NORTHEAST UTILITIES

THE CONNECTICUT LIGHT AND POWER COMPANY
WESTERN MASSACHUSETTS ELECTRIC COMPANY
HOLYOKE WATER POWER COMPANY
NORTHEAST UTILITIES SERVICE COMPANY
NORTHEAST NUCLEAR ENERGY COMPANY

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July 31, 1984

Docket No. 50-423
A04142

Director of Nuclear Reactor Regulation
Mr. B. J. Youngblood, Chief
Licensing Branch No. 1
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Reference: B. J. Youngblood letter to W. G. Counsil, Request for Additional Information, dated June 19, 1984.

Gentlemen:

Millstone Nuclear Power Station, Unit No. 3
Probabilistic Safety Study (PSS)

The above reference requested Northeast Nuclear Energy Company (NNECO) to submit additional information which resulted from the staff's review of information contained in Amendment No. 2 to the Millstone Unit No. 3 PSS. Enclosed please find documentation of all questions posed to NNECO along with our formal responses herein. We trust you will find this information fully responsive.

Very truly yours,

NORTHEAST NUCLEAR ENERGY COMPANY

W. G. Counsil

W. G. Counsil
Senior Vice President

C. F. Sears

By: C. F. Sears
Vice President
Nuclear and Environmental Engineering

B001
1/40

Question 720.74

As reported in the Dames and Moore hazard report, two ground acceleration attenuation equations were derived from MM intensity attenuation using the relationship of Klimkiewicz (1982). The staff questions the validity of these relationships. These ground acceleration relationships predict lower near source accelerations for the same magnitude when compared to those of Campbell (1981a) whose near source eastern United States values are determined using near source western United States strong motion data.

a) Why would near source ground motion (for the same m_b) be systematically lower in the east compared to the west, as you have assumed for these two equations?

Response to Question 720.74 1a)

The Applicant has used the intensity attenuation of Klimkiewicz (1982) in deriving two acceleration equations, labelled the "AI" and "AID" attenuations. In comparing estimates from these attenuations to Campbell's (1981a) attenuation, which uses data from California, the question is raised as to why near-source ground motion in the east would be less than that in the west. It should be noted that the Campbell (1981 a) equation was developed as a ground motion model for the Central United States based on near-source acceleration data. This model was chosen by Applicant for use in the seismic hazard analysis as an example of the wide range of opinion of ground motion in the near-field for the east coast. The Applicant feels the most appropriate comparison to use to resolve the question of difference between east and west near source ground motion is to compare the attenuation of the AI and AID equations and the Campbell (1981b) equations. The 1981b equations were developed by Campbell to characterize near-source attenuation specifically derived for California.

Figure 720.74-1 shows a comparison between estimates made using the AI and AID attenuations, and those from Campbell (1981b), using both his 'constrained' and 'unconstrained' models for California. These estimates are made at a distance of 5 km (epicentral distance in the case of AI and AID, surface fault distance in the case of Campbell). Figure 720.74-2 shows a similar comparison at a distance of 10 km. This indicates the same general results. To make a comparison on the basis of m_b, the conversion from moment magnitude to m_b recommended by Campbell (1981a) was used.

Peak accelerations from the AID equation equals or exceeds Campbell's near estimates at all magnitudes above 5.3, and the peak accelerations AI equation exceeds Campbell's estimates at magnitudes above 5.8. For Millstone 3 the majority of seismic hazard comes from magnitudes above 5.5, and therefore the higher accelerations predicted by the AI and AID equations is conservative. In that little seismic hazard comes from earthquakes of magnitude less than 5.5 there is no significant effect on seismic hazard risk from Campbell's accelerations being higher than the AI and AID accelerations.

A further justification of the attenuation equations used in the Applicant's study can be made by comparisons with data. Figure 720.74-3 is a plot of peak acceleration calculated using the four equations used in the Dames & Moore study, and peak acceleration data derived from eastern US strong motion records. The curves are plotted for $m_b = 5.0$, and the acceleration data have been scaled to $m_b = 5.0$ using the factor exp $(-1.1(5.0-m_b^*))$, where m_b^* is the magnitude of the earthquake generating the records. From the peak acceleration comparison in Figure 720.74-3, the Nuttli and Campbell equations are higher than the AI and AID equations at distances greater than 10 km; and at distances closer than 6 km, the AID attenuation gives the highest estimates.

Noting the reasoning stated above and the information presented in Figures 720.74-1 thru 720.74-3 it is clear that the range of attenuation functions used in the Dames & Moore report is adequate to characterize the seismic hazard for Millstone 3.

Question 720.74 1b)

The staff is concerned that the intensity attenuation of Klimiewicz, (1982) is strongly influenced by intensities less than MMI=IV. It is likely that ground motion is not linear with MM intensity (see for example actual results of Trifunac and Brady, 1975, Murphy and O'Brien, 1978). Weston Geophysical has noted in the New Brunswick earthquake report (pg. 138) that the intensity attenuation model may be low for near epicentral distances. If intensities less than MMI=IV were removed from the intensity-distance set, would the intensity attenuation equation and thus the two ground acceleration attenuation equations result in higher seismic hazard curves compared to those assumed? If so, revise your seismic hazard curves accordingly.

Response to Question 720.74 1b)

Intensity attenuation models (Klimkiewicz, 1982) were derived using all observed MM intensities for several regional Northeastern United States earthquakes. Elimination of certain lower intensities ground motions result in unrealistic attenuation models that are not consistent with theoretical models of attenuation of seismic waves in earth media.

In lieu of performing an analysis on all intensity data available for the northeastern US, it is more informative to show the effects of removing some intensities from the data set using just one earthquake. Figure 720.74-4 shows intensity data from the 1944 Cornwall-Massena earthquake, which has one of the least disperse sets of observations in northeastern North America. The solid line is a fit using all the plotted data; the dotted line is fit through intensities IV and above. It should be noted that intensities plotted as III are an undifferentiated combination of intensities I through III.

At distances less than 15 km and greater than 300 km the predictions of the two models differ. The dotted curve which excludes ground motion less than intensity IV is obviously incorrect when it shows increasing intensity at greater distances. At close distances (less than 15 km) there are no data to support either model, but theoretical considerations clearly give more weight to the solid curve by the following reasoning.

It can reasonably be assumed that observed intensities are correlated to the largest amplitude phase of a seismic coda. In the northeast, highest amplitudes for regional earthquakes are observed in part of the coda corresponding to higher mode surface waves that propagate as an Airy phase and are referred to as the Lg phase. Attenuation of the Lg phase is given in equation 1 (Ewing et al., 1959):

$$A(r) = A_o \left[\sin r\right]^{-1/2} r^{-1/3} \exp\left[\nu r\right] \qquad 1$$

Empirically derived attenuation models should be consistent with the theoretical model inasmuch as model coefficients have proper sign and, ideally, proper magnitude. It is demonstrated in Figure 720.74-[illegible] that elimination of lower intensities results in an unrealistic model.

It is further noted that regional attenuation studies (Pulli, 1983) in the Northeast conclude that anelastic attenuation for 1 hz waves is 0.0015/km. The anelastic attenuation term in the empirical intensity attenuation model (Klimkiewicz, 1982) is consistent with this regional value derived from interpretation of seismograms. In addition, the geometrical spreading term is consistent with that given in the theoretical model, equation 1.

We conclude that the exclusion of intensity data less than IV would not provide better estimates of intensity for northeastern US earthquakes. The results presented in the response to question 720.74-1a confirm that the Klimkiewicz (1982) intensity equation is appropriate for estimating seismic ground motions in northeastern North America.

Additionally, if the AI and AID equations were removed from the analysis, and total weight were given to the Nuttli and Campbell attenuation equations in the Dames & Moore report, the hazard curves would increase only marginally. For example, at the 10^{-4} level, the peak acceleration would increase from 0.17 g to about 0.20g. Thus the effect of the AI and AID attenuations in the analysis is not large. It is the position of the Applicant that 1) since the Klimkiewicz (1982) equation is appropriate and 2) even if the AI and AID equations were not used, the change in peak acceleration would be small and therefore there is no justification for revising the seismic hazard curves.

Question 720.74 1c)

Why were relationships such as Battis (1981), which would predict more severe strong motion in the east compared to the west, excluded from your hazard analysis?

Response to Question 720.74 1c)

Several attenuation equations have been published which would predict more severe strong motion in the east compared to the west. Battis (1981) gives one example mentioned in Question 720.74(c) in which peak acceleration estimates in the east exceed those in the west at all distances.

The Battis (1981) paper relies on estimating peak accelerations at short distances in the east under the assumption that near-source intensities are the same, east and west, and fixes peak acceleration at the limit of felt area to 6 cm/sec^2. A set of artificial accelerations at 10 km and at the limit of felt area are generated, and a functional form of arbitrary shape is fitted by least-squares regression analysis. The method is interesting but unproven; the comparison Battis makes with other equations and data is for California, where near-source assumptions are not critical since the peak acceleration generating function guarrantees a good fit at close distances.

In Battis' example developed for the central U. S., an intensity-to-magnitude conversion is required both for the central U.S. and California. The functions used by Battis (referencing Brazee, 1976, for California and his own work for the central U. S.) are not supported by other researchers in the field. One can only attribute the different functions adopted by Battis as the result of variability resulting from incomplete studies, not a final position on near-source ground motion in the two regions.

Indeed, the major thrust of Battis' paper is in presentation of his methodology not in the implications for ground motion estimates in the eastern U.S. In fact no comparisons are made to other studies in that area. No reasons are given as to why ground motions might be different at small distances in the two areas, nor even to justify a difference. This is not a criticism of Battis' study; his purpose was to demonstrate a methodology, not to produce attenuation equations for the eastern U. S. which would stand up under intense scrutiny.

Given the arbitrary way in which the Battis example was derived for the eastern U.S. it is not appropriate to place any weight on the Battis curves. These final equations for the central U. S. were published as examples only. The many caveats in Battis' paper make it clear that the ground motion estimates are not intended for use in engineering applications.

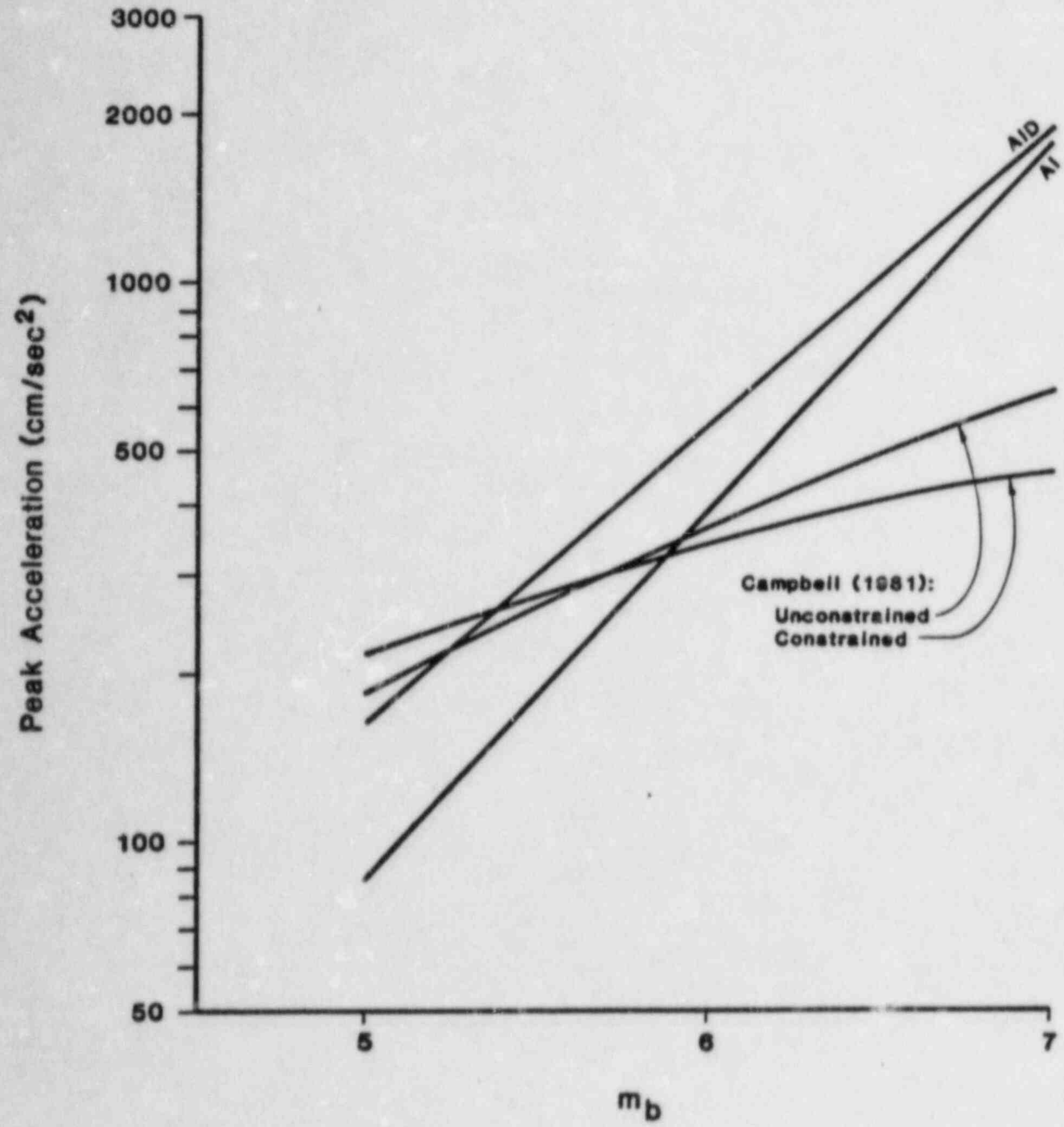


Figure 720.74-1
Peak Acceleration Versus m_b
at R=5 km

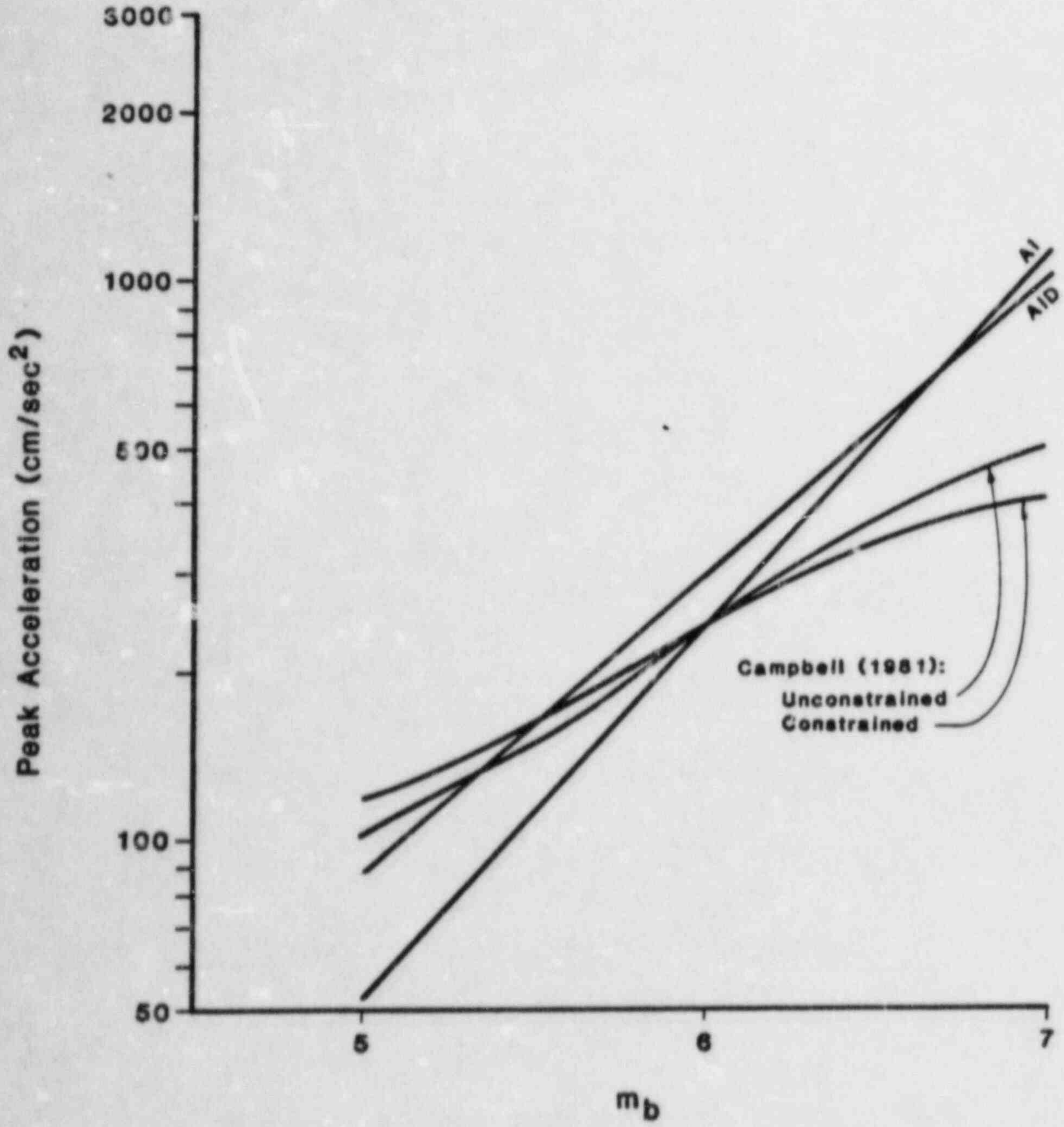
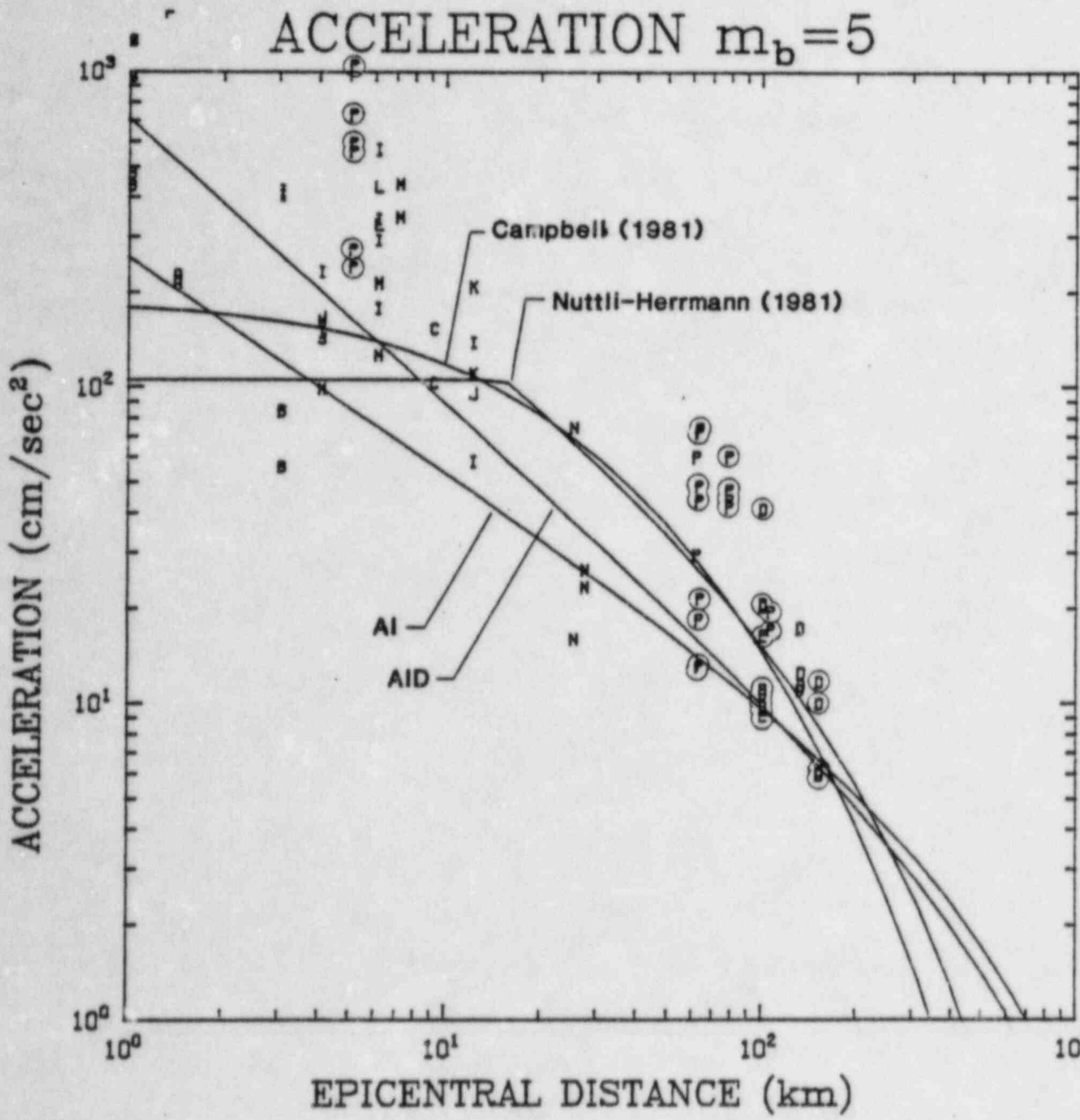


Figure 720.74-2
Peak Acceleration Versus m_b
at R=10 km



Key: Symbol	Date	m_b	Symbol	Date	m_b	Symbol	Date	m_b
A	073073	2.7	G	082778A	2.7	M	050682	4.0
B	080373	2.6	H	082778A	2.7	N	061682	4.6
C	061375	4.2	I	033182	5.0	O	072882	3.7
D	032576	5.0	J	040282	4.3	P	011982	4.4
E	032576A	4.5	K	041182	4.1	(P) Data from dams		
F	082778	2.7	L	042882	3.4			

Figure 720.74-3
Peak Acceleration Data (Scaled to m_b=5.0)
and Attenuation Equations

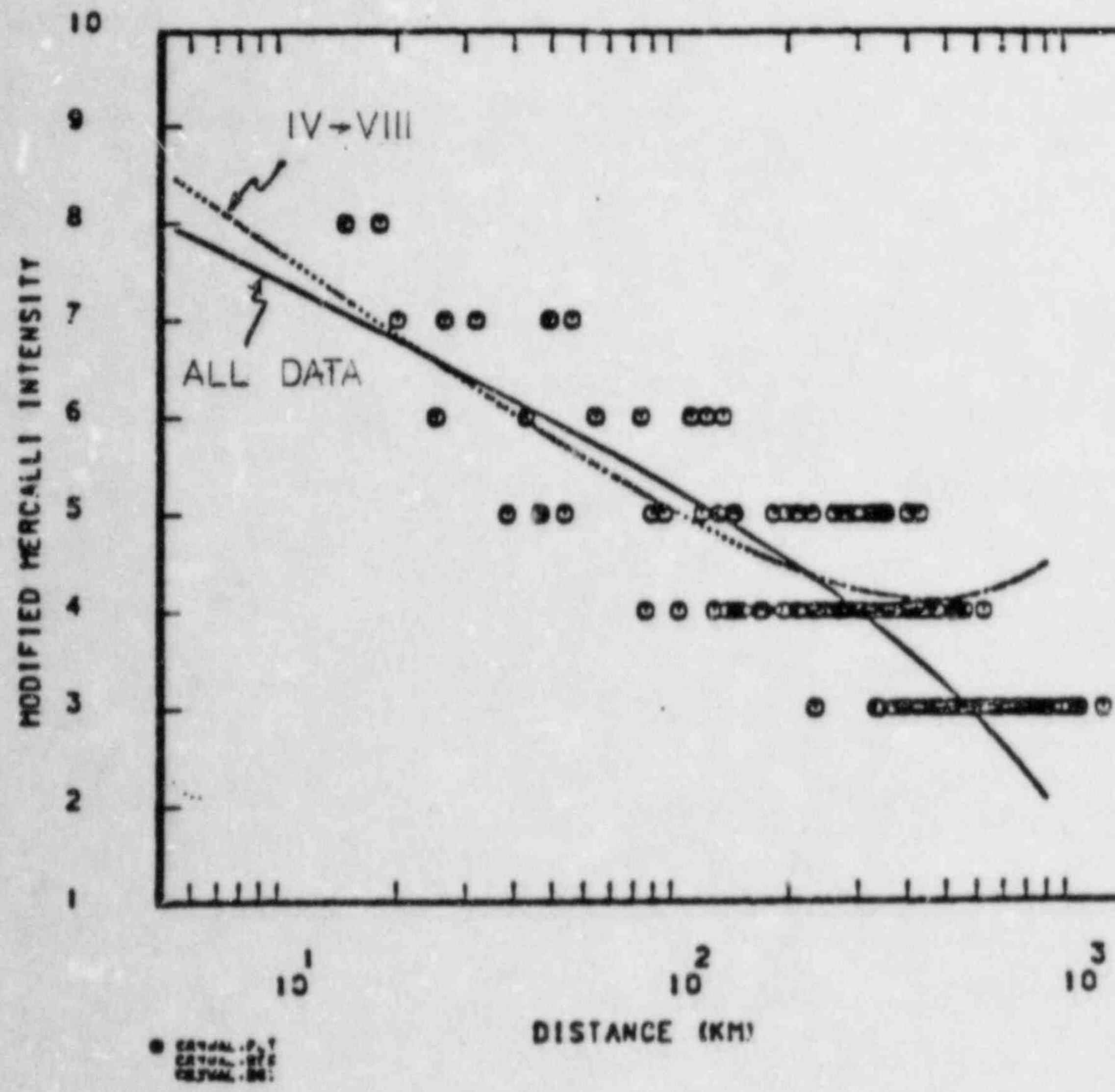


Figure 720.74- 4

Question 720.75

As noted in the attached internal staff memorandum (Enclosure 2), the staff questions the validity of the Weston (1982) epicentral intensity to magnitude equation. This equation is given a weight of 50% in your hazard analysis. Provide a response to the concerns in the attached memorandum, and if necessary, revise the seismic hazard curves assumed in your analysis.

Response to Question 720.75

The most appropriate equation to use to convert Modified Mercalli Intensity (MMI) to body wave magnitude (m_b) in the northeast is not resolvable theoretically, because intensity is an observed, empirical quantity. Some insight into the appropriate conversion can be obtained by examining all available earthquake data for the northeast that have both an instrumentally measured magnitude (m_b) and an assigned epicentral intensity. Table 720.75-1 is a listing of events meeting this criteria. These events are compiled primarily from the Chiburis catalog with additional events from Street and Turcotte and Yankee Atomic Electric Company Report #1331 (YAEC #1331). The events from YAEC-1331 come from the Weston Geophysical Corporation (WGC) catalog, and in particular are a subset of the events used in determination of the WGC conversion that are not found in either the Street or Chiburis data set. Figure 720.75-1 shows a plot of epicentral intensity versus body wave magnitude (m_b) for these events, along with the Nuttli and WGC intensity to magnitude conversions.

Table 720.75-2 presents summary statistics for the entire data set and for subsets. These statistics describe how well the conversions fit the data. In regression analysis, the explained sum of squares is defined by $(y - y)^2$, and the unexplained sum of squares is defined by $(yi - y)^2$ where y is the estimated magnitude from the conversion, y is the observed magnitude and y is the average magnitude in the subset. How well the conversions fit can be determined by comparing the unexplained sums of squares for both the Nuttli and WGC conversion and also by comparing the correlation coefficients for each conversion.

While the results vary among the individual subsets of data, the overall conclusion is that the WGC conversion provides a better fit, or at least as good a fit, to the data, as does the Nuttli conversion. This is consistent with the idea that characteristics of northeast US earthquakes, at least insofar as intensities are concerned, are slightly different from those of other earthquakes in the US.

As a result of these comparisons, the procedure used in the Dames & Moore report is justified. Specifically, the data from New England do not allow differentiation between the Weston Conversion and the Nuttli conversion, and it is appropriate to weight each equally.

TABLE 720.75-1

Year	Lat.	Long.	m_b	MMI	
1924	47.60	69.70	5.50	8.00	
1925	47.60	70.10	6.60	9.00	
1929	42.90	78.40	5.20	8.00	
1929	44.50	56.30	6.70	10.00	ST
1931	43.40	73.70	4.70	7.00	
1935	46.78	79.07	6.20	7.00	
1938	40.10	74.50	3.90	5.00	WOC
1939	47.80	70.00	5.60	6.00	WOC
1940	41.60	70.80	2.60	5.00	
1940	43.80	71.30	5.50	7.00	
1940	43.80	71.30	3.60	4.00	ST
1940	43.80	71.30	3.70	4.00	ST
1941	43.80	71.30	2.70	4.00	ST
1943	45.30	69.60	4.40	5.00	
1944	45.00	74.90	5.90	6.00	
1947	45.20	69.20	4.40	5.00	WOC
1949	44.80	70.50	4.50	5.00	
1951	41.25	74.25	3.80	5.00	WOC
1952	48.00	69.80	4.70	5.00	ST
1957	43.60	69.80	4.80	6.00	
1963	42.50	70.80	3.90	5.00	WOC
1963	42.70	70.80	2.40	5.00	WOC
1964	43.60	71.50	1.80	4.00	WOC
1964	43.30	71.90	2.60	5.00	WOC
1966	42.80	78.20	4.70	6.00	
1967	42.90	78.20	3.90	6.00	
1967	44.40	69.90	2.90	4.00	WOC
1967	44.38	69.87	3.40	5.00	WOC
1968	37.30	80.80	4.10	4.00	
1968	34.00	81.50	3.70	4.00	
1968	45.30	74.10	3.20	5.00	
1968	41.40	72.50	3.30	5.00	
1968	39.70	74.60	2.50	5.00	
1969	36.10	83.70	3.50	5.00	
1969	43.80	71.40	2.60	5.00	
1969	43.30	78.20	2.80	4.00	
1969	46.40	75.20	4.20	5.00	
1970	45.80	66.10	3.30	3.00	
1970	42.90	71.90	2.60	4.00	

TABLE 720.5-1 Continued

Year	Lat.	Long.	m_b	MMI	
1971	45.10	73.40	3.20	4.00	
1971	43.80	74.50	3.90	5.00	
1971	34.80	83.00	3.80	4.00	
1971	45.70	75.20	3.20	4.00	
1971	42.70	71.20	2.30	5.00	
1971	50.20	66.40	3.20	4.00	
1971	46.60	72.60	2.20	3.00	
1971	45.80	76.60	3.00	4.00	
1971	46.20	74.60	3.90	5.00	
1972	37.60	77.70	3.40	5.00	
1972	46.20	77.60	3.20	4.00	
1972	45.90	64.70	3.00	5.00	
1972	45.50	74.30	2.60	3.00	
1972	45.80	64.70	2.20	3.00	
1972	45.60	75.20	3.90	4.00	
1973	39.70	75.40	3.80	5.00	
1973	37.30	77.70	2.50	4.00	
1973	45.30	70.90	4.80	6.00	
1974	41.60	73.90	3.30	6.00	
1974	33.90	82.50	4.30	5.00	
1974	41.70	71.60	2.50	2.00	
1975	44.90	74.60	2.50	4.00	
1975	47.80	55.20	3.20	4.00	
1975	45.70	74.20	3.10	4.00	
1975	44.90	73.70	4.20	6.00	
1975	43.40	79.80	3.00	3.00	
1975	46.50	76.20	4.10	4.00	
1975	41.40	73.80	2.30	3.00	
1975	42.70	70.90	2.40	3.00	
1975	44.10	70.20	2.20	3.00	
1975	41.60	73.90	2.20	2.00	
1975	43.90	74.60	3.90	4.00	
1975	47.00	78.80	3.80	3.00	
1975	40.20	69.74	3.30	4.00	MDC
1976	41.60	71.20	2.90	5.00	
1976	41.00	74.40	2.40	5.00	
1976	41.70	70.00	2.80	5.00	
1976	40.80	74.00	3.10	6.00	
1976	44.20	70.10	2.40	3.00	
1976	41.50	72.50	2.20	3.00	
1976	41.50	71.00	2.70	4.00	
1976	45.20	74.10	2.90	4.00	
1976	47.80	69.80	4.20	5.00	
1976	41.50	72.10	2.20	2.00	
1977	46.00	74.40	3.40	4.00	
1977	49.80	67.10	3.90	4.00	
1977	41.80	70.70	3.10	4.00	
1977	43.20	71.70	3.20	4.00	
1977	41.84	70.70	2.40	3.00	MDC

TABLE 720.5-1 Continued

Year	Lat.	Long.	m_b	MMI
1978	44.00	70.50	3.20	4.00
1978	46.90	70.30	2.90	3.00
1978	46.30	74.10	4.20	5.00
1978	46.40	74.10	3.80	4.00
1978	43.50	79.70	2.10	2.00
1978	47.70	70.10	3.10	3.00
1978	41.10	74.20	2.90	4.00
1978	39.80	76.00	3.10	5.00
1978	45.70	74.40	3.80	4.00
1978	47.10	70.90	2.80	3.00
1978	47.60	70.10	2.90	3.00
1978	42.90	70.80	2.30	2.00
1978	42.50	71.50	2.00	2.00
1978	47.50	70.50	2.00	2.00
1978	47.60	70.40	2.70	3.00
1978	40.10	76.10	3.00	6.00
1978	45.00	69.50	2.20	2.00
1978	44.50	73.90	2.50	4.00
1979	44.80	73.20	2.80	2.00
1979	40.30	74.30	3.50	4.00
1979	40.70	74.50	3.10	3.00
1979	47.70	70.10	3.10	3.00
1979	44.00	69.80	4.00	5.00
1979	45.20	66.00	3.20	4.00
1979	43.00	71.20	3.10	3.00
1979	43.30	70.40	3.50	4.00
1979	47.70	69.90	5.00	5.00
1979	41.20	73.70	2.20	3.00
1980	48.70	68.10	4.10	4.00
1980	43.60	75.20	3.50	4.00
1980	47.50	70.70	3.60	4.00
1980	42.10	83.10	3.30	5.00

TABLE 720.75-2

Conversion	Subset	Number of Events	Average Magnitude	$(\hat{y} - \bar{y})^2$	$(\hat{y} - y_i)^2$	R
Nuttli	$I_o \geq 2.0$	120	3.40	95.49	79.72	0.74
WGC				110.52	49.57	0.83
Nuttli	$I_o \geq 4.0$	90	3.70	57.82	66.79	0.68
WGC				58.00	41.52	0.76
Nuttli	$I_o \geq 6.0$	17	4.90	7.74	11.42	0.64
WGC				11.67	9.43	0.74
Nuttli	$I_o \geq 8.0$	5	6.0	0.83	0.51	0.79
WGC				1.68	0.67	0.85

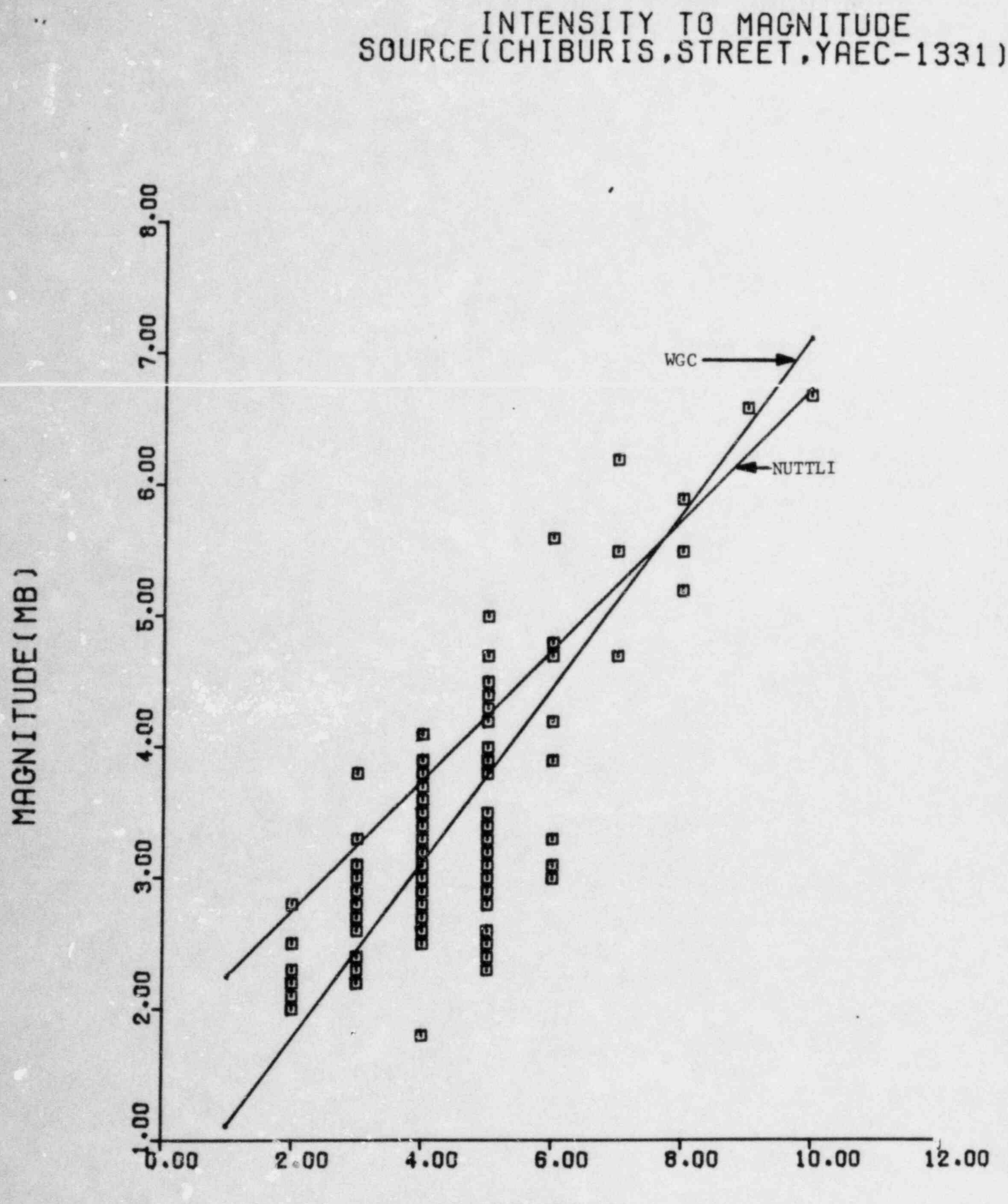


Figure 720.75-1

Question 720.76

As part of an ongoing joint NRC Office of Research and Office of Reactor Regulation effort, the staff is assessing the seismic hazard for all nuclear power plants east of the Rocky Mountains. The Millstone site is included in the first ten sites. Results are discussed and displayed in NUREG/CR-3756, a copy of which was forwarded to you. The hazard results in NUREG/CR-3756 are significantly more conservative than you have assumed in the PSS.

(a) The staff requests your comments on the above report, particularly with respect to differences between LLNL seismic hazard assumptions and those which you have used.

(b) If there is any information in this report which would alter your hazard assumptions provide revised seismic hazard curves.

Response to Question 720.76

The Applicant submits the attached report "Sensitivity of Seismic Hazard Results at Millstone to LLNL Study Assumptions on Attenuation and Seismicity" by Dames and Moore June 15, 1984 in response to Question 720.76 with the following comments:

There are several reasons that the LLNL results are high (conservative) for Millstone on an absolute basis. The rates of activity used by the seismicity experts have apparently been assessed conservatively with respect to both historical macroseismicity and recent microseismicity. The LLNL study includes the effects of earthquakes with magnitudes as low as 3.75, whereas such events are not known to cause structural or mechanical damage to engineered structures or equipment. The use of a lower-bound magnitude of 4.5 or 5.0 would be more realistic and would provide more meaningful results.

Finally, the LLNL results are high at Millstone relative to other sites examined in the study. One attenuation expert has chosen a very conservative relationship and has placed a subjective weight of unity on it. The relationship in question has a term which is dependent on foundation conditions and which increases the estimates of peak ground acceleration and spectral amplitudes for rock sites. Millstone, being a rock site, is treated in a very conservative fashion by this relationship, as compared to alluvium sites. Also, there is evidence that the method used in the LLNL study to convert historical MM intensity data to magnitudes is conservative for New England. Alternative conversion methods, the use of which indicate historical rates of earthquake occurrences consistent with recent instrumental seismicity, give lower rates of occurrences for large shocks. This means that the LLNL method of converting historical intensities to magnitudes gives conservative results for New England and thus affects Millstone.

The extent to which these factors contribute to overestimation of spectral amplitudes in the LLNL study has been estimated in a preliminary way. To remove conservatism introduced by the rates of earthquake activity used in the LLNL study, the spectral amplitudes should be reduced by about 15% for a given probability level (or return period). To remove conservatism associated with unrealistic ground motion estimates, spectral amplitudes should be reduced by 15%. Removing the effect of very small earthquakes means reducing spectral amplitudes by 12% to 25%.

Thus, removing the major conservatisms identified in a preliminary review of the LLNL work means that the spectral amplitudes for a given probability level would decrease by 33% to 43%. A visual representation of the effect is shown on Figure 720.76-1. The peak ground acceleration hazard curve developed by the Applicant is shown as the solid line, and compared to it are several original and revised hazard curves from the LLNL study. The long dashed curves are LLNL median curves, as originally published and as revised by reducing the accelerations at a given probability level to remove the conservatisms discussed above. These reduction factors are shown in Table 720.76-1. The revised median curves lies much closer to the Applicants result. An original and a revised LLNL 15% fractile curve is shown in Figure 720.76-2. The revised curve clearly encompasses the Applicants curve, meaning that the Applicants results and the LLNL revised median results match within one-half standard deviation of the uncertainty in the hazard curves.

With these preliminary revisions, it is clear that the LLNL study results are consistent with those of the Applicant, given the uncertainty in seismic hazard calculations. There are strong scientific justifications for the revisions we describe; use of the original LLNL results to evaluate seismic safety at Millstone is inappropriate, given the preliminary and conservative nature of the LLNL results.

CONCLUSION

In conclusion, on the basis of the responses to Questions 720.74, 720.75 and 720.76 the Applicant finds no justification to revise the seismic hazard curves.

TABLE 720.76-1

SUMMARY OF CHANGES* TO LLNL HAZARD RESULTS
(At 170 cm/sec^2 Ground Acceleration and 10 cm/sec Spectral Velocity)

EFFECT

Parameter	Attenuation	Activity Rates	$m_{b,min}$ 4.5	$m_{b,min}$ 5.0	Combined
Peak	.75/.90	.72/.88	.66/ .86	-	.36/.68
Acceleration	.75/.90	.72/.88	-	.42/ .72	.23/.57
Spectral	.63/.84	.82/.93	.76/ .90	-	.39/.70
Velocity (9hz)	.63/.84	.82/.93	-	.64/ .85	.33/.66
Spectral	.55/.80	.62/.84	1.0 /1.0	-	.34/.67
Velocity (5hz)	.55/.80	.62/.84	-	1.0/1.0	.34/.67

* Effects shown as: probability reduction factor/ground motion reduction factor. Thus the first number is the factor by which the probability should be multiplied (at a given ground motion level) to represent the effect shown at the top of the column. The second number is the factor by which the ground motion level should be multiplied (at a constant probability level) to represent the effect shown at the top of the column.

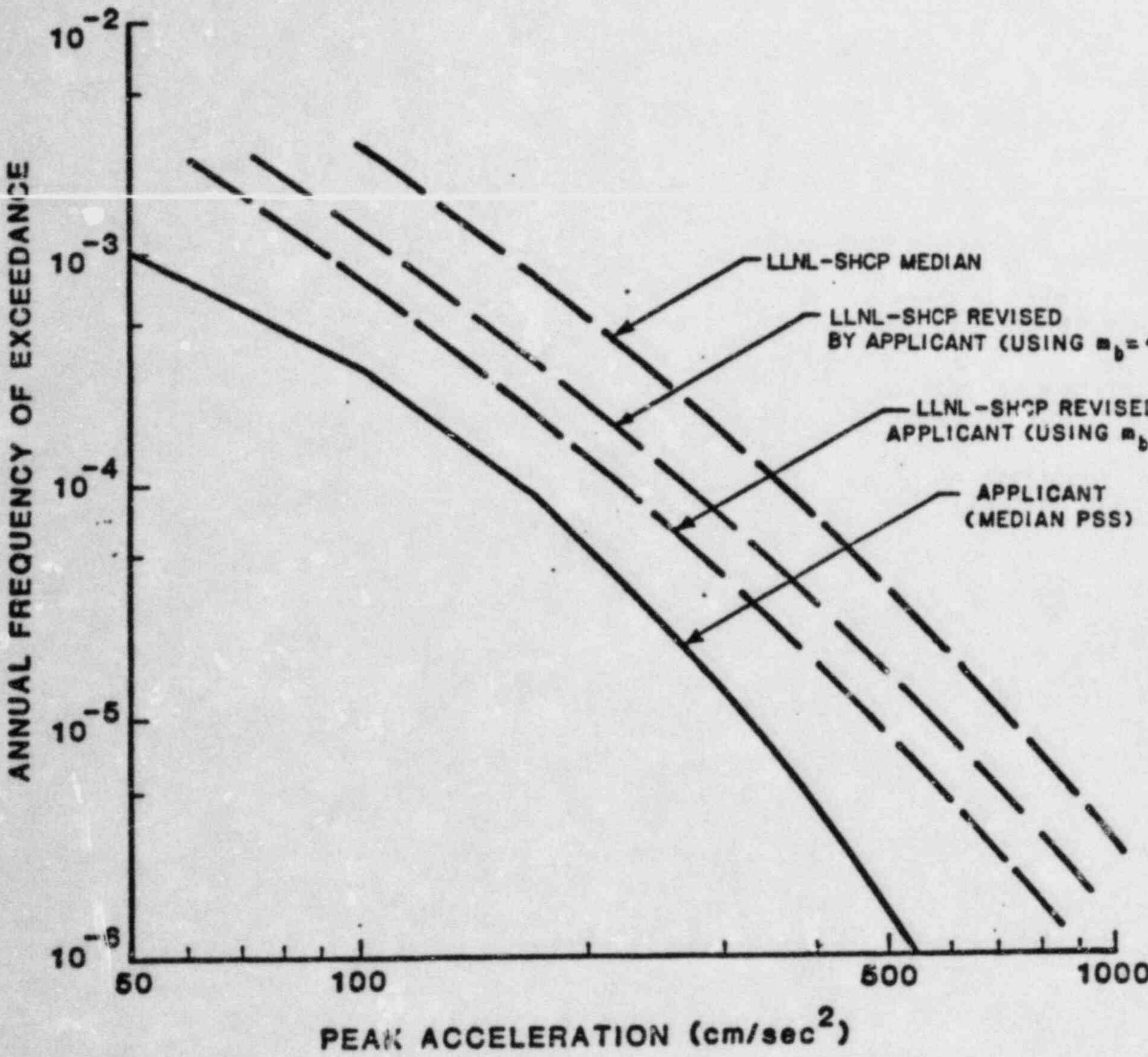


Figure 720.76 -1
Acceleration Hazard Curves

Dames & Moore

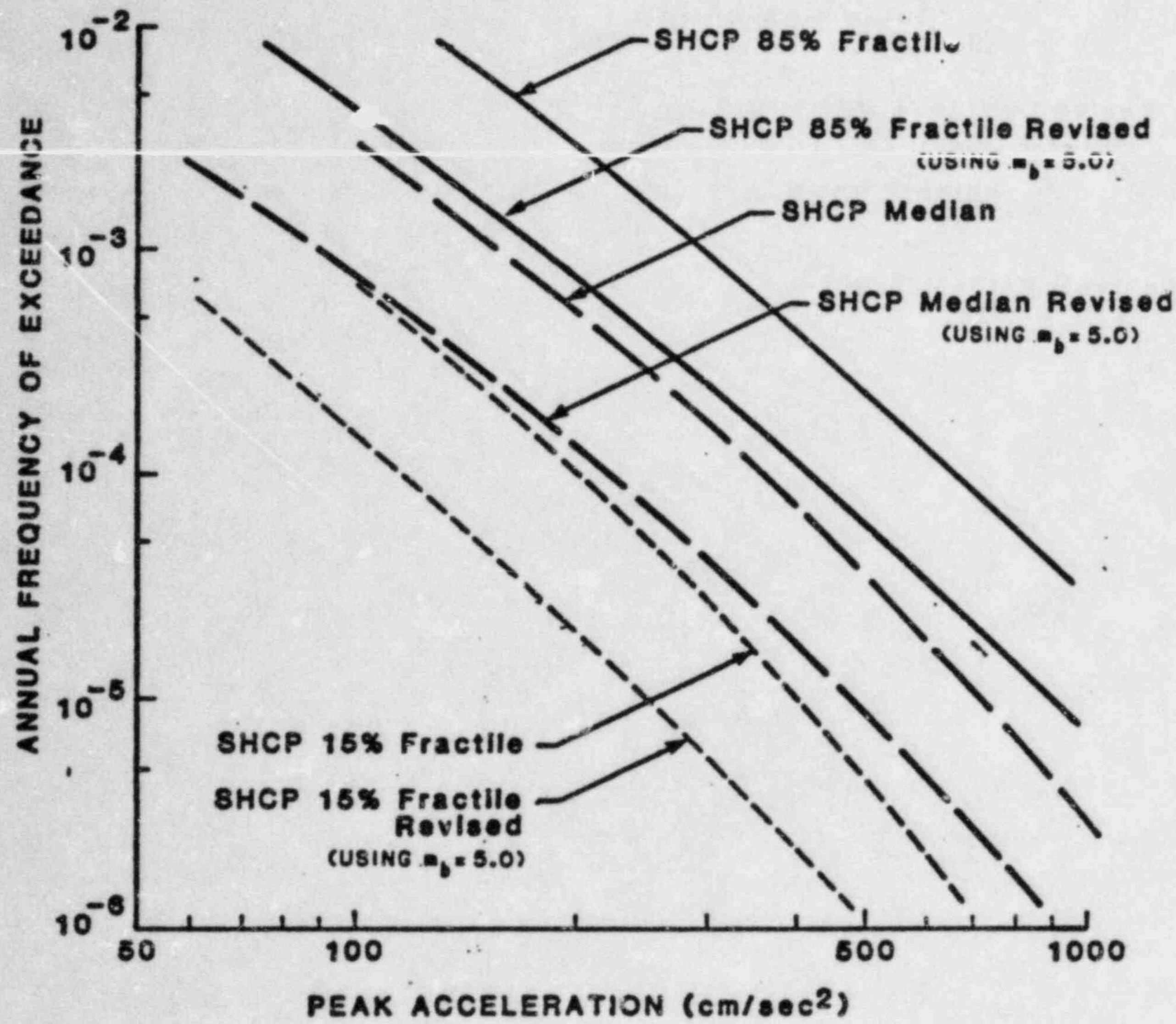


Figure 720.76-2

Acceleration Hazard Curves

Question 720.77

a. You have assumed in your fragility analysis that the majority of seismic risk results from earthquakes that have magnitudes between 5.3 and 6.3. What is the basis for this assumption? Note that Dames & Moore only states that for accelerations around 0.17g, magnitudes around 5.6 dominate the hazard; while Page 2.5-12 of the PSS indicates that over 95% of the total core melt frequency is from accelerations in excess of 0.60g. Considering that the contribution to accelerations of 0.60g and greater is likely to be from large earthquakes, would you alter your value of C_D which is used to calculate effective ductility, and if so, revise Table 2.5.1-1A accordingly. If you would not alter your value of C_D, provide justification considering the above comments.

b. You have also assumed that the median scale factors averaged over four model structure frequencies in Table 4-4 and 4-5 are applicable to Millstone. Considering that typical fundamental frequencies are in excess of 8 Hz, and that the scale factors for 8 Hz are systematically lower than you have assumed, justify the value of C_D you have selected. Would this, in addition to 4a alter your value of C_D and if so, revise Table 2.5.1-1A accordingly.

c. What is your best estimate of C_D for 8.54 Hz model structure frequency due to 6.5-7.5 Richter magnitude earthquakes?

Response:

a. Discussion with Dames & Moore indicated the majority of seismic risk for the Millstone site could be expected from earthquakes in the magnitude 5.3 to 6.3 range. This was reflected in the choice of magnitude 5.8 spectra as the median spectra representative for the site earthquake hazard (from Reference 1). Further analysis by Dames & Moore has indicated that events in this magnitude range dominate the hazard even for ground motions as large as 0.6g and higher. Thus, it is appropriate to use a single spectral shape to represent ground motion for calculation of structural and equipment fragilities. The choice of a magnitude 5.8 spectrum is appropriate, as it represents the stated range. Therefore, the value of C_D used in the seismic fragilities evaluation together with the corresponding variability associated with this factor is considered appropriate for the Millstone site.

b. The C_D correction factor on ductility was based on research available to date (Reference 2) and includes the effects of earthquake magnitudes as well as a contribution from a more accurate stiffness-degrading force-displacement model for concrete shear wall structures than was used in the Riddell-Newmark research (Reference 3).

Based on the limited research to date, C_D is considered to be frequency independent. The factor of safety for ductility, F_μ, on the other hand, is a function of both the frequency and the spectral shape. Based on the median magnitude 5.8 spectra used for the Millstone site, the Riddell-Newmark reduction factor based on the effective ductility of the structures is considered appropriate for structures in the 3 to 12 Hz range. Above approximately 12 Hz, some reduction in F_μ is expected. However, for typical fundamental frequencies in the 8 Hz range, the F_μ and C_D factors used in the fragilities evaluation are considered appropriate, although C_D has some conservative bias.

c. For Richter magnitude 6.5 to 7.5 earthquakes, a best estimate C_D factor of approximately 0.7 for concrete shear wall structures and 1.0 for moment resisting frame structures is indicated based on Reference 2.

REFERENCES

1. "Seismic Hazard and Design Spectra at the Millstone Nuclear Power Plant Unit 3", prepared by Dames & Moore, October 26, 1983.

2. Kennedy, R. P. et al, "Engineering Characterization of Ground Motion, Task 1: Effects of Characteristics of Free-Field Motion on Structural Response", NUREG/CR-3805, May, 1984.

3. Riddell, R., and N. M. Newmark, "Statistical Analysis of the Response of Nonlinear Systems Subjected to Earthquakes", Department of Civil Engineering, Report UILU 79-2016, Urbana, Illinois, August, 1979.

Question 720.78 The staff questions the validity of using a peak bedrock velocity of 28 in/sec/g for the calculation of sliding-induced failure. This value is inconsistent with both that recommended by Newmark and Hall (NUREG/CR-0098, 1978), and that calculated directly for the Millstone site reported in NUREG/CR-3756 (see Question 3). Additionally, the value of 28 in/sec/g based upon only a few Western United States strong motion records may not be appropriate for the Eastern United States where attenuation characteristics are different. Provide a response to these concerns, and if necessary, revise your estimates of fragilities due to sliding-induced failure.

Response:

Determination of fragility parameters for structure sliding-induced failure is, in part, dependent on the ratio of the peak ground velocity to the peak effective ground acceleration. A median velocity to acceleration ratio (v/a ratio) of 28 in/sec/g was selected for use in the Millstone 3 structural fragility evaluation based upon the recommendation of Reference 1 for rock sites. A value of 36 in/sec/g was recommended by Reference 2. However, the 36 in/sec/g v/a ratio is not appropriate for the fragility evaluation since it is a conservative value more appropriate for design.

The median site-specific ground response spectra used in the Millstone 3 fragility evaluation were the median rock spectra for Magnitude 5.8 earthquakes developed in Reference 3. These spectra were selected as being representative for the Millstone 3 site. The median v/a ratio of 28 in/sec/g is actually somewhat conservative when used in conjunction with the median rock spectra from Reference 3. For the median v/a ratio of 28 in/sec/g, the peak ground velocity scaled to the Millstone 3 safe shutdown earthquake (SSE) peak ground acceleration of 0.17g is:

$$\begin{aligned} v &= 0.17\ (28) \\ &= 4.76 \text{ in/sec} \end{aligned}$$

As shown in Figure 720.78-1, the median ten percent damped amplified ground response spectrum is exceeded by this velocity for frequencies less than about 1.4 Hz and greater than about 3.4 Hz. The peak ground velocity for the conservative v/a ratio of 36 in/sec/g, scaled to SSE peak ground acceleration, is:

$$\begin{aligned} v &= 0.17\ (36) \\ &= 6.12 \text{ in/sec} \end{aligned}$$

As shown in Figure 720.78-1, the median ten percent damped ground response spectrum is exceeded by this peak ground velocity through the entire frequency range. This occurrence is not substantiated by real earthquakes.

Inspection of Figure 720.78-1 would imply a median v/a ratio somewhat less than 28 in/sec/g. It can be concluded that the selected median v/a ratio of 28 in/sec/g, while somewhat conservative, is more appropriate for use in the Millstone 3 fragilities evaluation than the conservative design value of 36 in/sec/g.

REFERENCES

1. Newmark, N. M., "A Study of Vertical and Horizontal Earthquake Spectra", WASH 1255, Nathan M. Newmark Consulting Engineering Services, prepared for USAEC, April, 1973.

2. Newmark, N. M., and W. J. Hall, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants", NUREG/CR-0098, May, 1978.

3. Bernreuter, D. L., "Seismic Hazard Analysis, Application of Methodology, Results, and Sensitivity Studies", NUREG/CR-1582, Vol. 4, Lawrence Livermore National Laboratory, October, 1981.

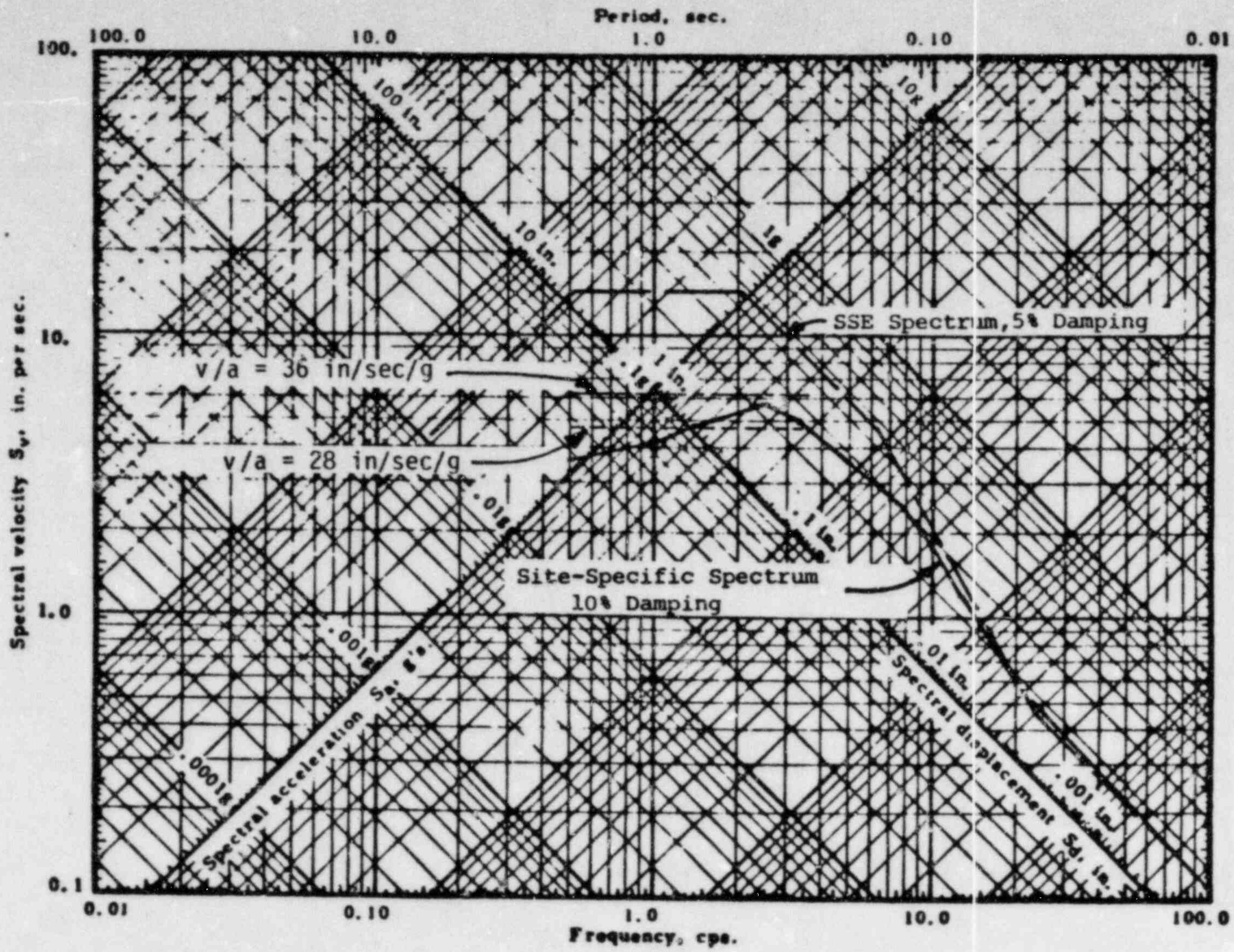


FIGURE 720.78-1. COMPARISON OF PEAK GROUND VELOCITY TO PEAK GROUND ACCELERATION RATIOS WITH MEDIAN SITE-SPECIFIC RESPONSE SPECTRUM

Question 720.79 6). We are unable to verify the fragility of the diesel generator system based on the information you have provided to date. Expand on what methodology was used, how you developed fragility parameter values (and what values were used in your analysis) based on the report:

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

Response:

The fragility of the diesel generator system was based upon the Lube Oil Cooler Anchor Bolts. The original fragility derivation identified the Lube Oil Cooler as the weakest link in the Diesel Generator system. SMA reviewed the five volume Colt Industries report cited and agree that the design margin for the lube oil cooler anchor bolts was indeed the lowest for the individual DG components. The code margin for these bolts was 1.08 based upon an allowable of 0.9 of the specified yield strength. Since the component was relatively rigid (fn ~ 17 Hz) and remained elastic at the point of anchor bolt failure, the failure mode was judged to be brittle and no credit for system ductility was taken. Thus, the factor of safety relative to failure would lie in the ratio of the bolt ultimate stength to the applied load.

The vendor calculations were reviewed so that seismic loads could be separated from normal operating loads. Some judgment was necessary to estimate the ratio of seismic and normal nozzle loading relative to the specified total loading. These loads were not individually reported, only the total load. The ratio was assumed to be 35% seismic and 65% normal. Seismic inertial loads were explicitly defined in the vendor report.

A source of conservatism was identified in the vendors analysis. The four dominant modes of vibration were combined as the absolute sum of the max response value of one mode plus the SRSS of the other three modes. SRSS of all four modes was considered to be a more median-centered estimate. The resulting tensile and shear stresses in the most critical bolt were:

Normal loads (nozzle + dead weight), $\sigma_t = 9911$ $\tau = 4086$ psi

Seismic loads (nozzle + inertia), $\sigma_t = 22,616$ $\tau = 8306$ psi

The bolting material was not identified; a yield strength of 57 ksi was specified. Using ratios of ultimate to yield strength for carbon steel bolting materials with 55-60 ksi specified yield strength, the specified ultimate strength was estimated to be about 90 ksi. Median strength for low alloy steels is about 20% greater than specified strengths, thus median ultimate strength was estimated to be 108 ksi. Applying the above stress values and using the bolting interaction equation for tension and shear and Equation 5-7 in the fragility report, the factor of safety against seismic-induced failure was computed to be 3.62. Uncertainties on each variable were identified and used in Equation 5-9 to compute β_R and β_U. The randomness, β_R, is zero since all variability arises from assumptions on loading, material strength and actual failure point. The uncertainty, β_U, was computed to be 0.31.

The overall equipment response factor was calculated in accordance with Section 5.1.1.2 of the fragility report. The variables considered qualification method, spectral shape, damping, modeling, mode combination and earthquake component combination.

The qualification method factor accounted for some unconservatism identified by comparing the spectral accelerations computed by the vendor to the best estimate spectral accelerations for the diesel generator pedestal. This factor for inertial loading was 0.89. There was no bias in the analysis of attached piping; thus, the factor for deriving piping loads was unity. When both factors are weighted according to their contributions to the critical anchor bolt load, the resulting qualification method factor was computed to be 0.92.

The spectral shape factor was estimated to be unity. Spectra were broadened at the pedestal fundamental frequency but the lube oil cooler frequency was well out of the broadened range; thus, there is no conservatism introduced by broadening. The spectra are also fairly smooth at the equipment fundamental frequency so smoothing introduces little conservatism.

The cooler and attached piping are sufficiently stiff that there is no appreciable difference between median (5%) damped spectra and design damping spectra. The damping factor was therefore set equal to unity.

Structural modeling by the vendor was considered to be median-centered and the modeling factor was set at unity.

The mode combination factor was likewise set at unity since the bias in the vendors mode combination method was already accounted for in the strength factor calculation.

The earthquake component combination was done by SRSS and was considered to be median-centered. The EQCC factor was set at unity.

Randomness and uncertainty were estimated for each of the variables in accordance with the general criteria outlined in Section 5.1.1.2 of the fragility report. The resulting equipment response factor and its randomness and uncertainty were:

$$F_{ER} = 0.92$$

$$\beta_{ER_R} = 0.12$$

$$\beta_{ER_U} = 0.1$$

Note that the β's are small since the component was fairly rigid which results in greatly reduced uncertainty in the response analysis.

The structural response factor was comprised of two variables, spectral shape and soil-structure interaction modeling. The lube oil cooler is mounted on a skid on the diesel generator pedestal which responds as a rigid body in the surrounding soil; thus, variables associated with structural flexure are not germain. The spectral shape factor was derived by comparing spectral acceleration for the design spectrum to that for the site-specific spectrum at the pedestal fundamental frequency of 6.7 Hz. The resulting factor was 1.23. The conservatism in the soil-structure interaction model for the pedestal was estimated to be about 1.3 and the product of the two factors is 1.6. The randomness, β_R, is associated with the variability of the actual site spectrum amplification and is about 0.21. The uncertainty, β_U, comes from both uncertainty in the spectral shape and the soil-structure interaction model and was calculated to be 0.28.

The overall factor of safety relative to the SSE is the product of the capacity factor, equipment response factor and structural response factor and is 5.33. The SSE peak ground acceleration is 0.17g and the calculated median ground acceleration capacity is 5.33 (.17) = 0.91g. The randomness and uncertainty on this capacity, expressed as logarithmic standard deviations are:

$$\beta_R = 0.24$$

$$\beta_U = 0.43$$

Question 720.80 With respect to structure sliding-induced failures of attached piping systems, Section 4.1.1.8 of Appendix 2-I of the PSS states that because piping systems are very ductile, the median relative end displacement necessary to cause piping failure was taken to be 4 inches. Provide additional information to justify the basis of considering failure of attached piping systems to be at relative displacement of 4 inches.

Response:

Some of the Millstone 3 structures may begin to slide on their foundations if sufficient seismic response occurs. The initiation of sliding does not constitute structural or equipment failure. Damage to safety-related equipment as a consequence of structure sliding can occur only if the resulting sliding displacements are of sufficient magnitude to cause impact with adjacent structures or failure of attached piping. An approach recommended by Newmark was used to conservatively determine the peak ground accelerations corresponding to the estimated sliding failure displacements.

Failure of buried piping entering a structure is conservatively assumed to occur when the sliding displacement transverse to the pipe axis causes buckling of the pipe. A total sliding displacement of approximately four inches was found to be necessary to develop pipe buckling. Typically, two inches of clearance exists between the pipe and the wall penetration sleeve. Significant pipe forces cannot be developed until the gap between the pipe and the wall penetration is closed. A sliding displacement of two inches therefore constitutes a lower bound on the displacement necessary to cause pipe buckling.

Calculations were performed to determine the additional pipe relative end displacement causing buckling. These calculations considered the anticipated bounds on safety-related buried pipe size and embedment depth. The pipe buckling moments were based upon the median-centered properties for typical buried pipe material and buckling coefficients presented in Reference 1. Equations to determine pipe stresses resulting from relative displacement between the structure and the adjacent soil are presented in References 2 and 3. These equations were derived from Hetenyi's solutions for beams on elastic foundations. Since the pipes are typically not rigidly anchored to the building wall, the condition at the penetration can be adequately represented as a hinged boundary. From Equation 47 of Reference 2:

σ_b = Maximum bending stress

$$= \pm 0.1612 \frac{kR}{\lambda^2 I} (\delta_R)$$

$$M_{max} = \sigma_b \frac{I}{R}$$

$$= 0.1612 \frac{k}{\delta^2} \delta_R$$

δ_R = Relative end displacement

k, λ = Coefficients dependent on soil and pipe stiffnesses (see Reference 2).

The stiffness of the soil backfill at static strain levels was based upon information contained in Section 2.5 of the FSAR (Reference 4). Backfill stiffness degradation due to strains occurring at failure acceleration levels was accounted for using Figure 5 from Appendix K of Reference 5. The coefficient of subgrade reaction for the calculated backfill stiffness was derived from Equation 10 of Reference 2. The pipe relative end displacement necessary to cause failure was found by equating the maximum applied moment to the pipe buckling moment. A relative displacement of two inches was determined to be a reasonable average value for the anticipated range of safety-related pipe sizes and embedment conditions.

A total structure sliding displacement of four inches was therefore determined to be necessary to develop pipe buckling. This displacement is composed of two inches to close the gap between the pipe and the wall penetration sleeve and two inches of pipe relative end displacement to develop the buckling moment. The effect of uncertainty in the calculated sliding displacement corresponding to piping failure was accounted for in the fragility evauations. The acceleration capacity for sliding-induced piping failure was shown to be relatively insensitive to the necessary displacement. For example, the peak ground acceleration causing an auxiliary building sliding displacement of two inches, which is the lower bound value for piping failure, was calculated to be 1.7g versus the 2.1g acceleration capacity for four inches of displacement. Thus, a 50% reduction in the permissible sliding displacement resulted in only a 20% reduction in the acceleration capacity.

Since Newmark's method to predict sliding displacements introduces conservatism (c.f. Section 4.1.1.7 of Reference 6) and since buckling does not necessarily result in failure, the 4-inch displacement selected as a median value is considered to represent a realistic, although conservative, value for failure of buried pipes.

REFERENCES

1. "Buckling of Thin-Walled Circular Cylinders", National Aeronautics and Space Administration, NASA SP-8007, 1965.

2. Iqbal, M. A. and E. C. Goodling, "Seismic Design of Buried Piping", 2nd ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, New Orleans, Louisiana, December 8-10, 1975.

3. Yeh, G. C. K., "Seismic Analysis of Long Buried Structures", Proceedings, Second International Conference on Structural Mechanics in Reactor Technology, Berlin, Germany, September, 1973, Vol. 4, Part K, K4/8.

4. Millstone Nuclear Power Station - Unit 3 Final Safety Analysis Report, Northeast Utilities.

5. "Structural Design Criteria for Northeast Utilities Service Company, Millstone Unit 3", Stone and Webster Engineering Corporation, January 19, 1983.

6. Wesley, D. A., et al, "Seismic Fragilities of Structures and Components at the Millstone 3 Nuclear Power Plant", prepared for Northeast Utilities by Structural Mechanics Associates, SMA 20601.01-R1-0, March, 1984.

<u>Question 720.81</u> It does not appear that your seismic analysis of the service water pumphouse considers the failure of the adjacent safety-related retaining wall.

(a) Determine the effect on the pumphouse, its piping, or contents if the wall fails.

(b) If failure of the wall can fail the service water system function, estimate the median ground acceleration capacity (B_R and B_U) of this wall.

<u>Response:</u>

The retaining wall extends from the west side of the pumphouse in the north direction. The service water lines emanating from the pumphouse are buried in the soil backfilled on the east side of the retaining wall. Riprap providing slope protection against the sea was placed against the west side of the retaining wall.

Failure of the service water lines may result if the retaining wall fails and sufficient lateral displacement of the soil behind the wall ensues. Calculations were performed to determine seismic fragilities for the retaining wall. Because the controlling applied loading on the wall acts in the outward (seaward) direction, the wall will tend to displace away from the soil. The lateral displacement at failure of the wall will be sufficient to develop the active failure state in the soil. As shown in Figure 28.1 of Reference 1, the active failure wedge lies in back of the vertical plane passing through the heel of the wall footing. Static loading on the wall consists of active pressure from this failure wedge. No net loading on the wall occurs from hydrostatic pressure since this pressure acts on both sides of the wall. Seismic loading consists of dynamic active earth pressure, dynamic water pressure, and inertial forces associated with the mass of the wall itself and mass of the soil above the wall footing. The magnitudes and distributions of the dynamic active earth pressure and dynamic water pressure were based upon Reference 2. The dynamic water pressures acting on both faces of the wall act in the same direction. The inertial force from the soil above the wall footing was included as an additional load since this soil is not part of the active failure wedge. The ability of the retaining wall to resist these applied loads was investigated. Failure modes that were addressed included sliding, overturning, and failure of the structural members composing the wall. A comparison of the applied loads against the available capacities indicated that sliding of the retaining wall at the interface between the bottom of the footing and the top of the fill concrete is the controlling failure mode. Resistance against the initiation of sliding consists of friction at the interface and additional capacity provided by the shear key. Additional resistance against outward displacement of the wall provided by the riprap bearing against the seaward face was conservatively neglected. The median coefficient at friction for rough concrete interfaces was estimated to be 1.0 (see response to Question 720.86). The net weight transmitted at the interface, including weight of the wall and soil above the footing, buoyancy force, and vertical seismic effects, was used.

Sliding of the retaining wall was found to initiate at a median ground acceleration of about 0.4 g. However, initiation of sliding does not constitute failure of the service water lines. The service water lines will fail only when the sliding displacement is sufficient to buckle the pipes. This displacement was limited to the four inches typically required to fail buried piping (see response to Question 720.80). Using Newmark's approach described in Section 4.1.1.7 of Reference 3, the following values were determined for failure of the service water lines due to sliding of the retaining wall:

$$\check{A} = 1.2g$$

$$\beta_R = 0.23$$

$$\beta_U = 0.47$$

As noted in Reference 3, acceleration capacities for sliding-induced failures predicted by Newmark's approach are considered to be conservative.

The above capacity was developed assuming the pumphouse does not slide. Since the pumphouse has essentially the same sliding capacity and the direction of sliding will be the same, it is expected that the retaining wall and the pumphouse will slide approximately the same amount and in phase so that the relative displacement between the wall and the pumphouse will actually be less than the displacement assuming one structure remains fixed. Consequently, including the retaining wall fragility is not expected to affect the overall risk from seismic excitation.

REFERENCES

1. Terzaghi, K. and R. B. Peck, Soil Mechanics In Engineering Practice, 2nd Edition, John Wiley and Sons, Inc., 1967.

2. Seed, H. B. and R. V. Whitman, "Design of Earth Retaining Structures for Dynamic Loads", ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell University, Ithaca, New York, 1970.

3. Wesley, D. A., et al, "Seismic Fragilities of Structures and Components at the Millstone 3 Nuclear Power Plant", prepared for Northeast Utilities by Structural Mechanics Associates, SMA 20601.01-R1-0, March, 1984.

Question 720.82

9). You appear to have used test data from the SAFEGUARD shock test program as part of your nuclear plant equipment fragility data base.

a. Since this program's test motions were of relatively shorter duration, the longest being about two seconds, and of different wave form than those used in nuclear power plant equipment seismic qualification, justify the inclusion of this data in your database.

b. If upon review you believe the SAFEGUARD data is not admissible, revise your data base.

Response:

SAFEGUARDS test data were used to estimate the trip failure mode level for 4160 V switchgear and 480 V motor control centers. The lowest failure mode in both components was defined as initiation of chatter at 120% of the achieved test level. The second lowest failure mode was assumed to be trip. This assumption was based upon the SAFEGUARDS test data interpretations as documented in NUREG/CR-2405. SAFEGUARDS generic results were used to estimate the median trip level. The uncertainty on the trip level was derived assuming that the lowest failure mode (chatter) capcity was a 5% non-exceedance probability ($-1.65\ \beta$) fragility value for trip.

SAFEGUARDS tests were complex waveform biaxial tests of short durations. For components that fail in a ductile structural mode, the SAFEGUARDS tests are deemed to be optimistic since the short duration limits the energy available to inelastically deform structures. However, for components that fail in a brittle mode or that exhibit functional failure while remaining elastic, the SAFEGUARDS data are considered applicable. The SAFEGUARDS test response spectra define the "elastic" response of single-degree-of-freedom systems just as would seismic test response spectra. The critical devices for switchgear and motor control centers are acceleration sensitive and are mounted in cabinets which are elastic at the component functional failure point; thus, the SAFEGUARDS data are considered applicable for estimating generic fragility levels.

Question 720.83 We believe seismically-induced failure of circulating water pipelines may deteriorate the foundation support at the service water pipes and the electric duct bank to the extent that the pipeline and duct bank may fail. Either modify your analysis to consider such an event or justify that this failure mode of the service water pipelines and the electric duct bank is without merit.

Response:

The circulating water pipelines are not safety-related and were not evaluated as part of the Millstone 3 PSS. Since the effective wave propagation velocity at the site is very high, soil strains resulting from seismic excitation will be small and the seismic capacity of the pipelines is expected to be high. Failure of the circulating water pumps will be governed by loss of offsite power which is expected to occur at a relatively low seismic level. Thus, even if failure of the pipelines occurs, only the water volume retained in the pipe would be lost, and this is considered to be insufficient to cause flooding to the extent that loss of foundation support and failure of the duct bank could occur.

Question 720.84 You assumed symmetrical resistance for all sliding analyses. However, for some structures the shear resistance in the two directions may be different and the displacement may be larger than that calculated by assuming symmetrical resistance. We believe this assumption is inappropriate for the intake structure and the emergency generator enclosure. Reanalyze sliding displacement for the intake structure and the emergency generator enclosure, or justify the use of symmetrical resistance.

Response:

Symmetrical sliding resistance was not assumed in the structural fragilities evaluations. Fragilities associated with sliding-induced failures were reported for the horizontal direction having the lower capacity. This approach is consistent with the derivation of the structure shear wall and diaphragm fragilities which were also typically based upon the structural element exhibiting the lowest capacity regardless of earthquake direction. This approach, although possibly somewhat conservative, was adopted since the direction of maximum seismic excitation is not known for the Millstone site.

The Millstone 3 structures are typically subjected to different base shears in the two horizontal directions. The resistances to sliding are also typically different in these two directions. In the structural fragilities evaluation, prediction of the median peak ground acceleration capacity at which structure sliding initiates was based upon a comparison of the sliding resistance in a given horizontal direction with the structure base shear acting in that same direction. Sliding was assumed to occur in the direction having the lower acceleration capacity against the initiation of sliding.

When necessary, the median ground acceleration capacity against the initiation of sliding was determined for both orthogonal horizontal directions. This procedure was used in the evaluation of the control building where separate acceleration capacities were determined in the north-south and east-west directions. For some of the structures, the direction having the lowest capacity against sliding was obvious from an inspection of the available resistances and the applied loading. For example, the pumphouse has shear keys providing additional resistance against sliding in the north or south directions but no shear keys resisting east or west motion. Also, the structure butts against bedrock at its north and east perimeters. This inspection of the available resistance indicated that the lowest resistance to sliding will obviously occur in the west direction. As another example, the emergency generator enclosure will obviously be more likely to slide in the north or south directions since inspection of the seismic responses reported in the FSAR indicates that the north-south accelerations are greater than the east-west accelerations. Also, there is slightly greater resistance against sliding in the east-west direction.

Question 720.85 Your sliding analysis assumed the median sliding coefficient of friction between concrete and rock equals 1.1 but no test data or basis for this value was provided. Either provide a basis for a coefficient of 1.1 or reanalyze your sliding analysis using the 0.7 value recommended in NAVFAC DM-7 when no test data is available.

Response:

The pumphouse base mat bears upon excavated rock or fill concrete poured on intact rock. An estimated 75% of the base mat plan area bears upon the excavated rock surface based upon available excavation drawings. The surface of the fill concrete was raked during construction.

Actual test data for foundation interface coefficients of friction are generally unavailable. Development of median coefficients of friction for the determination of structure sliding resistances was based upon known test results for other interface conditions and engineering judgment. Reference 1 reports the results of testing conducted by Gaston and Kriz to determine the coefficient of friction for formed concrete interfaces. Individual concrete blocks (not masonry) were cast using steel or plastic-coated plywood forms. Based upon testing of these blocks, a coefficient of friction of about 0.8 was found for these relatively smooth concrete surfaces. The raked surface of the fill concrete below the pumphouse base mat is rougher than the concrete block specimins tested by Gaston and Kriz. A median coefficient of friction of 1.0 was therefore estimated for the base mat fill concrete interface. The surface of the excavated rock is very uneven and is therefore expected to have higher friction than the raked surface of the fill concrete. Consequently, a median coefficient of friction of 1.2 was estimated for the base mat/excavated rock interface. A median coefficient of friction of 1.1 was used in determining the sliding resistance of the pumphouse. This value represents an average of the coefficients of friction for the two different foundation interface conditions. It is considered to be somewhat conservative since an estimated 75% of the base mat plan bears on excavated rock which has the higher coefficient.

NAVFAC DM-7 (Reference 2) recommends the use of a coefficient of friction of 0.7 for mass concrete on clean sound rock when specific test data is unavailable. However, this is a conservative value for the design of structures and is not appropriate for use as a median value in fragilities evaluations. Additional conservatism is probably introduced to account for the lack of specific test data. In contrast, the Millstone 3 structural design criteria (Reference 3) specifies a coefficient of friction of 1.0 for concrete foundations poured on blasted rock or raked fill concrete.

Potential uncertainty in the estimated coefficient of friction was accounted for in the fragility evaluation of the pumphouse. Calculations performed indicated that the pumphouse acceleration capacity for sliding-induced failure was not highly sensitive to the foundation interface coefficient of friction. For example, use of the very conservative coefficient of friction of 0.7 would represent a reduction of 36% from the more appropriate median value of 1.1.

However, the resulting acceleration capacity would be 1.1g which is a reduction of only 15% from the median acceleration capacity of 1.3g. Based upon this discussion, it can be concluded that a coefficient of friction of 0.7 is not appropriate for use as a median value and that the median acceleration capacity is not sensitive to the estimation of the median coefficient.

REFERENCES

1. Walker, H. C., et al, "Summary of Basic Information on Precast Concrete Connections", PCI Journal, December, 1969.

2. "Design Manual - Soil Mechanics, Foundations, and Earth Structures", NAVFAC DM-7, Department of the Navy, March, 1971.

3. "Structural Design Criteria for Northeast Utilities Service Company, Millstone Unit 3", Stone and Webster Engineering Company, January 19, 1983.

Question 720.86 Your sliding analyses of the control building and the emergency generator enclosure may be nonconservative. We believe you assumed the shear resistance equals the arithmetic sum of the soil friction resistance and the flexural strength of the shear keys. Provide a basis for such an assumption or reanalyze using a failure plane developed along the bases of the shear keys which cut through the soil.

Response:

The control building is supported on a single base mat typically 4'-0" thick. The base mat bears on one to four feet of compacted structural backfill overlying basal till varying from one to fifteen feet in thickness. Shear keys into the soil were provided beneath the exterior walls. A typical cross-section through the control building foundation is shown in Figure 720.86-1a.

The soil layer bounded by the base mat and the shear keys is essentially constrained or entrapped by the structure. A free-body diagram of the forces acting on this entrapped soil layer is shown in Figure 720.86-1b. In this figure, P_1 and V_1 are the vertical and horizontal forces imparted to the entrapped soil layer by the structure base mat and V_2 is the horizontal force applied by the shear key. P_3 and V_3 are the vertical and horizontal forces acting on the entrapped soil layer at the horizontal plane through the bottom of the shear keys. P_S and V_S are the vertical and horizontal forces associated with the entrapped soil layer itself. The vertical forces P_1 include reductions in the gravity forces to account for the possibility of an upward vertical seismic acceleration. Horizontal forces V_1 are due primarily to seismic response.

Different resistances are available to resist the horizontal shear forces. These resistances are shown in Figure 720.86-1c. The shear resistance at the base mat - entrapped soil layer interface, V_{U1}, is equal to the net vertical force, P_1, factored by the interface coefficient of friction, μ. The ultimate strength capacity of the shear key is denoted by V_{U2}. The shear resistance at the potential horizontal shear plane, V_{U3}, is equal to the net vertical force, P_3, factored by $\tan \phi$, where ϕ is the soil internal angle of friction. Two possible conditions can lead to the initiation of sliding. Condition 1 occurs if V_{U3} is less than the sum of V_{U1}, V_{U2}, and V_S. For this condition, the horizontal shear plane forms in the soil at the elevation of the bottom of the shear keys and the entrapped soil layer slides along with the structure as a rigid body. Condition 2 occurs if V_{U3} is greater than the sum of V_{U1}, V_{U2}, and V_S. If the potential horizontal shear plane has sufficient resistance, sliding will occur at the base mat - entrapped soil layer interface. Sliding at this interface also causes failure of the shear keys since they are constrained to displace along with the base mat.

Both Conditions 1 and 2 were considered in the determination of fragility values for sliding - induced failure of the control building. Based on this evaluation, sliding at the base mat - entrapped soil layer interface with associated shear key failure (Condition 2) was found to govern. Assuming that a horizontal shear plane forms at the bottom of the entrapped soil layer would lead to slightly greater capacity for this failure mode. The fragility values for sliding-induced failure are not nonconservative since the actual condition leading to structure sliding was considered. It should be noted that these fragility values include some conservatism inherent in the use of Newmark's method as described in Section 4.1.1.7 of Reference 1.

The walls of the Emergency Generator Enclosure (EGE) surrounding the diesel generator units are supported by strip footings bearing on soil at EL 9'-0". Soil was backfilled between these walls up to the slab on grade at EL 24'-6". The diesel generator pedestals bear on this backfilled soil, but are separated from the slab on grade by one-inch-thick compressible material. Walls enclosing the fuel oil tank vault were cast integrally with and are supported by a base mat. The bottom of the base mat is located at EL 1'-6". Finish grade of the soil surrounding the EGE is at EL 24'-0".

Inspection of the applied loadings and available resistances for the EGE indicated that sliding will occur in the north-south direction (see response to Question 720.84). In this direction, the structure will slide as a rigid body over the interfaces with the soil at the bottom of the strip footings and the vault base mat. Resistance to the initiation of sliding is composed of three sources:

1. Friction along the base of the foundation generated by the weight of the structure.

2. Shear resistance provided by the soil entrapped by the main structure walls along the horizontal shear plane at the base of the strip footings.

3. Friction on the faces of the exterior east and west walls generated by the lateral earth pressure.

The resistance of Item 2 was based upon the internal soil friction developed at the horizontal shear plane by the weight of the entrapped soil and the diesel generator units and pedestals bearing on this soil. The resistance of Item 1 was based upon friction at the soil-foundation interface generated by the structure weight less any weight accounted for in Item 2. The ability of the structure walls and slabs to resist local loadings resulting from structure sliding was verified in separate calculations. The sliding resistance of the EGE was determined consistent with the approach described in Question 720.86. It should be noted that the resistance against sliding-induced failure of the EGE conservatively neglected the additional contribution from the passive pressure developed by the embedment soil. Also, Newmark's method to predict sliding displacements introduces additional conservatism as discussed in Section 4.1.1.7 of Reference 1.

REFERENCES

1. Wesley, D. A., et al, "Seismic Fragilities of Structures and Components at the Millstone 3 Nuclear Power Plant", prepared for Northeast Utilities by Structural Mechanics Associates, SMA 20601.01-R1-0, March, 1984.

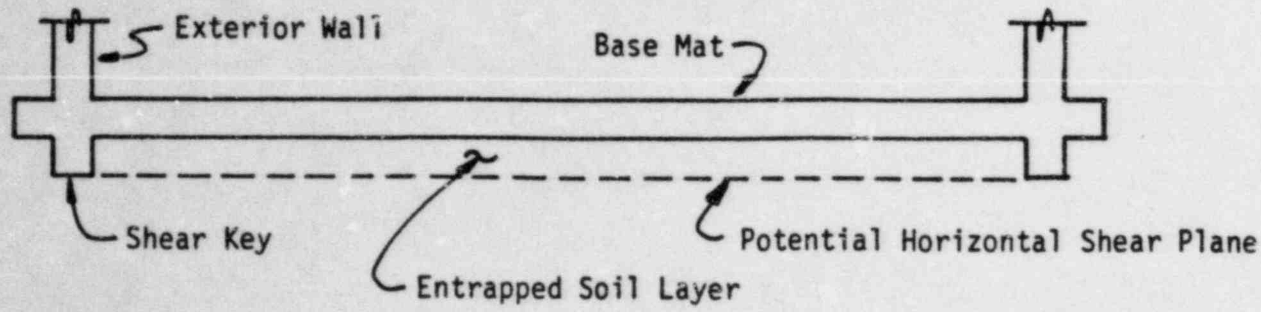


FIGURE 720.86-1a. TYPICAL CROSS-SECTION THROUGH CONTROL BUILDING FOUNDATION

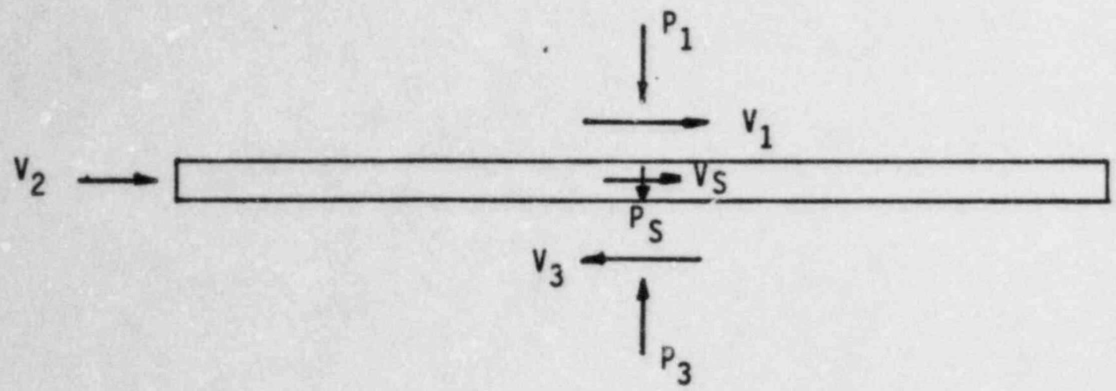


FIGURE 720.86-1b. FREE-BODY DIAGRAM OF ENTRAPPED SOIL LAYER

V_{U1} = Sliding resistance at base mat - soil interface

= μP_i

V_{U2} = Ultimate strength of shear key

V_{U3} = Sliding resistance at horizontal shear plane = $P_3 \tan \phi$

Condition 1: Sliding at horizontal shear plane

$$V_{U3} < V_{U1} + V_{U2} + V_S$$

Condition 2: Sliding at base mat - soil interface and shear key failure

$$V_{U3} > V_{U1} + V_{U2} + V_S$$

FIGURE 720.86-1c. POTENTIAL SLIDING CONDITIONS

Question 720.87 We believe your analysis of the emergency generator enclosure assumed the foundation was anchored in till. We understand the enclosure actually has a foundation partly on till, partly on fill, and partly on till and fill. We believe a new analysis, taking into account as-built conditions, may show your analysis is optimistic. Demonstrate your analysis is not overly optimistic.

Response:

Discussion in Millstone 3 FSAR Section 2.5.4.5.1 indicates that the emergency generator enclosure was founded on approximately 20 feet of structural backfill overlying a 20-foot-thick layer of basal till. The basal till is a fairly stiff soil having a low strain shear modulus of 140,000 psi. The backfill exhibits greater flexibility than the basal till.

With the exception of the structure response parameters, the structural fragilities of the emergency generator enclosure are not influenced by the soil material on which the structure is founded. Structure capacity and response factors and their variabilities were derived from the structure accelerations and in-structure response spectra presented in the Millstone 3 FSAR. These accelerations and spectra were generated as part of the design seismic analysis. The soil material assumed in the original design analysis is not known, but approximate calculations and discussion in the original fragility calculations indicate that the soil may have been represented as the stiffer basal till.

If the original design seismic analysis assumed that the emergency generator enclosure was founded on basal till rather than a combination of backfill and till, reductions in the frequencies from these values originally calculated would be implied. The structure failure modes having the lowest median acceleration capacities result from north-south seismic response. Inspection of the in-structure response spectra shown in FSAR Figures 3.7 B-52, 55, and 58 indicate that modes having frequencies of about 9 Hz and 14 Hz contribute the most to response in this direction. Spectral accelerations at these frequencies for the median-centered, site-specific ground response spectrum can be obtained from Figure 720.87-1 (taken from Figure 3-1 of Reference 1). The spectral accelerations are nearly uniform at the maximum value around the fundamental frequency of approximately 9 Hz. In fact, a reduction in response would be expected if a revised representation of the soil stiffness resulted in a fundamental frequency less than about 8 Hz since spectral accelerations begin to decrease at this value. A slight increase in the seismic response associated with the second mode would be expected. However, significant change in the second mode response would require a substantial reduction in the soil stiffness.

Seismic response of the emergency generator enclosure is a function of the combined stiffness of the soil-structure system. The stiffness of the structure itself is independent of the soil conditions beneath the foundation. The soil stiffness is directly influenced by the representation of the material beneath the structure. However, because the soil-structure system frequency is a function of the square root of the system stiffness and the system stiffness is a combination of the separate soil and structure stiffnesses, representation of the soil median as being composed of basal till rather than till and backfill is not expected to lead to a significant change in the emergency generator enclosure fragilities. Possible uncertainty in the soil-structure representation was accounted for by an increase in the variability associated with this mode of failure.

REFERENCES

1. Wesley, D. A., et al, "Seismic Fragilities of Structures and Components at the Millstone 3 Nuclear Power Plant", prepared for Northeast Utilities by Structural Mechanics Associates, SMA 20601.01-R1-0, March, 1984.

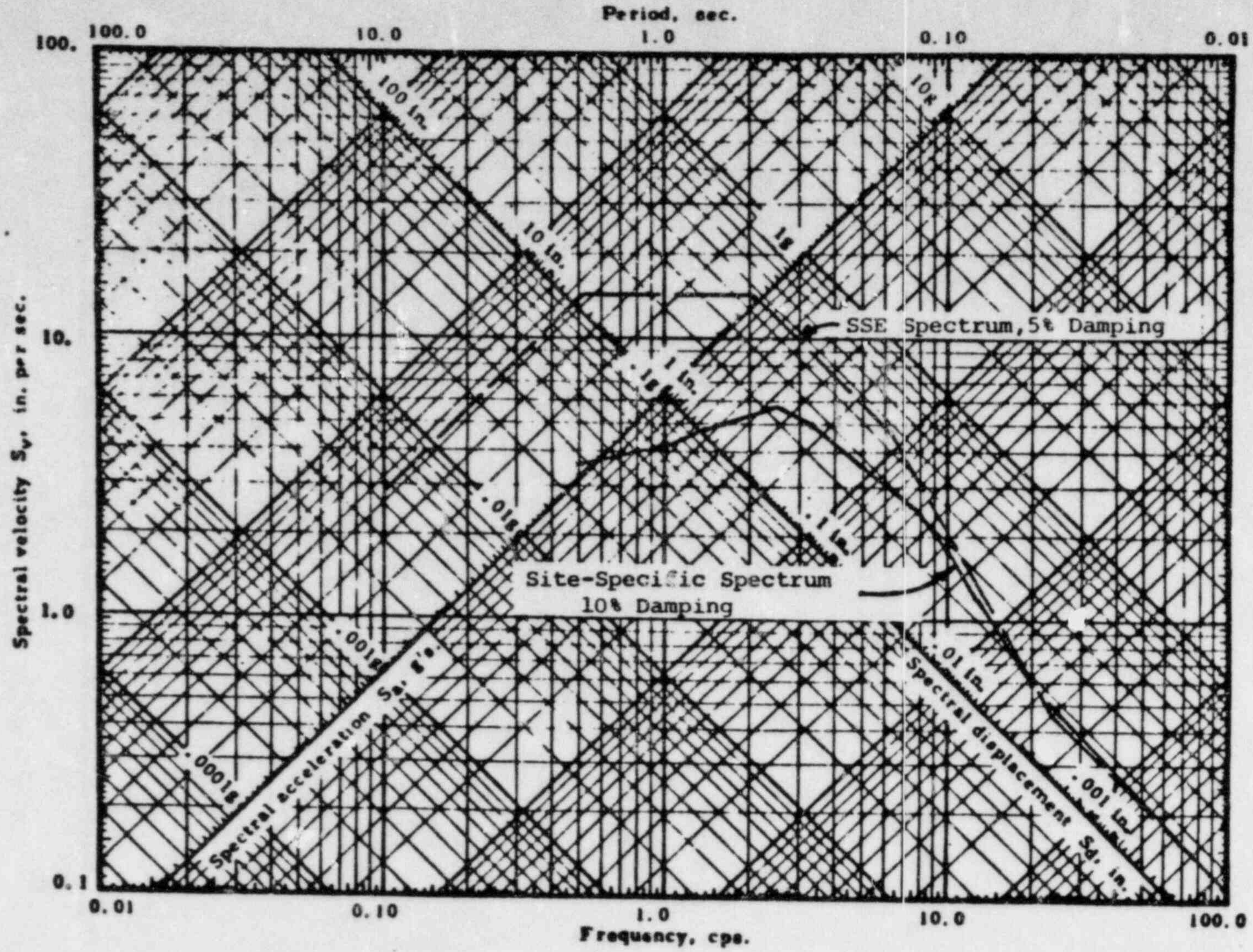


FIGURE 720.87-1. MEDIAN SITE-SPECIFIC GROUND RESPONSE SPECTRUM

Request for Additional Information
Millstone Nuclear Power Station, Unit 3
Docket No.: 50-423

720.88 In the seismic analysis, in Section 2.5.1.3 of the Millstone PSS, the probabilities of the various plant damage states, conditional on a given peak ground acceleration, are calculated. These probabilities are uncertain, and the uncertainty distribution for these plant damage state probabilities are obtained by propagating the uncertainties associated with the basic event probabilities on the fault trees. It is our understanding that the uncertainty distributions for the basic events were assumed to be log-normal in the calculations performed in the PSS. However, the correct distribution for the probability (failure fraction, in the terminology of SMA) is given by eq (A-13) of the SMA report, Appendix 2-I of the PSS, and is not log-normal. As an example, the staff, using eq (A-13) of the SMA report, calculates that the mean probability of plant damage states V3, given peak ground acceleration of .8g is .03, considering only the containment wall failure and neglecting the failure of the steam generator tubes. In contrast, the mean probability of plant damage state V3 given a peak ground acceleration of .8g is .005, according to the PSS, Table 2.5.1 - 21EE. Similar discrepancies will likely affect other plant damage states (e.g., TE or SE) in the neighborhood of .45g.

Justify using a distribution for the failure fraction different than that given by eq (A-13) of the SMA report, or correct the analysis.

Response: Northeast Utilities is currently investigating this and will provide additional information at a later date.

Question 720.89   16). We believe that generic fragility values were used to represent primary system large bore piping in your PSS seismic analysis. Most nuclear power plant seismic design analyses combined SSE and LOCA loads. We believe a less conservative combination of loads was used in the Millstone 3 design process. Therefore, we believe the use of generic fragilities to represent large bore primary system piping may be optimistic. Justify the use of generic fragilities for primary piping or modify your seismic analysis.

Response:

SMA did not develop fragility descriptions for equipment that had a medium ground acceleration capacity of greater than 1.5g derived in the original evaluation. The original fragility derivation's primary coolant loop piping capacity was derived to be 1.59g. Based on our prior experience, we felt this to be a conservative estimate. In fact, studies conducted under the NRC sponsored load combination program indicate that the probability of a directly-induced primary coolant loop pipe break is negligible [1]. In that research, the most probable cause of a primary coolant loop pipe break was found to be from indirect causes such as NSSS component support failures [2]. These failure modes were also included in the original study and the predicted capacities were very high; thus, SMA did not reevaluate them. The work performed by SMA in [2] and LLNL in [1] included Millstone 3 plant specific data and indicated that there is insignificant probability of a primary coolant system pipe break due to direct or indirect causes. This work supports the judgment used in all PRA's to date that earthquakes and LOCA do not occur simultaneously; thus, we do not feel that the primary coolant loop piping fragility derived by the original fragility derivation is optimistic; if anything, it is conservative.

REFERENCES

1. Lo. T., H. H. Woo, G. S. Holman and C. K. Chou, "Failure Probability of PWR Reactor Coolant Loop Piping", Seismic Events Probabilistic Risk Assessments, American Society of Mechanical Engineers, PVP-Vol. 79, 1984.

2. Ravindra, M. K., R. D. Campbell, R. P. Kennedy and H. Banon, "Assessment of Seismic-Induced Pipe Break Probability in PWR Reactor Coolant Loop", Seismic Events Probabilistic Risk Assessment, American Society of Mechanical Engineers, PVP-Vol. 79, 1984.

Question 720.90 If necessary, provide revised estimates of both core melt and public risk due to changes in seismic hazard and/or seismic fragilities resulting from question 720.79-720.89.

Response:

Review of concerns raised by the staff's questions, 720.79 through 720.87 and 720.89, on the seismic capacities of the Millstone 3 structures and equipment components has indicated no significant changes are expected. Question 720.88 is presently being evaluated. No changes in the risk resulting from seismic excitation is anticipated, and no revised estimates of either core melt or public risk are considered necessary.

Question 720.91

Storms of lesser severity than the PMH can have wave run-up which exceeds the height of the door threshold for the service water (SW) pump rooms inside the pumphouse. We understand that due to the design of the circulating water system, water will rise inside of the circulating water pump bays as the water level increases outside.

(a) Estimate the annual frequency that the water level, due to wave action from a storm including run-up, is above the door threshold of the intake structure SW pump rooms.

(b) Estimate the probability that these doors (which provide entry into the SW pump room) will not function as water-tight barriers due for example to door seal leakage or improper door closure.

Response to Question 720.91

The response to this question is still in the course of preparation, it is anticipated that this response will have no adverse affects on core melt or public risk.

Question 720.92

The intake structure has hatches over the service water pumps. Each service water pump room has two service water pumps. We believe that failure of the two pumps in a pump room due to roof leakage is completely coupled.

(a) What is the probability that the service water pump hatch seals leak during a severe storm and disable the pumps?

(b) Estimate the common cause failure probability for loss of service water pumps in both rooms due to roof leakage.

Response to Question 720.92

The response to this question is still in the course of preparation, it is anticipated that this response will have no adverse affects on core melt or public risk.