0.09 for damping and 0.10 for the effect of material properties on dynamic response. The damping term should be obtained specifically for this component since this factor is sensitive to the frequency of the component and the corresponding floor response spectra (i.e., greater variability occurs near spectral peaks).

Also, it is not clear whether the uncertainty value of 0.09 is due to damping or due to the effects of damping variability on response (see discussion above for the Steam Generator). The randomness component is based on a value of 0.22 for spectral shape and 0.05 for the effects of material properties on dynamic response. The latter value was assumed generically, and the basis for the former value is not known, but probably is the same as the corresponding spectral shape value for the building. In addition, there should be randomness included for damping and combination of earthquake components. Also, modeling errors (i.e., frequency and mode shape) and modal combination effects are apparently not included.

The structural response safety factor, FRS, is reported to be 2.00. This value can be separated into the contribution from spectral shape (i.e., 1.5) and combination of earthquake components (i.e., 1.3). The latter factor is incorrect and should be 1.0, since the equipment was designed based on an SRSS combination of earthquake components.

The reported structure response factor variability is 0.42, which agrees with the value in the calculations. A logarithmic standard deviation value of zero was assumed for the effect of damping randomness, which is low; however, the corresponding value of 0.22 used for combination of earthquake components is both high and not appropriate since this variability should be included only in the equipment response contributions. Also, the 0.17 value for modal contribution should not be used since the floor response spectra were probably obtained by a direct integration time-history analysis.

In summary, the median capacity of 0.68g is low primarily due to the unrealistically low value for the equipment response factor. The total variability logarithmic standard deviation reported is 0.67; however, the value in the calculations is 0.54. The latter value is more consistent with previous seismic PRAs, but it is not clear which is the better estimate.

Service Water Piping

The failure of the service water piping is assumed to be the same as the failure of the Engineering Safety Features (ESF) building. However, it is stated in the calculations that compressible material has been provided between the ESF building and the pipe line. No compressible material apparently has been provided between the Emergency Generator Enclosure (EGE) and the service water line. It is concluded in the calculations that the failure of the service water line is represented by the sliding fragility of the EGE (not the ESF building). Apparently there is a discrepancy between the calculations and the report; however, the difference is small.

As discussed above for the sliding failures of the various structures, incipient sliding does not imply immediate failure of the interconnecting components (e.g., service water line). The failure capacity is higher. Because these structures are embedded, it is unlikely that the relative sliding displacements will be significant at the median capacities which are given in the report.

Rocking of a structure such as a containment building can lead to problems for interconnecting piping (e.g., Zion seismic PRA). This is a more likely condition than a sliding-induced failure. This potential failure mode should have been considered in the fragility analysis of structures.

Engineered Safety Features Building (sliding)

The design capacity safety factor, FCSD, for sliding is equal to 2.4, which is based on the capacity of the ring girder in bending where it is assumed that the reinforcement is designed to allowable values. Based on this assumption, FCSD is equal to 2(1.1/0.9), where 2 is the ductility factor, 1.1 is the SSE load factor, and 0.9 is the code capacity reduction factor for bending. Note that the ductility factor was included in FCSD and not in the inelastic energy safety factor, as done for other structures.

It appears that a conservative approach was taken in the development of FCSD. First, it is likely that there is extra capacity in the ring girder which was neglected in the analysis. Second, sliding is also resisted by the contact between the foundation and the rock base. The dynamic coefficient of friction may be large due to irregularities at the foundation/rock interface. Even if the structure starts to slide, the amount of displacement and its effects on interconnected equipment is the important consideration for determining the median capacity value.

The uncertainty logarithmic standard deviation value of 0.20 for the capacity factor was determined generically as discussed above for the Auxiliary building. This value seems low for the sliding failure of the ESF building. Also, since the failure is controlled by the ring girder, randomness for the effect of variability in material properties and ductility should be included.

In developing the variability for the spectral shape safety factor, it was assumed that the uncertainty component is zero, which is similar to what was done in past seismic PRA studies. As discussed above for the Auxiliary building, this value is not reasonable.

The median safety factor for earthquake combination is equal to 1.3. As discussed below for the Emergency Generator Enclosure, this generic value should not be used. Instead a building-specific value should be developed. The logarithmic standard deviation value for randomness equal to 0.22 appears to be high.

In summary, both the median capacity of 0.74g and the combined logarithmic standard deviation of 0.39 appear to be low.

Emergency Generator Enclosure (sliding)

The design capacity safety factor, FCSD, for sliding is equal to 1.8, which is based on the incipient shear failure of the Basil till beneath the building. The capacity was obtained by conservatively reducing the normal force by the vertical earthquake component (corresponding to 0.17g) and multiplying this reduced force by the tangent of 40°, where 40° is the internal friction angle of the till. A small additional capacity was added to this result for the frictional force along the outside walls. This capacity was divided by the base shear due to the horizontal earthquake (probably due to only one component) to obtain the value of 1.8 for FCSD.

This is a conservative capacity factor corresponding to incipient sliding. Because the building is embedded approximately 15 feet, it is unlikely that it will slide very far even for an earthquake with accelerations greater than the calculated median capacity.

The basis for the uncertainty logarithmic standard deviation value of 0.1 for capacity is unknown. It may have been selected to be the same as the value used for the Demineralized Water Storage Tank. See discussion for this component above. This value seems very low.

The uncertainty for spectral shape was assumed to be zero. As discussed above for other buildings, this is not reasonable.

The combined earthquake safety factor, FRSE, value of 1.3 probably is incorrect since it appears that only one earthquake component was used to develop the capacity factor. The logarithmic standard deviation value for randomness equal to 0.22 appears to be high (see discussion for Auxiliary building).

In summary, the median capacity value of 0.75g is low, because the design capacity factor was based on incipient sliding. A more realistic definition of failure should be used to correspond to actual failure conditions of interconnecting equipment. The combined variability logarithmic standard deviation value of 0.37 is low. Because of the uncertainties which are present, a higher value would be more reasonable.

Containment (crane wall failure)

The design capacity safety factor, FCSD, for the containment is equal to 1.75 which is based on an analysis of the columns which support the crane wall (these columns are really wall segments 10 feet by 3 feet in plan and 25 feet high). The calculations which are provided are difficult to follow. The following inconsistencies were noted:

- The axial forces in the columns due to overturning moments were based on plane sections remaining plane. This may not be true.

- The interior walls at the center were neglected which biased the results to the conservative side.

- The maximum moment in the critical column was based on single curvature. Because of the relative dimensions, a fixed boundary condition at the top and bottom would be a more reasonable assumption. This would decrease the applied moment by a factor of 2.

- Because the critical column will be braced by other wall segments, the slenderness effects can be neglected.

- The analysis was performed using code-required strength reduction factors which biased the results to the conservative side.

Also, as the critical walls begin to fail the loads will be transferred to other walls. It is concluded that the FCSD value of 1.75 is overly conservative. A more detailed analysis should be conducted.

The uncertainty logarithmic standard deviation for strength failure of structures was assumed to be 0.20 for all buildings. It was not based on the uncertainty of the different strength prediction models, but was assumed based on values used for Zion and Oyster Creek PRAs (Refs. 1 and 2). Because of the possibility of load redistribution, this value is on the low side.

The randomness logarithmic standard deviation for material strength of the Containment building was based on the variability of individual materials (i.e., steel and concrete). The variability of the strength equation should have been used rather than the variability of the individual material strengths. However, the value of 0.14 which was assumed is reasonable.

The inelastic energy safety factor, FCSE, was assumed to be 1.0 since the column failure would be a compression failure. As discussed above, the analysis is overly conservative, and the failure mode may be tension or shear. It is likely that there is ductile capacity and FCSE should be greater than 1.0.

In developing the variability for the spectral shape safety factor, it was assumed that the uncertainty is zero, which is similar to what was done in past seismic PRA studies. As discussed above for the Auxiliary building, this value is not reasonable.

The analysis conducted for the Containment internal structure for the PRA combined the seismic effects by the SRSS method. Thus the earthquake components safety factor, FRSE, value of 1.3 is not appropriate. A value of 1.0 should have been used. As discussed above for the Auxiliary building analysis, the randomness logarithmic standard deviation for this safety factor is high (i.e., 0.22 was used).

In summary, the median capacity of 0.87g appears to be low. The combined logarithmic standard deviation of 0.47 may also be on the low side.

2.3 CLOSURE

The overall impression of the seismic fragility analysis is that the results are very conservative. The numerous conceptual and philosophical errors encountered produce a lack of confidence that the fragility analysis was properly performed. Although the amount of computations, as evidenced by the thickness of the calculation file (approximately eight inches thick), and the additional response spectra analyses performed for the seismic PRA indicate that considerable resources were expended; however, the final results are not consistent with the state-of-the-art.

Based on comparing the fragility results of the Millstone PSS with similar data from the Indian Point and Zion PRAs, it is judged that the median fragility estimates are a factor of 2 to 3 low. This estimate is speculative since an independent analysis has not been performed to confirm the reasonableness of the higher structure and equipment capacity values.

Although the variabilities (i.e., randomness and uncertainty) were obtained in an incorrect manner, the final results are consistent with results from other PRA studies. However, the uncertainty values for Millstone and other PRA studies are generally on the low side.

3.0 WIND

In Section 1.2.5 of the Millstone 3 PSS, it is concluded that wind does not contribute significantly to plant risk. The governing wind event at the Millstone site is the occurrence of severe tornados. In general, the effects of tornados, hurricanes, and extratropical cyclones (i.e., normal winter storms and thunderstorms) should be considered in the wind risk analysis. As discussed below, it is agreed that tornado effects, which potentially create much larger loads, do not contribute significantly to plant risk; thus, the effects of other wind loads are implicitly included.

It is stated that all Millstone Unit 3 safety-related structures are of reinforced concrete construction with wall thicknesses of at least two feet. Except for some of the Quench Spray system components, all other safety-related components are contained in safety-related structures (Ref. 9, Table 3.2-1).

Based on the analysis described in Section 1.2.5.1.1 of the Millstone 3 PSS, it is stated that the frequency of exceeding the design tornado wind speed of 360 mph is approximately 5.4×10^{-6} per year. It is believed that this value is very conservative as discussed below.

At the Indian Point site, which is approximately 100 miles away and which is in an area with higher tornado activity based on historic data, the mean maximum tornado wind speed at the 10^{-7} per year frequency level is 230 mph with an 80 percent confidence range of 170 to 340 mph (Ref. 1). Other independent point estimates for the Indian Point site at this frequency level are 236 mph and 200 mph (Ref. 3). Note that these results are significant since the reported mean rate of tornado occurrence in the Millstone Unit 3 PSS is 1.87×10^{-4} per square mile per year, which is lower than the value of 2.4×10^{-4} per square mile per year used in the Indian Point study (Ref. 1).

A recent technical paper by Twisdale gives velocity/frequency curves for four regions of the contiguous U.S. (Ref. 10). None of the curves extend beyond 300 mph. Finally, using an approach developed by Reinhold (Ref. 11), the mean frequency using a tornado occurrence rate of 1.87×10^{-4} per square mile per year was found to be less than 10^{-8} per year. It is concluded that the mean frequency of occurrence of tornados with maximum wind speeds equal to or greater than 360 mph is less than 10^{-8} per year.

On the capacity side of the problem, all safety-related structures are designed, using code procedures and allowable strength values, to resist wind speeds of 360 mph and associated tornado missiles. From a probabilistic viewpoint, the frequency of structural failure or missile-induced damage given a 360 mph tornado would be one to two orders of magnitude lower than the frequency of the tornado occurrence.

Because of the extremely low mean frequencies of failure (i.e., on the order of 10^{-9} to 10^{-10} per year), it can be safely concluded that tornado (and hence other lesser wind types) effects are not significant. Even considering the contribution of uncertainty it is unlikely that the effects of wind would contribute significantly to the plant risk.

4.0 EXTERNAL FLOODING

In Section 1.2.3 of the Millstone PSS, it is concluded that external flooding is an insignificant contributor to plant risk. Only two sources of external flooding are considered to potentially impact the Millstone site: tidal flooding and intense precipitation. Since there are no major rivers or streams in the vicinity of Millstone Point, river flooding and dam failure are not considered applicable to the site. Tsunamis are also excluded since there is an extremely low probability that these events will occur along the North Atlantic coast line.

The justification for excluding external flooding from the formal risk analysis is made on a qualitative basis. No formal probabilistic analysis was performed. Tidal flooding and intense precipitation are based on the effects of the Probable Maximum Hurricane (PMH) and the Probable Maximum Precipitation (PMP), respectively. No probability values are given; however, these events are judged to have a point estimate frequence of occurrence between 10^{-6} to 10^{-4} per year. This estimate is based on an approximate analysis using available hurricane hazard data in the vicinity of the Millstone site (Refs. 1 and 12).

The description of the calculations, which were conducted to obtain the maximum wave runup and standing wave height due to the PMH and the flood depth due to the PMP, are contained in the FSAR (Ref. 9). It is apparent from the description given that conservatisms were included in the calculations (e.g., the most severe combination of hurricane parameters were used to represent the PMH and the site yard drains were considered ineffective in the PMP analysis). However, the amount of additional conservatism is not known. It is not necessarily true that single extreme events are the only circumstances that contribute to the risk. Also, the PMH and PMP may be correlated since the PMP could be caused by the PMH.

In contrast to the seismic analysis, the external flooding analysis did not explicitly consider the uncertainty (which is large) in the underlying parameters and models. Even at the 100-year storm level, the coefficient of variation on water depth is expected to be approximately 0.2 to 0.3. Thus, the conclusion that external flooding has a very low frequency of occurrence is not convincing without some formal quantification of the hazard.

By including the effect of uncertainties in the external flood analysis, a distribution on the frequency of occurrence can be obtained. The present analysis implies that the frequency of flooding above the protected elevation is small. However, the margin of safety above the PMH and PMP design elevation is also small (less than 1 foot for the PMH and less than an inch for the PMP).

As an example, the point estimate for the PMH might be 10^{-5} per year; however, because of the large uncertainties that are present, there is a small but finite probability that the frequency of the PMH is 10^{-4} per year or larger. Similarly, it can be argued that there is a potential hurricane bigger than the PMH which could produce a wave runup which exceeds the water-tight elevation of 25.5 feet msl. The point estimate for this event might be on the order of 10^{-6} per year; however, due to uncertainty there also is a small but finite probability that it is 10^{-5} per year or larger. Proceeding in this manner, it can be shown that including uncertainty will result in a family of hazard curves which may increase the mean frequency of water depth above the value obtained using only a single point estimate value (i.e., the PMH). In order to evaluate the implications of a water level greater than 25.5 feet msl, it is necessary to either conservatively assume core melt or to develop event trees, fault trees, and equipment fragilities to systematically incorporate the unique features of the plant into the uncertainty analysis.

In summary, a formal analysis should be conducted which provides frequencies of occurrence and includes uncertainty in the external flood models and parameters. Because of the large uncertainties which exist for external flood, there is the possibility that the mean frequency of core melt is larger than 10^{-6}. In order to conclude that the contribution from external flooding is insignificant relative to other hazards, a complete statement of the probability distribution on frequency of occurrence should be provided.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The following are conclusions based on the review of the Millstone PSS and recommendations for additional work which should be conducted. The information is given below for seismic, wind, and external flood hazards.

5.1 SEISMIC

Because of the very conservative assumptions and numerous errors which have been made, the fragility parameter values do not represent the results of a state-of-the-art analysis. The median ground acceleration values for the components which are significant contributors to the frequency of core melt are judged to be a factor of 2 to 3 low compared to similar components from other recent PRA studies. This estimate is speculative and was not confirmed in detail for the specific components. In contrast, the logarithmic standard deviations for randomness and uncertainty are generally consistent with results obtained from other PRA studies; although numerous errors (many which are compensating) were made in calculating the variabilities. The logarithmic standard deviations for uncertainty are on the low side; however, the effect of this bias is probably small on the final risk results.

Based on the findings of the review, the fragility parameters should be recalculated to eliminate the excessive conservatisms and to correct the errors which have occurred. The new fragility curves should be incorporated in the systems analysis and combined with the seismic hazard curves to produce more realistic distributions on frequency for core melt and other consequences.

Also, after the plant is completed a review should be conducted to determine if any non-safety related structures or components could fail, fall, and impact the safety-related items in the plant.

5.2 WIND

The conclusion in the Millstone PSS that wind effects do not contribute significantly to the risk of radiological consequences is reasonable. Because all safety-related structures have been designed to resist tornado loads and resultant missiles for wind speeds up to 360 mph, the minimum thickness of concrete walls and roofs is two feet. A point estimate frequency for this speed is on the order of 10^{-8} per year at the Millstone site. Failure of the concrete structures would be one to two orders of magnitude lower. Even incorporating the effects of uncertainties, wind hazard will not become a significant external event.

5.3 EXTERNAL FLOODING

The exclusion of external flooding as a significant event is based on qualitative arguments. No formal probabilistic analysis or even point estimate values are offered in defense of this conclusion.

An approximate analysis indicates that the PMH has a frequency of occurrence between 10^{-6} and 10^{-4} per year. If uncertainties are included, the risk of flood waters exceeding critical elevations may be significant. No information is given regarding the consequences (i.e., flooding-induced equipment fragilities or systems analysis) if the protected elevations are exceeded. In conclusion, there is no quantitative basis to conclude that external flooding is not a problem.

In order to provide a rational basis for judging whether external flooding is a significant contributor to off-site consequences, a formal probabilistic hazard analysis should be conducted which incorporates the uncertainties of the methods and parameters. If the probabilities of the frequencies of exceeding the protected elevations are significant, then fragility and systems analyses for flooding may be required.

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