APPENDIX A

REVIEW OF THE REVISED
MILLSTONE UNIT 3 PROBABILISTIC SAFETY STUDY
SEISMIC FRAGILITY

by

John W. Reed

Martin W. McCann, Jr.

Prepared for

Lawrence Livermore National Laboratory

Livermore, California

May 4, 1984

Jack R. Benjamin & Associates, Inc.
Consulting Engineers
Mountain Bay Plaza, Suite 501
444 Castro Street, Mountain View, California 94041

TABLE OF CONTENTS

1. INTRODUCTION

Jack R. Benjamin and Associates, Inc. (JBA) was retained by Lawrence Livermore National Laboratory (LLNL) to perform a review of the fragility analysis of the structures and components at the Millstone Unit 3 Nuclear Power Station. The fragility analysis was performed by Structural Mechanics Associates (SMA) for Northeast Utilities Service Company (NUSCO) (Ref. 1). A previous review by JBA of the fragility analysis originally used in the Millstone Unit 3 Probabilistic Safety Study (referred to as the Millstone PSS) is documented in Reference 2. In regards to the original seismic fragility analysis, JBA recommended that the fragility parameters should be recalculated to eliminate excessive conservatisms and to correct errors which had occurred. In addition, it was recommended that 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.

In response to the first recommendation, NUSCO retained SMA to perform a reanalysis of the seismic fragilities, which are documented in Reference 1. This report presents our review of the revised analysis.

1.1 SCOPE

Jack R. Benjamin and Associates, Inc. has performed similar reviews of the Indian Point Probabilistic Safety Study (IPPSS) (Ref. 3) and the Zion Probabilistic Safety Study (ZPSS) (Ref. 4). (See Reference 5 for the IPPSS review. The ZPSS has not been published.) Based on experience gained from the initial review of the Millstone PSS and the IPPSS and ZPSS reviews, the evaluation of Reference 1 was conducted in a short time period in order to quickly determine the adequacy and accuracy of the results and to make recommendations based on the findings. In contrast to the previous reviews of the IPPSS and ZPSS which consisted of an in-depth evaluation of each section and subsection, this review focused only on critical components and issues which may impact the results.

This review consisted of reviewing Reference 1 and studying the calculations provided by NUSCO which document the development of the fragility parameter values. The new fragilities were developed only for the safety-related structures and for components with median ground motion capacities less than 1.5g. Note that all capacities cited in this report are referenced at the free-field ground surface level. The results of the original analysis were used to screen the components and only the low capacity ones were selected for reanalysis. We agree that this is a reasonable approach since the original analysis is excessively conservative. However, it is implicitly assumed that components with median capacities greater than 1.5g do not contribute significantly to core melt or risk.

The revised hazard and systems analyses were not reviewed. It is assumed that the NRC will evaluate these analyses in their entirety. Because of the overlap between the fragility analysis and the hazard and systems analyses, Amendment 2 to the Millstone PSS was quickly read. Based on this reading, we question whether the 5.3 to 6.3 range on earthquake magnitude that is assumed in the fragility analysis in Reference 1 is realistic. The implications of a higher range is discussed in Chapter 2. Also, we do not believe that the systems fragility curves and the hazard curves have been properly integrated. The mean annual frequency of core melt value of 1.7×10^{-5} seems high. This concern was communicated to the NRC in a telephone conference call on April 19, 1984.

In Chapter 2, the effect of earthquake characteristics on fragility calculations is discussed. In this chapter, the effect of earthquake duration and magnitude are considered. This has been a troublesome philosophical (and practical) problem in previous PRA studies. The approach used in Reference 1 is different from other PRAs. An evaluation of the current approach in relationship to previous procedures for handling this issue is given. Also, the effect of using a site-specific response spectrum shape and the relationship between peak ground velocity and peak ground acceleration are discussed in Chapter 2. This latter issue is important to the structure sliding analyses and the resulting median capacities. In Chapter 3, the fragility analysis is addressed. General comments are given and the results of our review of specific structure and component fragilities are provided. Finally, Chapter 4 gives conclusions and recommendations based on the findings of our review.

1.2 OVERALL METHODOLOGY

The methodology used in Reference 1 to develop seismic fragility data is appropriate and adequate to obtain a realistic estimate of structure and component fragility. In general, we believe that more representative capacity values have been developed in the revised analysis as compared to the original fragility analysis. We have some specific concerns as discussed subsequently in this report.

As discussed in Chapter 3, some revisions to the methods have been made, which has improved the analysis approach. The following three issues have been considered in Reference 1 in a different manner as compared to past seismic fragility analyses. Comments concerning these issues are given below.

- Design and construction errors
- Lower-bound fragility cut-off
- Correlation between failure modes

Design and Construction Errors

The issue of design and construction errors is discussed in Reference 1. As in other PRAs, this type of error is not generally included in the fragility calculations. However, in contrast to other PRA reports, it is stated that there is the possibility that unidentified design and construction errors may exist which can affect the seismic capacity. This recognition is important, although not much data is available to explicitly incorporate this effect in the analysis. This is an important area which is in urgent need of research.

Lower-Bound Fragility Cut-Off

A mathematical procedure for establishing a lower-bound cut-off on fragility curves is given in Reference 1. The method is reasonable, but is based on engineering judgment without any data to support the values used. In Amendment 2 to the Millstone PSS, it is stated that components were eliminated from the systems analysis if the acceleration capacity at two standard deviations below the median capacity is greater than 0.8g. In Table 2.5.1-1A in Amendment 2 to the Millstone PSS, the 37th (last) component listed (i.e., the steam generator tubes rupture) is the only component which satisfies this criteria and hence could be eliminated. For the Millstone 3 reanalysis, this cut-off issue is not of any practical significance, since it appears not to have affected the analysis.

Correlation Between Failure Modes

The issue of correlation between failure modes is discussed in Reference 1. We have raised this issue in our review of past PRAs. Although correlation has been treated conservatively in the past, it is important not to ignore potential unconservative situations which may arise in future PRAs. It is stated in Reference 1 that consideration should be given to possible correlation between controlling seismically-induced failure modes. In a quick reading of Amendment 2 to the Millstone PSS, we saw no evidence that this issue had been considered. We trust that the NRC will investigate this concern as part of their review of the systems analysis.

These concerns and other general philosophical concerns from past PRA studies also apply to the Millstone PSS. Reference 5 discusses these issues in depth based on the review of the IPPSS. The reader is directed to Section 2 of Appendix A of Reference 5 for a general discussion of these concerns.

2. EARTHQUAKE CHARACTERISTICS EFFECTS ON FRAGILITY

2.1 EFFECT OF EARTHQUAKE MAGNITUDE AND DURATION

It has been generally recognized that the use of instrumental peak ground acceleration is an ineffective basis to predict the damage potential of earthquake ground motion. Other factors, such as the number of cycles and frequency content of ground motion are also important. As a result, an effort has been made by SMA to account for these additional factors in the development of seismic fragility curves for structures and equipment. As new PRAs are performed, SMA has attempted to improve the procedure to do this. The Millstone PSS is the most recent attempt to do this.

Background

As background to the review of the Millstone fragility analysis, a brief review is given of previous attempts to develop a damage effective ground motion parameter. This is an area of ongoing development, that is at times troublesome and difficult to understand.

The Zion (ZPSS) and Indian Point (IPPSS) PRAs (Refs. 3, 4) were the first attempt to define a damage effective ground acceleration which was applied in a seismic risk analysis of a nuclear power plant. In developing a damage effective acceleration, two steps were taken. First, an effective peak acceleration (EPA) was defined which was an acceleration value that could be used to scale a broad-band response spectrum (e.g,. WASH 1255 spectrum (Ref. 6)) such that the predicted spectral accelerations in the frequency range 2 to 10 Hz are consistent, in a median sense, with spectral levels of real earthquakes in the earthquake magnitude range of interest. As indicated in Reference 4, the EPA value is dependent on earthquake size. For small magnitude events, the EPA is significantly less than the instrumentally recorded peak acceleration (IPA). This is due partially to the fact that smaller magnitude earthquakes have narrow, peaked response spectra and short durations. For large magnitude events, which have a broad response spectrum shape, the effective peak acceleration would equal the IPA. Anchoring a broad-band response spectrum shape to an EPA provides an elastic response spectrum that is median centered in the 2 to 10 Hz frequency range.

To determine a median-centered, broad-band spectrum, SMA recommended in the Zion and Indian Point PRAs that the EPA be equal to

$$EPA = 1.25 * A_{3F} \tag{2.1}$$

where A_{3F} is the third-highest peak acceleration or sustained acceleration in a low-pass filtered acceleration record. Frequencies beyond 9 Hz were eliminated. Implied in equation 2.1 is the assumption that earthquakes that contribute to failure are small to moderate size events (i.e., $5.3 < M \leq 6.3$).

In the next step, the elastic response spectrum is modified to reflect its potential to damage structures or equipment with natural frequencies in the 2 to 10 Hz range. The basis for this second step is the fact that in order for damage to occur, a structure or equipment item must experience multiple cycles of response. Consequently, for small magnitude earthquakes that have relatively short durations, the expected amount of damage is small, and thus the elastic response level would be significantly reduced. For large magnitude events, which last longer, little or no modification is required, according to the Zion method.

In order to estimate the damage potential of earthquake ground motion, a damage effective acceleration was defined as,

$$\begin{aligned} A_D &= \frac{EPA}{F} \\ &= \frac{1.25}{F} * A_{3F} \end{aligned} \tag{2.2}$$

where the factor F is a function of earthquake magnitude and duration, and the level or type of damage. The intent of the F factor is to account for the less damaging effects of small earthquakes by effectively reducing the intensity of ground motion that is input to a structure. At the time the Zion and Indian Point studies were done, only limited information on the possible values of F was available. It was felt by SMA that F would lie in the range of 1 to 3. Thus, a single value of 1.25, reported to be conservative, was used. This resulted in $A_D = A_{3F}$, and the need to shift the seismic hazard curves by a factor 1/1.25 to sustained acceleration values where they had been defined in terms of sustained peak accelerations.

With respect to the approach used in the ZPSS and IPPSS, a number of comments are given. First, the definition of effective peak acceleration is based on the use of a broad-band response spectral shape, which when anchored to the EPA gives the median spectral acceleration in the 2 to 10 Hz frequency range. For Zion and Indian Point, the median spectral shape in Reference 6 was used by SMA. As a result, the definition of EPA is strongly dependent on these factors, and would presumably change, if a different broad-band spectrum was used, or a different frequency range were considered. Estimates of EPA are therefore relative to these factors. If a magnitude-dependent spectral shape is used, the estimate of an EPA would be different. This is discussed later in this section.

In support of equation 2.1, SMA has reported the results of a study where the response spectra for twelve earthquakes were compared to WASH 1255 broad-band response spectra anchored to an EPA as defined in equation 2.1 (Ref. 7). Although the visual comparisons in Reference 7 appear convincing, statistical analyses were not conducted to empirically define an appropriate EPA relationship. There is an implied modeling uncertainty in this approach, since more realistic approaches could have been used to determine a definition of effective peak acceleration.

In comparing actual earthquake response spectra to broad-band spectra scaled by an EPA, the mean plus one standard deviation WASH 1255 amplification spectrum was used by SMA in their analysis (Ref. 7). It would have been more appropriate, in our opinion, to have used the median-centered amplification spectrum. As a result, there is some doubt in our minds as to the appropriateness of equation 2.1 to estimate an EPA, and thus there may be a bias in the 1.25 factor. The arguments given by SMA are less convincing without the benefit of a statistical analysis to support their conclusions.

From Reference 7 we note that the estimate of effective peak acceleration is explicitly defined for frequencies less than 8 Hz, while the Zion and Indian Point studies assume an applicable range of 2 to 10 Hz. This appears to be inconsistent.

Following the Zion and Indian Point studies, the Limerick Severe Accident Risk Assessment (Limerick SARA) was published (Ref. 8). In this study, the results of research work were used to revise the seismic risk model. Ground motion intensity was expressed in terms of effective peak acceleration and a broad-band response spectrum (Ref. 6). However, in performing the seismic risk calculations, the seismic hazard curves were shifted to convert from EPA to $A_D = A_{3F}$. Thus, an adjustment identical to that in the ZPSS and IPPSS was made, suggesting the F factor in equation 2.2 was again taken as 1.25.

However, in the Limerick SARA an Earthquake Duration factor of 1.4 was incorporated in the fragility analysis to account for the less damaging effects of small magnitude earthquakes. The earthquake duration factor has the effect of increasing structure capacities, when the size of the expected earthquakes is small, as opposed to decreasing the hazard, by the 1/F factor given in equation 2.2. It was concluded in our review (Ref. 9) with concurrence by SMA, that the F factor in equation 2.2 and the earthquake duration factor included in the fragility analysis accounted for the same phenomena, and therefore only one factor should be used. On this basis we conclude that for the methodology used in the Limerick SARA, the earthquake ground motion hazard is more appropriately characterized by the EPA as defined by equation 2.1, keeping in mind that the factor on A_{3F} is still a function of earthquake magnitude.

In summary, the F factor previously used to shift the accelerations in the seismic hazard analysis, was incorporated in the seismic fragility analysis for Limerick, as an earthquake duration factor. When the earthquakes that contribute to risk are small, then the duration factor serves to increase the capacity of structures, because of the less damaging effects of smaller, shorter duration earthquakes. The median value of this factor as used by SMA was 1.40 based on work reported in Reference 10. This represented an increase from the previous value of 1.25 used in ZPSS and IPPSS. In our review of the Limerick study (Ref. 9), we generally agreed with this approach, but felt the factor of 1.40 may be too high.

Generally speaking, the Limerick SARA study represented an improvement in the seismic risk analysis. Detailed comments on this method are provided in Reference 9.

Millstone PSS

The latest effort by SMA to establish a realistic ground motion characterization and seismic fragility model was performed for the Millstone PSS (Ref. 1). This approach is summarized below, followed by review comments. Based on the work reported in Reference 10, a procedure somewhat different from that used in previous PRAs was developed. In terms of the seismic hazard, peak ground acceleration was used to characterize the intensity of ground motion. In addition, a magnitude-dependent response spectrum shape, developed by LLNL (Ref. 11) was used, rather than the WASH 1255 broad-band spectrum. Discussion of the magnitude-dependent spectrum is given in the next section. A response spectrum shape corresponding to earthquakes with magnitudes 5.3 to 6.3 was selected, which according to the seismic hazard analysis in Appendix 1-B to Amendment 2 to the Millstone PSS was the range of earthquake magnitudes that contributed to accelerations around 0.17g, the SSE level. This is troublesome, since the accelerations that contribute to the mean frequency of core melt appear to be much higher. Whether it can be assumed that earthquakes of this size are the dominant contributors to failure, is discussed later.

As discussed above in regards to the ZPSS and IPPSS, the characterization of effective ground acceleration was defined relative to the frequency range of interest, a WASH 1255 broad-band spectra, and earthquake magnitude. In the case of Millstone, rather than using a broad-band spectrum, a magnitude-dependent spectrum was selected. As a result, the definition of effective peak acceleration used in ZPSS and IPPSS no longer applies. Instead, the effective peak acceleration for a median-centered, magnitude-dependent response spectrum is the instrumental peak acceleration. To understand this, recall that in the case where a broad-band spectrum is used, if large earthquakes are dominant contributors to risk, then the EPA used to scale the spectrum shape is equivalent to the IPA. This will be the case since the response spectra of large magnitude events are also broad-band. The same analogy can be made when a magnitude-dependent spectrum is used. We therefore agree that peak ground acceleration is the appropriate parameter to characterize strong ground motion for the Millstone seismic analysis.

In previous PRAs the effect on seismic capacity of earthquake magnitude and duration was accounted for by shifting the seismic hazard curve (e.g., ZPSS and IPPSS) or increasing the seismic capacity relative to an EPA value (e.g., Limerick SARA). Based on research conducted by SMA (Ref. 10), larger magnitude earthquakes that have longer durations and thus produce many cycles of structure response, will exhibit less ductility at failure than smaller events with short durations, and lower levels of ground shaking intensity. In Reference 10, the available or effective ductility in single-degree-of-freedom systems of various frequencies subjected to earthquake ground shaking was calculated. The results of this study provided the basis to estimate an Inelastic Energy Absorption factor of safety, based on an effective ductility and the Riddell-Newmark formula. The effective ductility, μ^*, is estimated to account for the influence of earthquake magnitude and duration. In this approach, the following formulation was used by SMA:

$$\mu^* = 1.0 + C_D\,(\mu - 1.0) \tag{2.3}$$

where the factor C_D is a function of earthquake magnitude and is the structure ductility ratio. For earthquakes in the range 4.5 to 6.0, C_D was given as 1.4, suggesting the effective ductility is higher for small magnitude events. For large earthquakes, C_D = 0.70, which gives a lower effective ductility.

As indicated earlier, the magnitude range 5.3 to 6.3 was assumed to make the greatest contribution to risk, thus a C_D value of 1.3 was assumed. This value was subjectively selected to reflect the slightly higher magnitudes that are expected. A quantitative basis was not given to support this value.

A brief review was conducted to assess the adequacy of the analysis procedure used in Reference 1, and to evaluate the parameters used in the analysis. Overall, the approach used in the Millstone PSS represents an improvement over past PRAs. Based on a preliminary review of the Inelastic Energy Absorption factor, F , with the incorporation of magnitude/duration effects, a number of questions or concerns are raised. In addition to Sections 4.1.2 and 4.1.2.1 of the fragility analysis report (Ref. 1), we also reviewed SMA's supporting calculations and Reference 10.

The C_D factor in equation 2.3 was developed from data reported in Reference 10 for two magnitude ranges: 4.5 to 6.0 and 6.5 to 7.5. In addition, two structure ductilities of 1.85 and 4.27 were considered. SMA calculated C_D equal to 1.40 for the lower magnitude earthquakes and 0.70 for the larger events. We attempted to reproduce the C_D values SMA calculated for each magnitude range/ductility pair and were unable to do so. In one case, our estimate of C_D varied considerably from that of SMA, while in other cases small differences occurred. From the four estimates of C_D, a value for each magnitude range was used in the report. It is not clear from the calculations how the final values of C_D of 1.40 and 0.70 were determined. They are not strict averages within each magnitude range, but appear to be subjectively chosen.

Of greater concern is the frequency dependence exhibited by the data in Reference 10. Based on a preliminary assessment, we observe that depending on the natural frequency of the structure, C_D will vary at low frequencies, from a value greater than 1.0, implying greater effective ductility, to less than 1, or less effective ductility, for higher frequency structures. This observation is independent of both magnitude and ductility ratio. Intuitively, this appears reasonable since we expect a structural system to respond ir an oscillatory manner, consistent with its natural frequency, in an earthquake. As a result, it is reasonable to expect that high frequency structures and components will experience many more cycles of response than structures with lower natural frequencies for the same amplitude and duration of ground motion input. Consequently, lower effective ductilities for higher frequency structures are anticipated. This can have a significant impact on the estimate of the effective ductility. It should be noted that the total impact of this observation is dependent on magnitude and the ductility ratio. To illustrate this relationship we estimate that for structures with natural frequencies of 2.14 Hz and ductilities of 1.85 and 4.27, C_D should be greater than 1.0 for large magnitude earthquakes, as opposed to 0.70 as suggested by SMA.

As a general concern, only 10 earthquake records were used to estimate the C_D values in the Millstone PSS. This is a relatively small sample set to effectively estimate the magnitude/duration dependence of C_D. This is apparent in the fact that the entire magnitude range is not fully represented (i.e., magnitudes 6.0 to 6.5 are not included, and only two large magnitude ranges could be considered). In addition, for an earthquake of a given magnitude, there is considerable variability in the duration of ground motion that can be expected (Ref. 12). As a result, the true variability in C_D is large. Consequently, we feel the available data set provided in Reference 10 is not adequate to fully characterize an effective ductility.

To estimate the variability for the inelastic energy absorption factor, F , it was assumed that there is a 1% chance of F being less than 1 for C_D = 0.70. On this basis, an estimate of β_C, the composite variability was derived by SMA. In principal, we do not agree with this approach to estimating variabilities since it suggests that the assumed lognormal distribution is correct and can be used to prescribe what the variability <u>ought</u> to be. Furthermore, it tends to combine the notions of randomness and uncertainty, which in principal are different. However, we recognize the problems encountered in estimating variabilities, including a lack of data to estimate β_R and the concern that unreasonable frequencies of failure are estimated by the lognormal model at low ground accelerations. As a result, the analyst attempts to constrain the model by fixing the lower tail. In some ways, the engineer is forced to live with the lognormal model and the unrealistic values it predicts, particularly when there is large uncertainty, β_U, in his estimate. This is one example where the lognormal model breaks down by being overly conservative. In general we feel that the engineer should utilize the available data and his judgment to estimate β_R and β_U separately.

An important assumption made in the fragility analysis is that the earthquakes which are dominant contributors to core melt are in the magnitude range 5.3 to 6.3. It is reported in the seismic hazard analysis that accelerations around 0.17g are produced by earthquakes of about magnitude 5.6. However, the chance of core melt may be dominated by accelerations greater than 0.70g. Of greater importance is to know the size of earthquakes that contribute to these levels of ground shaking. Results for the Limerick PRA indicate that the average magnitude will consistently increase for increasing acceleration. As a result, we expect that the average earthquake magnitude that contributes to plant risk may be 6.0 or greater. This would suggest that the duration of ground shaking will be longer than is assumed in the fragility analysis. Thus, the available ductility will be less. Similarly, the magnitude-dependent response spectrum shape which is applicable in the 5.3 to 6.3 magnitude range may not be appropriate.

Conclusion

1. We agree that the magnitude-specific response spectrum should be anchored to IPA.

2. The effective ductility is an appropriate concept, but in addition to depending on magnitude it is also frequency-dependent. We recommend that the dependence of the effective ductility on the natural frequency of structures be taken into account. This influence may have a significant effect on the effective ductility for structures and components with high natural frequencies.

3. If the average magnitude of earthquakes which contribute to risk are greater than 6.3; then effective ductilities will be lower and a different response spectrum shape should be used.

2.2 RESPONSE SPECTRUM SHAPE

In the Millstone PRA, a magnitude-dependent response spectrum shape was used to characterize the intensity of ground motion. This step is a change from other PRAs where a broad-band spectral shape has been used. When using a magnitude-dependent response spectrum the definition of effective peak acceleration changes as a more realistic spectral shape is considered. In this section we review the response spectra and compare it to other spectra available for the site. An evaluation of the site spectra with respect to its influence on the fragility analysis was conducted. It is our understanding that the NRC is performing a critical review of the seismic hazard analysis, including the magnitude-dependent spectrum.

The response spectrum shape for earthquake magnitudes in the range 5.3 to 6.3 developed in Reference 11 for rock sites was used. Figure 2-1 shows this spectra with the Millstone design spectra for 10 percent damping. The procedure described in Reference 11 to convert the 5 percent damped spectrum to 10 percent damping was used. Each spectrum in the figure is scaled to 0.17g, the SSE level. Also shown in the figure is the WASH 1255 broad-band response spectrum.

In addition to these spectra, LLNL (Ref. 13) has conducted a new seismic hazard analysis for the Millstone site. In Figure 2-1, the 1000 year return period spectral shape scaled to 0.17g is shown.

Based on the comparison in Figure 2-1 we find that the magnitude-dependent spectra are generally higher than the design spectra for frequencies greater than 5 Hz. Among these, the most recent spectra developed by LLNL has the highest spectral level. At frequencies less than 5 Hz, the design spectrum exceeds the site-specific spectrum, with the greatest variations occurring at frequencies less than 2 Hz.

In comparison to the WASH 1255 broad-band spectra, the LLNL site-specific spectra both have higher spectral levels at frequencies beyond 5 Hz. In the 2-5 Hz region, the WASH 1255 is higher.

The impact of these spectra on the fragility analysis are summarized in Table 2-1 in terms of their ratio to the Millstone design spectra, for frequencies corresponding to the Control Building, Auxiliary Building, the Containment Crane Wall, Emergency Generator Enclosure, and the Engineering Safety Features Building. These results indicate that the latest spectrum developed by LLNL has considerably higher spectral levels than the Millstone design spectra.

2.3 VELOCITY/ACCELERATION RELATIONSHIP

As part of the seismic fragility analysis for structures (e.g., Control Building) and equipment items (e.g., DWST), the resistance to sliding was evaluated. In predicting sliding displacements due to ground shaking an approximate approach developed by Newmark was used. In Chapter 3, comments are provided on the analysis technique itself. In this section, comments are given on the ground motion characterization aspects of the sliding analysis, as described in Sections 4.1.1.7 and 4.1.1.8 of the SMA fragility report (Ref. 1).

Briefly, the Newmark approach predicts the amount of sliding displacement due to a single acceleration pulse. Based on the relative displacement that is needed to cause failure of buried piping, a relationship was derived to estimate the capacity in terms of peak ground acceleration (e.g., equation 4-9 in the fragility analysis report). Equation 4-9 relates the sliding displacement to the coincident ground velocity and ground acceleration. Based on peak ground motion estimates made by Newmark (Ref. 6), a relationship between peak ground velocity (PGV) and peak ground acceleration of 28 in/sec/g was assumed. From this, the sliding displacement was expressed in terms of peak ground acceleration.

From a review of Reference 6, the 28 in/sec/g ratio was based on four horizontal ground motion records at two stations during the 1971 San Fernando earthquake. The use of only two stations from the same earthquake in our opinion is inadequate. Also, the use of the two horizontal components from a single station is inappropriate, since these acceleration traces are correlated. SMA used these four data points to estimate the variability of the PGV/g ratio and thus is equally inappropriate. To establish an estimate of the median acceleration capacity corresponding to a displacement limit, the peak ground velocity is assumed to occur in the same cycle as the peak acceleration. In general, this is not the case, although the PGV may occur near the PGA within a few cycles. In fact, the joint occurrence of ground accelerations and velocities is random, thus there is a distribution of possible velocity/acceleration pairs that can occur.

Because of the different ground motion attenuation properties between the eastern and western U.S. it is not clear that waveforms expected in the east will have the same characteristics as those in the west. This is particularly true for large magnitude, distant events that could produce high velocities and low accelerations.

As part of this review, data for rock sites in the western U.S. reported in Reference 13 were used as the basis to estimate a peak ground velocity to acceleration ratio. For a total of 15 data points, the estimated mean value was 24.6 in/sec/g with a corresponding logarithmic standard deviation of 0.39. This compares to the 28 in/sec/g mean value and 0.31 standard deviation used by SMA.

As a further comparison, the results of the LLNL probabilistic seismic hazard analysis for Millstone (Ref. 11) were used to estimate a PGV/PGA ratio for annual frequencies of 5×10^{-3}, 1×10^{-3} and 2.5×10^{-4}. For these three values, a mean value of 64.6 in/sec/g was obtained. Although this estimate is considerably higher than the value used in the PRA, it should be noted that this is not an entirely appropriate comparison. The hazard analysis for PGA and PGV were conducted independently, therefore the correlation between PGA and PGV was not preserved. However, this result indicates a possible upper bound.

In our opinion, the value of 28 in/sec/g used in the PRA is reasonably consistent with data recorded in the western U.S. However, it is recommended that this value be looked at from the perspective of the expected ground motion in the east. We also feel the variability in this factor is underestimated.

TABLE 2-1. SPECTRAL RATIOS (10 Percent Damping)

Building	Frequency (Hz)	WASH 1255 / MDS	SS(OLD)* / MDS	SS(NEW) / MDS
Control Building	8.3	1.08	1.20	1.40
Auxiliary Building	8.8	1.08	1.25	1.46
Containment Crane Wall	5.5	0.90	0.97	1.19
Engineering Safety Features Building	12.8	1.18	1.29	1.82
Emergency Generator Enclosure	9.0	1.0	1.30	1.57

* SS = LLNL Magnitude-Specific Spectrum

MDS = Millstone Design Spectra

Note: SS(OLD) is the spectrum used in the fragility analysis.

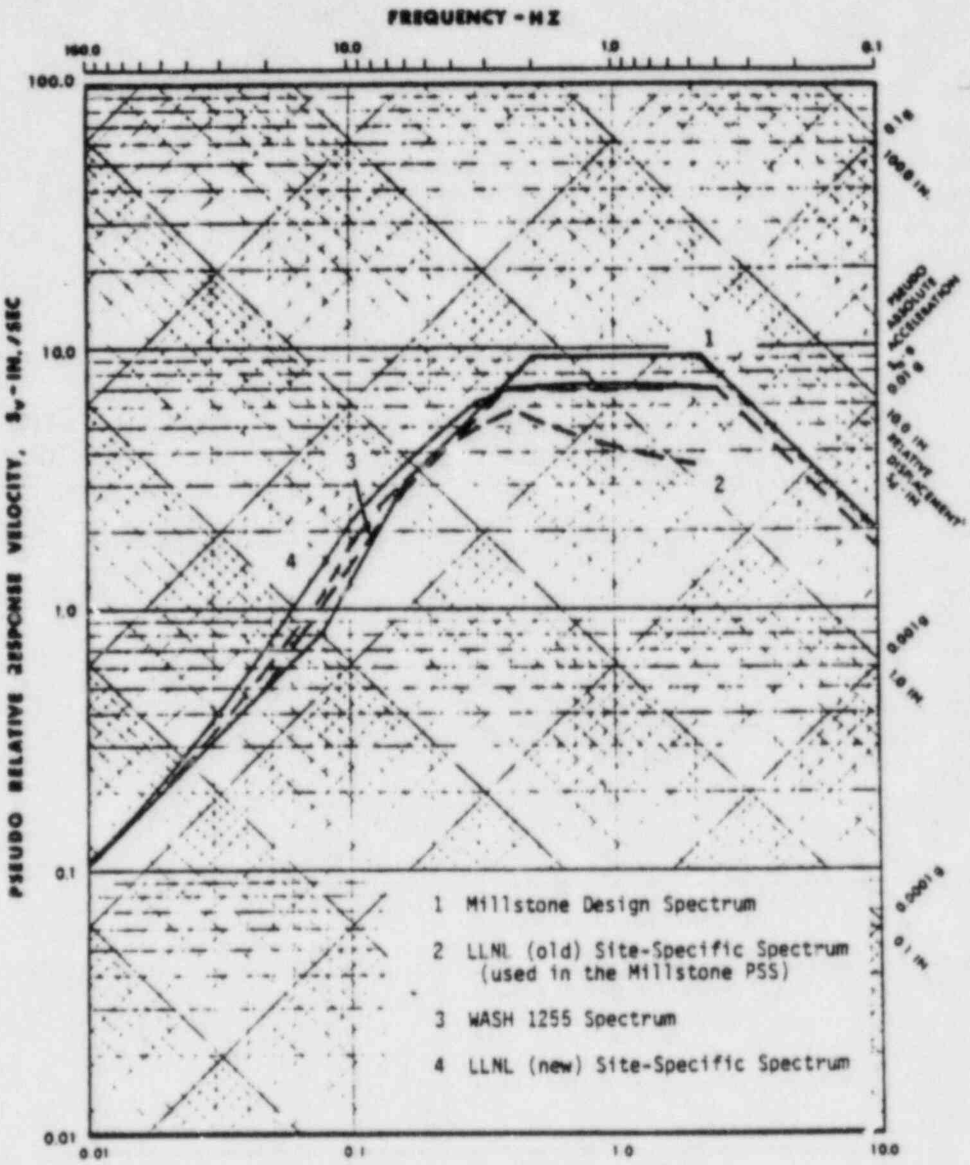

FIGURE 2-1. MILLSTONE RESPONSE SPECTRA FOR 10% DAMPING

3. FRAGILITY ANALYSIS

The focus of the review of the fragility analysis contained in Reference 1 was directed to the critical components which are significant contributors to the Millstone PSS. Based on information provided by the NRC and NUSCO, the following ten structures and components were reviewed.

Structures

- Emergency Generator Enclosure
- Pumphouse
- Control Building
- Engineering Safety Features Building
- Containment Crane Wall

Components

- 4160 V Switchgear
- Service Water Piping
- Emergency Diesel Generator
- RPV Core Geometry
- Control Rod Drive Mechanisms

The review of each of these structures and components is discussed in Sections 3.2 (structures) and 3.3 (components). Section 3.1 gives general comments on the fragility analysis.

3.1 GENERAL COMMENTS

The structure capacity calculations are generally more detailed than previous calculations performed for seismic PRA studies. Except for the Emergency Generator Enclosure, new response spectrum dynamic analyses of the major safety-related structures were performed for the seismic PRA study. The original models developed for the plant design were modified to reflect median properties. Based on a review of the PRA calculations, evidence of the model properties being checked was found. In some cases (discussed below) the models were changed to reflect the correct properties. The median response spectrum assumed in the seismic PRA was used as input to the models, which eliminated the uncertainty of extrapolating from the design analyses for the structures. Note that new floor response spectra were not developed; hence, the fragility analyses for components were performed similar to past PRAs.

Forces from the dynamic analysis were generally distributed to walls using the computer program WALLDI (SMA proprietary program) which is based on the stiffness characteristics and geometry of the structural elements. Both the new dynamic analysis and the force distribution step are improvements over previous PRA studies, where forces were generally obtained based only on the original design analysis results. This new approach reduces uncertainty and should lead to more realistic results (although the logarithmic standard deviations for uncertainty are as large or larger compared to corresponding values in previous PRAs).

In contrast to previous seismic PRAs, more systematic checking of structural elements (i.e., shear walls and diaphragms) was performed. This provides confidence that the critical strength sections have been found. Effects of soil pressure on buried walls was considered; although, the capacity of these walls was not found to be critical.

Sliding analyses were performed for the safety-related structures. In general, both incipient sliding and displacement sliding capacities were determined. It was assumed for cases where sliding is not restricted that a 4-inch displacement corresponds to failure of interconnecting piping. The basis for this criterion is not known. A reference to page DT-48 is given in the calculations for the Emergency Generator Enclosure; however, pages DT-39 through D-57 have been deleted from the Demineralized Water Storage Tank calculations. The basis for the 4-inch displacement value should be justified and reviewed.

An approximate procedure developed by Newmark was used to compute the sliding displacement capacity. Resistance to sliding includes friction between the base mat and foundation, shear keys, and side wall-to-soil friction. Reduction for the effects of the vertical earthquake component and buoyancy due to water were also included.

The Newmark approximate procedure is claimed to be conservative. A quick comparison of the approach with results from nonlinear time history sliding analyses indicates that it gives conservative results for a single sliding excursion. However, due to multiple sliding excursions, which may not be evenly balanced to each side of the starting position, a net drift displacement may occur. In some cases we have found that displacements using an "exact" approach exceed the values obtained from the Newmark procedure. The potential for drift is earthquake magnitude dependent. Since the sliding capacities were calculated to be larger than 1g median, the associated earthquakes are likely to come from large magnitude, long duration events, and hence there will be time for multiple excursions to occur.

An important assumption made in the sliding analysis is the relationship between peak ground acceleration and peak ground velocity. It was assumed in the seismic PRA that 1g corresponds to 28 in/sec. As discussed in Chapter 2, this value may not be appropriate for the Millstone site. Note that the sliding displacement is proportional to the velocity raised to a power between 1 and 2, depending on the size of the vertical earthquake component.

Table 1 lists the coefficients of friction assumed in the analysis. These values were not reviewed in detail, although they appear to be reasonable.

The inclusion of the vertical earthquake component likely produces conservative results. For the 4-inch displacement considered in the sliding analysis, the time during which sliding will occur is approximately 0.3 seconds. In this time period the vertical component may reverse direction several times and its effect on horizontal sliding would be minimal.

In conclusion, there appears to be conservatisms and unconservatisms which tend to balance out. However, we recommend that the velocity to acceleration ratio be verified by the NRC since this assumption will have a major impact on the sliding capacities. Also, justification should be given that a 4-inch sliding displacement corresponds to the median capacity for buried piping.

The calculations for the component fragility values appeared to be more organized and consistent (i.e., between components) compared to similar calculations in previous PRAs. Based on our review, we have differences of opinion on several aspects of the component fragility analysis as discussed below. As discussed in Section 3.3, we found several small errors.

Factors of safety for earthquake component combinations were developed generically and are listed in Table 5-3 of Reference 1. Development of these factors is a complicated task and other engineers are likely to produce values different from those given in Table 5-3. We attempted to develop these factors directly ourselves and found that we disagree only slightly. However, one exception is the F_{ECC} value of 1.25 for Case 4 for the second design condition in Table 5-3 (corresponding to the situation when the SRSS value of the responses from the two horizontal directions was combined absolutely with the vertical component in the original design). Note that this design condition apparently applies only to balance of plant piping since the median SRSS rule was used for all other components. We calculate a value of 1.15 for this factor which is about 10 percent lower than the value of 1.25 given in Table 5-3.

In regards to the multi-directional effects factor for testing, we obtain correction factors that are approximately 10 percent lower for bi-axial testing (i.e., 0.77 compared to 0.853) and 13 percent lower for uniaxial testing (0.64 compared to 0.735). This difference is statistically small since there is considerable uncertainty that the methods for computing these factors (i.e., ours and theirs) are exact.

In contrast to the development of fragility values for structures, the uncertainty in response due to uncertainty in frequency is treated generically with logarithmic standard deviation values (which also include uncertainty in the mode shape) that vary from 0.10 to 0.20. This parameter should be developed specifically for each component as is done for structures. In situations where the median component frequency is close to a structure's natural frequency, the variability in response can be large due to uncertainty in the relative relationship between the two frequencies.

The ductility adjustment factor discussed in Chapter 2 for structures also has been applied to components in Reference 1. This is the first time that capacities of components have been modified for the effects of a duration or a ductility factor. In general, the same comments given for structures also apply to components.

3.2 REVIEW OF STRUCTURE FRAGILITIES

The results of the review of the fragility calculations for the Emergency Generator Enclosure, Pumphouse, Control Building, Engineering Safety Features Building, and Containment Crane Wall are given below.

Emergency Generator Enclosure

The following elements were analyzed for the Emergency Generator Enclosure:

- Sliding of the entire building
- Wall footing
- Slab at elevation 24 feet
- Roof slab
- Shear walls (in-plane and out-of-plane)
- Diesel generator pedestal stability

The inertial forces used in the analysis were developed from the original design analysis which consisted of a soil-structure interaction model, and no new dynamic analyses were performed. Forces were distributed to walls using the program WALLD7 developed by SMA. This structure is relatively stiff with a fundamental frequency near 9 Hz.

Sliding analyses were conducted to determine the incipient sliding capacity (i.e., 0.31g median) and the capacity corresponding to a 4-inch displacement (i.e., 1.30g median). The resistance against sliding included friction between the soil and the footings and side walls using a coefficient of friction equal to 0.55 (this is based on coarse grain soil containing no clay or silt) and the shear capacity of the soil enclosed between the buried walls. The effect of the vertical earthquake component was conservatively included in the analysis. The 4-inch displacement criterion corresponds to failure of buried piping as discussed above. The sliding analysis was based on the Newmark approximate approach and is subject to the limitations as also pointed out above.

The footings which support the EW direction walls span between the north wall footing and the vault base mat were the critical structural elements. Friction between the soil and footings was used to provide part of the resistance. Apparently a conservative coefficient of friction of 0.45 was used (compared to 0.55 used for sliding of the entire building). The footing capacity was found to be 0.88g, which appears to be on the conservative side.

Pumphouse

The following elements were analyzed for the Pumphouse:

- Sliding of the entire building
- Shear walls (in-plane and out-of-plane)
- Diaphragm (at elevation 14 feet)

A dynamic analysis of the Pumphouse using the basic properties developed in the original design (i.e., masses, stiffnesses, and geometry) was performed by SMA. Forces were distributed to the walls using the program WALLDI. This structure is relatively stiff with fundamental frequencies of 9.5 Hz and 14.8 Hz in the EW and NS directions, respectively.

Sliding analyses were conducted to determine the incipient sliding capacity (i.e., 0.48g median) and the capacity corresponding to a 4-inch displacement (i.e., 1.30g median). Only sliding in the westward direction is considered possible (in the other directions either the structure is keyed into or butts against rock). Only friction between the concrete mat and the foundation was assumed to resist sliding. A coefficient of friction equal to 1.1 was used, which was an average value for concrete on excavated rock or raked concrete fill (i.e., coefficient equal to 1.2) and concrete on intact rock (i.e., coefficient equal to 1.0). Similar to the sliding analysis for the Emergency Generator Enclosure, a 4-inch displacement criterion was assumed and the sliding capacity was calculated using the Newmark approximate approach. However, it was noted that a 1-inch displacement capacity was assumed at minus two standard deviations below the median, which is different from the corresponding value of 2 inches assumed in the sliding analysis for the Emergency Generator Enclosure. This is a minor inconsistency.

The exterior shear walls were analyzed for both in-plane loads and out-of-plane fluid and soil loads. The single north wall is the weakest wall corresponding to a median capacity of 1.6g. The diaphragm at the pump support level was also analyzed and found to have a median capacity of 1.5g. The critical section near the north wall contains numerous openings which controls the diaphragm capacity.

No mention of the capacity of the roof slab was found. This slab also has numerous openings. In contrast to the crib house roof slab at Zion, which was a critical component, the in-plane forces in the diaphragm at Millstone are resisted by buttresses on the intake side of the building. Thus it is unlikely that the roof slab will be a significant contributor.

Control Building

The following elements were analyzed for the Control Building:

- Sliding of the entire building
- Diaphragm

- Roof slab
- Shear walls
- Block walls

A dynamic analysis of the Control Building was performed by SMA. They found that the structural mass was about 30 percent larger than the mass used in the original design analysis. This was explained by construction changes made since the original analysis was conducted. Forces were distributed to the walls using the program WALLDI. This structure is relatively stiff with fundamental frequencies of 8.9 Hz and 8.3 Hz in the EW and NS directions, respectively.

Analyses were conducted to determine the incipient sliding capacity (i.e., 0.43g median) and the capacity corresponding to a 2-inch displacement (i.e., 1.2g median). A 2-inch displacement criterion was used because of potential impact with the turbine building. Shear keys add additional capacity, which explains in part the 1.2g capacity for only a 2-inch displacement. (Note that other structures have a 1.3g capacity for a 4-inch displacement criterion.)

The shear walls were analyzed for in-plane loads. Their capacities are higher than the 1.0g median capacity for the diaphragm at elevation 64'-6" which is controlled by a section with numerous openings adjacent to the west exterior wall. A systems ductility ratio of only 1.3 was assumed, which seems conservative.

The block walls adjacent to critical safety-related equipment were analyzed. These walls are reinforced and supported by a steel frame. A dynamic analysis of a critical panel was conducted by SMA and found to have a 2.0g median capacity.

<u>Engineering Safety Features Building</u>

The following elements were considered for the Engineering Safety Features Building:

- Sliding of the entire building
- Diaphragm
- Shear walls

A dynamic analysis of the Engineering Safety Features Building was performed by SMA using the basic properties developed in the original design (i.e., masses, stiffnesses, and geometry). Slight discrepancies were found by SMA regarding the center of rigidity and induced torsional forces. Modifications were made to the model. Forces were distributed to the walls using the program WALLDI. This structure is very stiff with a fundamental frequency of 12.8 Hz.

The strength of various shear walls and the critical diaphragm section were analyzed and the capacities were found to exceed 2.0g median ground acceleration for these failure modes.

The potential for sliding was considered for this building. In three of the four directions it was argued that sliding was not a realistic failure mode. In the west direction (i.e., toward the containment), an incipient sliding analysis was performed. Because of the high resulting capacity, only the shear key and support provided by the adjacent containment base mat were assumed to provide resistance. The high buoyant force and vertical acceleration component eliminated the friction capacity between the soil and base mat. This portion of the analysis appears to be on the conservative side.

Because the incipient sliding capacity was found to be high (i.e., 1.7g median), no sliding displacement analysis was performed.

Containment Crane Wall

A dynamic analysis of the Containment Building was performed by SMA using the basic properties developed in the original design. Median properties and seismic input were used to obtain gross forces acting on the internal structures. A refined model of the internal structures including the crane wall elements was developed by SMA. Forces from the dynamic analysis were applied statically to the model. As each element reached its yield capacity the model was modified, and an additional incremental load was applied until the maximum resistance was obtained. A system ductility ratio of 3 was assumed in the analysis.

The capacity of the crane wall was determined to be 2.2g median ground acceleration. This is considerably higher than the 0.87g capacity calculated in the original analysis. The revised value is more realistic.

3.3 REVIEW OF COMPONENT FRAGILITIES

The results of the review of the fragility calculations for the 4160 V Switchgear, Service Water Piping, Emergency Diesel Generator, RPV Core Geometry, and the Control Rod Drive Mechanisms are given below.

4160 V Switchgear

Both relay chatter and relay trip failure modes were developed for the 4160 V Switchgear, which is located on the base mat in the Control Building (i.e., elevation 4'-6"). The relay chatter median capacity of 0.88g is based on the assumption that chatter will occur at a level 20 percent higher than the qualification level (based on judgment). The uncertainty logarithmic standard deviation for this estimate is only 0.08. A value between 0.2 and 0.4 is probably more appropriate. We also disagree slightly with the median factors of safety assumed for earthquake components and building response spectral shape. In conclusion, we estimated the median relay chatter capacity to be 0.85 (compared to 0.88g) with logarithmic standard deviation for randomness and uncertainty to be 0.26 and 0.47, respectively (compared to 0.29 and 0.40, respectively in the SMA report).

The relay trip capacity is based on generic data developed from the Army Corps of Engineers shock tests. The extrapolation of this data to seismic fragility values has been recently questioned (Ref. 15). However, the capacity for this mode is relatively high (i.e., 3.09g median). In addition, a very large logarithmic standard deviation for uncertainty has been used (i.e, 0.81). It is unlikely that the median capacity for this failure mode is less than 1.5g; although, this conclusion is speculative and not based on any data.

Service Water Piping

The critical failure mode for the service water piping is displacement failure caused by sliding of the connecting buildings. Capacities of the piping within the buildings is relatively high and failure in the ground due to wave passage effects in the surrounding soil is unlikely at accelerations in the range of potential sliding failures. The analyses of the sliding failure mode for the various safety-related structures are discussed in Section 3.2.

It is our understanding that a concrete wall retains soil through which the service water piping pass between the pumphouse and the pl .t. Failure of this wall may lead to failure of the adjacent piping. A fragility analysis should be conducted for this wall.

Emergency Diesel Generator

The capacity of the Emergency Diesel Generator is controlled by the strength of the lube oil cooler anchor bolts. This component is located in the Emergency Generator Enclosure at elevation 24'-6". We are unable to confirm the reasonableness of the fragility calculations since the seismic stress report (Ref. 16) was not provided with the package of calculations. This reference is needed to verify the fragility parameter values.

The soil-structure interaction (SSI) factor of safety was assumed to be 1.3. The basis for this value is not given. Since the diesel generators are supported on their own foundations separate from the Emergency Generator Enclosure, a separate design analysis was performed for them. We speculate that SMA obtained a copy of this analysis and judged that the modeling of SSI resulted in a factor of safety of 1.3. We have no other basis to determine whether this value is reasonable.

RPV Core Geometry

The upper support plate was determined to be the weakest element in the RPV core. A total of seven potential failure modes were evaluated. It was assumed in the analysis that the code allowable stress corresponds to failure. This assumption acknowledges that the faulted design values allow significant inelastic deformation. Since deflection limits are not included, it is assumed by SMA that inelastic deformation does not constitute a functional failure and that Westinghouse has demonstrated satisfactory control rod insertion at the allowable loads. The only increase incorporated in the strength factor is the difference between median properties and nominal values used in the design (i.e., a factor between 1.20 and 1.25).

In deve[illegible]ng the structural response factors a factor of safety is developed fo[illegible] difference between the median ground response spectrum and the respo[illegible]ctrum used in the original design. A spectral value of 0.51g was used for the original design value (corresponding to 4.7 Hz at 5 percent damping). Based on Figure 3.7B-6 of the Millstone Nuclear Power Station Unit 3 FSAR the value is approximately 0.45g. This difference lowers the median ground acceleration capacity to 0.87g instead of 0.99g. No other significant differences were found for this component.

Control Rod Drive Mechanisms

Bending in the control rod was determined to be the weakest element in the Control Rod Drive Mechanisms. Similar to the upper support plate in the RPV, the allowable stress was assumed to be the failure stress. An increase of 25 percent was included to reflect the difference between median properties and the nominal values used in the design.

The same apparent mistake made in determining the structural response factor for the RPV Core Geometry (see discussion above) was also made for this component. If the spectral value is corrected, the median capacity is 0.88g instead of 1.00g.

TABLE 3-1. COEFFICIENTS OF SLIDING FRICTION ASSUMED IN THE SEISMIC AREA

Condition	Coefficient
Concrete against soil with silt and clay	0.45
Concrete against soil without silt and clay	0.55
Smooth concrete against smooth concrete	0.80
Concrete poured against rough concrete	1.00
Foundation against intact rock	1.00
Foundation against excavated rock or raked concrete	1.20

4. CONCLUSIONS AND RECOMMENDATIONS

Based on review of Reference 1 and the supporting calculations we generally believe that the revised fragility parameter values are realistic. However, we have found various problems which may affect the results of the risk analysis. We recommend that the NRC investigate the impact of these problems on the resulting frequency of core melt and other risk consequences. From the results of our review we recommend the following.

1. NUSCO should provide justification that a 4-inch displacement corresponds to the median capacity of buried piping. This justification should be reviewed by the NRC.

2. The NRC should determine if the range of earthquakes contributing to the risk analysis are greater than magnitude 5.3 to 6.3. If this is the case, then the effective ductility ratios will be lower and a different response spectrum shape should be used. This will result in lower median capacity values.

3. Because the structures at Millstone have high natural frequencies, the dependence of the Inelastic Energy Absorption factor on frequency should be incorporated into the analysis. NUSCO should revise their Inelastic Energy Absorption factor estimates to reflect the frequency characteristics of the structures. For estimation purposes, a lower bound on the Inelastic Energy Absorption factor is 1.0.

4. The NRC should determine if the site-specific spectrum used in the fragility analysis is appropriate. See Table 2-1 and Figure 2-1 for a comparison of different response spectra.

5. The NRC should investigate the correlation between failure modes to determine if it significantly affects the risk analysis.

6. The NRC should determine if the velocity to acceleration ratio of 28 in/sec/g is a representative median value for the Millstone site. If the value is significantly higher, then the structure sliding capacities should be reevaluated. A conservative bounding assumption is that the median capacity is inversely proportional to the square of the velocity to acceleration ratio.

7. Table 4-1 lists revised fragility values based on our review. The impact of these values on risk should be investigated by the NRC. These values do not include adjustment for the effects of larger earthquake magnitudes, the effects of the dependency of the Inelastic Energy Absorption factor on the frequency of structures, or the effects of site-specific spectra (see Nos. 2, 3, 4 above).

8. NUSCO should provide Reference 2 and the fragility analysis for the Emergency Diesel Generator should be reexamined in light of this information.

9. NUSCO should perform a fragility analysis for the concrete wall which retains soil through which the service water piping passes from the pumphouse to the rest of the plant.

10. As recommended in our first review (Ref. 2), a study should be conducted after the plant is completed to determine if any non-safety related structures or components could fail, fall, and impact the safety-related items in the plant.

TABLE 4-1. REVISED FRAGILITY PARAMETER VALUES

Component/Parameter	Revised Values	Reference 1 Values
<u>4160 V Switchgear</u>		
(Chatter Failure Mode)		
Median	0.85g	0.88g
β_r	0.26	0.29
β_u	0.47	0.40
<u>RPV Geometry</u>		
Median	0.87g	0.99g
<u>Control Rod Drive Mechanism</u>		
Median	0.88g	1.00g

REFERENCES

1. Wesley, D. A., et al., "Seismic Fragilities of Structures and Components at the Millstone 3 Nuclear Power Station," Prepared for Northeast Utilities, Structural Mechanics Associates Report No. SMA 20601.01-R1-0, March 1984.

2. Reed, J. W., "Review of the Millstone Unit 3 Probabilistic Safety Study--Seismic Fragility, Wind, and External Flooding," Prepared for Lawrence Livermore National Laboratory, December 22, 1983.

3. Power Authority of the State of New York and Consolidated Edison Company of New York, Inc., "Indian Point Probabilistic Safety Study, Units 2 and 3," Dockets 50-247 (Unit 2) and 50-286 (Unit 3), March 5, 1982.

4. Commonwealth Edison Company, "Zion Nuclear Plant Units 1 and 2 Probabilistic Safety Study," Dockets 50-295 (Unit 1) and 50-304 (Unit 2), September 8, 1981.

5. Kolb, G. J., et al., "Review and Evaluation of the Indian Point Probabilistic Safety Study," Prepared for U.S. Nuclear Regulatory Commission, NUREG/CR-2934, December 1982.

6. Newmark, N. M., "A Study of Vertical and Horizontal Earthquake Spectra," WASH 1255, Nathan M. Newmark Consulting Engineering Services, prepared for U.S. Atomic Energy Commission, April 1973.

7. Kennedy, R. P., W. H. Tong, S. A. Short, "Earthquake Design Ground Acceleration Versus Instrumental Peak Ground Acceleration," prepared for Nathan M. Newmark Consulting Engineering Services, Structural Mechanics Associates Report No. SMA 12501.01R, December 1980.

8. Philadelphia Electric Company, Limerick Generating Station Severe Accident Risk Assessment, 1983.

9. M. A. Azarm, et al., "A Preliminary Review of the Limerick Generating Station Severe Accident Risk Assessment, Volume I: Core Melt Frequency," Engineering and Risk Assessment Division, Department of Nuclear Energy, Brookhaven National Laboratory, 1984.

10. R. P. Kennedy, et al., "Engineering Characterization of Ground Motion Effects of Characteristics of Free-Field Motion on Structural Response," SMA 12702.01, prepared for Woodward-Clyde Consultants, 1983.

11. Bernreuter, D. L., "Seismic Hazard Analysis Application Methodology, Results and Sensitivity Studies," U.S. Nuclear Regulatory Commission, NUREG/CR-1582, UCRL-53030, Vol. 4, October 1981.

12. McGuire, R. K., and T. P. Barnhard, "The Usefulness of Ground Motion Duration in Predicting the Severity of Seismic Shaking," Proceedings of the 2nd U.S. National Conference on Earthquake Engineering, Stanford University, August 22-24, 1979.

13. Bernreuter, D. L., J. B. Savy, R. W. Mensing, D. H. Chung, "Seismic Hazard Characterization of the Eastern United States: Methodology and Interim Results for Ten Sites," U.S. Nuclear Regulatory Commission, NUREG/CR-3756, UCRL-53527, Draft, April 1984.

14. Joyner, W. B. and D. M. Boore, "Peak Horizontal Acceleration and Velocity from Strong Motion Records Including Records from the 1979 Imperial Valley California Earthquake," Bulletin of the Seismological Society of America 71, December 1981.

15. Kana, D. D, and D. J. Pomerening, "The Use of Fragility in Seismic Design of Nuclear Plant Equipment," Prepared for U.S. Nuclear Regulatory Commission, Southwest Research Institute Report No. SWRI-6582-004, March 1984.

16. Colt Industries Operating Corporation, "Seismic Analysis For Emergency Diesel Generator Systems, Millstone Unit No. 3 of NUSCO," Fairbanks Morse Engine Div., Analytical Engineering Dept., Approved by Stone & Webster Engineering, B. A. Bolton, February 14, 1979.