ENCLOSURE 1

NORTHEAST NUCLEAR ENERGY COMPANY'S RESPONSE TO JACK R. BENJAMIN & ASSOCIATES' REVIEW OF THE REVISED MILLSTONE UNIT NO. 3 PROBABILISTIC SAFETY STUDY SEISMIC FRAGILITY May 4, 1984

DECEMBER, 1984

REVIEW AND COMMENTS ON JBA 105-045, May 4, 1984

DESIGN AND CONSTRUCTION ERRORS

Comment

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.

Response

As discussed in the Fragilities report, insufficient data exists to explicitly determine the effects of possible design and construction errors. It is agreed that additional research is needed. In the Millstone 3 fragilities evaluation, the structures models were reviewed to the extent they could be re-run in order to develop the civil structure load distributions. This provides some limited additional confidence against errors in the seismic modeling phase of the design. Any construction errors are likely to affect only a limited number of components and it is unlikely that these would necessarily be the items with controlling seismic capacities. Therefore, the possibility of design and construction errors significantly affecting the seismic fragilities is considered unlikely.

CORRELATION BETWEEN FAILURE MODES

Comment

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.

Response

Correlation between failure modes was investigated as part of the sensitivity study performed for Millstone 3 (Reference 1). The results indicated that for bounding assumptions of perfect dependence and independence, the four dominant plant damage states do not exhibit significant changes in the frequencies of failure.

Reference 1 Ravindra, M. K., et al, "A Program to Determine the Capability of the Millstone 3 Nuclear Power Plant to Withstand Seismic Excitation Above the Design SSE", prepared for Northeast Utilities, Structural Mechanics Associates Report No. NTS/SMA 20601.01, November, 1984.

MILLSTONE PSS

Comment

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 in 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.

Response

The question of frequency dependency of C_D was addressed in the response to NRC Staff Question 720.92. Contrary to the expectation that high frequency structures and components will experience many more cycles of response than structures with lower frequencies, this does not appear to be the case based on Reference 1.

Reference 1 Kennedy, R. P., et al, "Engineering Characterization of Ground Motion", NUREG/CR-3805, 1984.

RESPONSE SPECTRUM SHAPE

Comment

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 broadband 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.

Response

The implication throughout this section that the Millstone median ground response spectra are not broadband is not correct. These smoothed spectra were developed from a number of earthquake records in the appropriate magnitude range, and in the frequency range of interest for the Millstone structures, in fact exceed the WASH 1255 spectra.

SLIDING ANALYSES

Comment

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.

Response

The basis for the 4-inch displacement was addressed in the response to NRC Staff Question 720.80. In addition, sensitivity studies were conducted (Reference 1) for sliding displacements of 2 and 6 inches. Compared to the base case for the 4-inch displacement, the annual seismic-induced core melt frequencies for the four plant damage states which contribute the majority of the seismic risk show very little change for either the 2 or 6 inch displacements.

1. Ravindra, M. K., et al, "A Program to Determine the Capability of the Millstone 3 Nuclear Power Plant to Withstand Seismic Excitation Above the Design SSE", prepared for Northeast Utilities, Structural Mechanics Associates Report No. NTS/SMA 20601.01, November, 1984.

PUMPHOUSE

Comment

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.

Response

The capacity of the roof slab of the Millstone pumphouse was investigated and found not be controlling.

4160 V - SWITCHGEAR

Comment

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 (Reference 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.

Response

The two failure modes of concern are chatter of protective circuit relays and trip of the breaker assemblies. Both may occur from inertial effects of the earthquake. Relay chatter may or may not cause a circuit breaker trip. This depends upon the circuit design and is addressed by the systems analyst.

Response (continued)

When components are qualified by test, fragility predictions are typically made using either a generic fragility or a subjectively determined factor above the achieved test level.

Generic data for relay chatter shows a low median capacity and extremely large scatter. There was some difficulty achieving the test level without relay chatter so we chose to estimate the median chatter capacity as a modest 20% above the successfully achieved test level. It was further assumed that there was a 1% chance of failure at the achieved test level (-2.33 β's below the median), thus the calculated β was very small (0.08). We agree that, in general, the uncertainty should be larger but, if the assumed median is conservatively biased toward the achieved test level, the assigned uncertainty must be small to reflect the achieved test level. If we were to use a value of 0.2 or greater as suggested by the reviewer, the achieved test level would represent a 15% or greater failure rate which is not realistic.

The breaker trip fragility derived from Corps of Engineers Shock Test data is believed to be valid for acceleration sensitive devices, such as breakers. The validity is only questionable for structural failure modes requiring repeated cycles of inelastic response to produce failure (see, SMA response to NRC Question 720.82, dated July 26, 1984)

The reviewer did not state the basis for his slight disagreement with the earthquake component and building response factors so we can not offer a response. The suggested revision in median capacity of 0.85g instead of 0.88g should have insignificant effects on the resulting core melt frequency.

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 plant. Failure of this wall may lead to failure of the adjacent piping. A fragility analysis should be conducted for this wall.

Response

A fragility analysis was conducted and results are included in SMA's response to NRC Question 720.81, dated July 26, 1984. The median seismic capacity of the retaining wall is approximately 1.2g.

EMERGENCY DIESEL GENERATOR

Comment

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 (Reference 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.

Response

Separate analyses were apparently performed for the Emergency Generator Enclosure (EGE) and diesel pedestal. The details of the diesel pedestal analysis were not available for review at the time the fragilities evaluation was performed. It was assumed the soil-structure interaction analysis performed for the independent pedestal was appropriate for the moderate seismic response in the range of the SSE and lower. However, at higher accelerations, the EGE and the diesel pedestal will tend to move together, in-phase, and with approximately the same response. This occurs because the entrapped soil down to the base of EGE footings (Elevation 9'-0") surrounds the pedestal. Since the mass of the EGE building is significantly greater than that of the diesel and pedestal,

Response (Continued)

the motion of the combind EGE/pedestal system will be controlled by the EGE structure. Since the horizontal elastic response of the EGE base slab (Elevation 24'-6") is about three-quarters of the elastic response in the frequency range of the lube oil cooler, it was judged that a factor of safety of approximately 1.3 could therefore be expected at high acceleration levels where both the EGE and pedestal are constrained to move essentially in-phase.

RPV CORE GEOMETRY

Comment

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 inelstic 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 developing the structural response factors a factor of safety is developed for the difference between the median ground response spectrum and the response spectrum 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.

Response

Figure 3.7B-6 shows both the spectrum resulting from the artificial time history applied to the structural model and the ground spectrum specified for design. The 0.51g value corresponds to the spectrum resulting from the input time history which is the appropriate spectrum to use for comparison of design values to median values.

CONTROL ROD DRIVE MECHANISMS

Comment

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.

Response

The correct values were used in determining the structural response factor (see response to comment on RPV Core Geometry).

ENCLOSURE 2

MILLSTONE NUCLEAR ENERGY COMPANY'S
RESPONSE TO REQUEST
FOR ADDITIONAL INFORMATION
QUESTION 720.92

DECEMBER, 1984

REQUEST FOR ADDITIONAL INFORMATION
MILLSTONE NUCLEAR POWER STATION, UNIT 3

DOCKET NO. 50-423

QUESTION 720.92 - Reliability and Risk Assessment Branch

The staff needs further information to establish the validity of your response to question 720.77. You have stated that further analysis by Dames and Moore has indicated that events in the magnitude range of 5.3 to 6.3 dominate the hazard even for ground motions as large as 0.6g and higher.

(1) How can this be the case considering the peak ground acceleration truncation which you assume? Provide specific Dames and Moore analysis which you have used.

(2) In addition, what is the magnitude range of events which dominate the seismic hazard, specifically for seismic source zones with upper magnitude cutoffs in the range of 6.5 to 7.0, at accelerations of 0.60g to 1.0g?

The staff also needs additional information on your statement that C_D, the correction factor on ductility, is considered to be frequency independent.

(3) Give evidence to support your statement that C_D is considered to be frequency independent. Provide the specific C_D factors associated with the 8.54 HZ model structure frequency for Tables 4-4 and 4-5 included in the Structural Mechanics Associates seismic fragility analysis.

(4) If the values of C_D from question (3) are lower than the C_D you have used (C_D = 1.3), provide justification for your assumptions.

Response to Question 720.92

(1) Currently-accepted methodology for nuclear plant seismic PRAs requires deriving a seismic hazard curve for peak ground acceleration, anchoring a spectral shape to this, and associating probabilities at exceedance of the anchoring peak acceleration with the scaled spectral amplitudes. To apply this methodology in practice, an appropriate spectral shape must be chosen to represent a range of ground motions which might affect the plant. For the Millstone PRA, a body-wave magnitude of 5.8 was chosen as a representative spectral shape.

The probabilities of exceedance of various ground motion values result, in general, from a range of earthquake magnitudes. Exceedances of higher peak ground accelerations tend to be caused by larger magnitudes, but the entire range of magnitudes, from small (which are more frequent) to large (which are rare), can cause exceedances of any acceleration level, in general. For the Millstone hazard study, Dames & Moore truncated the acceleration distribution as a function of magnitude, to reflect concepts of effective ground motion (these are discussed in the Dames & Moore report). The truncations used (Table 4 of the Dames & Moore report) mean, for example, that peak ground accelerations above 0.62 g cannot be caused by magnitudes below 5.8. Magnitudes greater than 5.8 (in particular, in the range 5.8 to 6.3) do contribute significantly to exceedances of 0.6 g and 0.7 g.

The statement made in the response to Question 720.77 (that the majority of seismic risk results from magnitudes of 5.3 to 6.3) refers to peak accelerations around the SSE level. For higher peak accelerations (around 0.6 g), the dominant magnitude (mean magnitude which causes an exceedance of 0.6 g) is still in this range, but is at the upper end of the range. This is shown in Table 720.92a, which shows the mean magnitudes causing exceedances of 0.6 g for all hypotheses on zonation, attenuation, intensity conversion, and maximum magnitude. The only minor exception is for the rift and intersection zones. Here, only the best estimate maximum magnitudes which carry 80 percent subjective weight were used. The averages for each zonation, not counting zero values, and averages over all zonations are also shown. For 0.6 g, the dominant magnitude causing an exceedance is 6.2.

The post-analysis observation that accelerations above 0.6 g (and magnitudes above 5.8) dominate the consequence analysis does not invalidate the choice of the 5.8 spectrum. The effect of larger magnitudes on the spectral shape can be estimated by noting first that only frequencies below about 3 hz will be affected; spectral amplitudes for higher frequencies are governed by the effective peak acceleration and will not increase with the longer period motions associated with larger magnitudes. Below 3 hz frequency, it can reasonably be assumed that the spectral velocity is proportional to peak ground velocity. Thus, any increase in peak velocity relative to peak acceleration would translate into the same increase for spectral velocity. A typical magnitude dependence of peak velocity/peak acceleration is exp (0.7 magnitude). (Nuttli finds a dependence of exp (1.15 magnitude), but states that his predictions of peak velocities are conservative for high magnitudes, implying that his velocity-

magnitude scaling is too high.) Thus, an increase in magnitude from 5.8 to 6.2 would imply about a factor of 1.32 increase in spectral amplitudes below about 3 hz.

In order to investigate the effects of other possible v/a ratios for the Millstone 3 plant, a limited sensitivity study was conducted for the principal structure (service water pumphouse) affected by sliding (Reference 1). In this study, seismic induced core melt annual frequencies were developed for the four plant damage states contributing the majority of risk. In addition to the base case with a median v/a ratio equal to 28 in/sec/g, v/a atios of 17 and 36 in/sec/g were assumed. As shown in Table 720.92e, only minor differences were calculated in the plant damage state annual frequencies for these assumptions. It is therefore concluded that using a 5.8 mb spectral shape is valid in the range of engineering interest for the Millstone PRA.

(2) The magnitudes which dominate exceedances of 0.6 g are shown in Table 720.92a. Average magnitudes for 0.7 g, 0.8 g, and 1.0 g are presented in Tables 720.92b, 720.92c and 720.92d, respectively.

REFERENCES

1. Ravindra, M. K., et. al, "A Program to Determine the Capability of Millstone 3 Nuclear Power Plant to Withstand Seismic Excitation Above the Design SSE", prepared for Northeast Utilities, Structural Mechanics Associates Report No. NTS/SMA 20601.01, November, 1984.

Response

(3) As discussed in the fragilities evaluation report (Reference 1), including the effects of ductility at seismic response levels above yield is necessary to correctly predict the capacities for most structure and equipment modes of failure other than those controlled by brittle failures, elastic buckling, or sliding. In determining the seismic fragilities for Millstone 3, this was accomplished by use of the Riddell-Newmark ductility-modified response spectra approach (Reference 2) which is a sufficiently accurate alternative to conducting expensive and time consuming nonlinear time history analyses for the many structures and equipment items required for the PSS. The Riddell-Newmark ductility modified response spectra method is based on the results of time history analyses of single-degree-of-freedom systems with various damping ratios and resistance functions. It is appropriate for use in conjunction with relatively broadband response spectra such as the median ground response spectra for the Millstone site, or in the constant amplification range for more sharply peaked spectra.

In the Millstone 3 seismic fragilities evaluation, the ability of the structures and equipment to resist seismic response levels above those corresponding to yield through ductile behavior was accounted for by the inelastic energy absorption factor, F_μ. The Riddell-Newmark ductility modified response spectra approach can be used to predict the inelastic energy absorption factor, F_μ, corresponding to some ductility ratio, μ, in the following manner:

$$F_\mu = [(q+1)\mu - q]^r$$

where: $q = 3.0\gamma^{-0.30}$ in the amplified acceleration region.

$= 2.7\gamma^{-0.40}$ in the amplified velocity region.

$$r = 0.48\gamma^{-0.08} \quad \text{in the amplified acceleration region.}$$

$$= 0.66\gamma^{-0.04} \quad \text{in the amplified velocity region.}$$

γ = percent of critical damping.

One drawback of the ductility modified response spectra approach is that it does not reflect the relationship between earthquake magnitude and ductility. It is well known that lower magnitude earthquakes are not as damaging to structures and equipment as higher magnitude earthquakes with the same peak ground accelerations. The reason for this is that the lower magnitude response spectra have lower energy content and shorter durations which develop fewer strong response cycles. Structures and equipment are able to withstand larger deformations (i.e., higher ductility) for a few cycles compared to the larger number of cycles resulting from longer duration events.

The method used in the Millstone fragilities evaluation to account for this effect was based on the use of an effective ductility (μ^*) in conjunction with the Riddell-Newmark ductility modified spectra approach. The following formulation was developed to calculate the effective ductility.

$$\mu^* = 1.0 + C_D\,(\mu - 1.0)$$

where the ductility correction factor, C_D, is a function of the earthquake magnitude.

A limited amount of research is available for use in developing C_D factors. In Reference 3, structures with elastic frequencies of approximately 2, 3, 5 and 8 Hz were subjected to 12 earthquake records scaled to sufficient intensity to produce ductility ratios of approximately 1.9 and 4.3. Included was one artificial record which developed

response spectra which envelope the US NRC Reg. Guide 1.60 spectra. The C_D factors used in the Millstone fragilities evaluation were based on the results from Reference 3. As discussed in the response to Question 720.77, C_D is considered to be frequency-independent based on these limited data.

The factor of safety resulting from ductility effects, F_μ, is dependent on both duration and spectral shape. Figure 720.92-1 is reproduced from Reference 3 and clearly shows the effect of strong motion duration for a ductility ratio of approximately 4.3. However, F_μ is most strongly influenced by the spectral shape and the frequency of the structure. Table 720.92-1 is also reproduced from Reference 3 and shows the F_μ factors for the various earthquake records and structure frequencies for the 4.3 ductility ratio.

A cursory review of Table 720.92-1 would typically indicate lower F_μ factors associated with higher frequency structures compared with lower frequency structures for a given earthquake record. However, use of these results for application with the Riddell-Newmark method in conjunction with the broadband Millstone site response spectra must be done with care. It is inappropriate to include results from Reference 3 for frequencies which lie in a steeply rising or falling portion of a sharply peaked region of the response spectra. As a structure reaches significant levels of inelastic response, there is a decrease in the resonant frequency of the structure. If the elastic frequency of the structure is in a portion of the response spectrum where the frequency shift results in lower response, a relativey higher F_μ will be developed. Conceptually, this is shown in Region A in Figure 720.92-2. Conversely, if the elastic frequency of the structure lies in a region of the response spectrum where the frequency shift results in increased response, a relatively lower F_μ will be predictd as shown in Region C of Figure 720.92-2. In general, this tends to be the case for most of the 8.5 Hz structures analyzed in Reference 3. A review of the data from Reference 3 indicates that many of the F_μ factors shown in Table 720.92-1 do, in fact, lie in

steeply rising or falling regions of the response spectra. Figures 720.92-3 through 720.92-14 have been reproduced from Reference 3 which show this effect for the actual earthquake records used in that investigation.

The Millstone median ground response spectra, however, are relatively broadband and contain significant energy throughout the frequency range from approximately 2.5 Hz to over 12 Hz. Thus, even though a number of structures at Millstone have fundamental elastic frequencies in the 8 Hz range, it is incorrect to use all the F_μ factors directly from the 8.5 Hz results from Reference 3 together with the Riddell-Newmark method and the Millstone median spectra.

Table 720.92-2 shows the original Reference 3 results together with those F_μ factors which result from structure response in relatively flat portions of the respective response spectra and which are considered appropriate for use in the fragilities evaluation. Also shown in Table 720.92-2 are the approximate weighted averages considered appropriate for use in the fragilities evaluation. Because of several anomalies in the Parkfield and Goleta records compared to expected east coast earthquakes appropriate for the Millstone site, these results were not included. However, for earthquakes in the magnitude 6.5 to 7.5 range represented by the first 7 records, an average value of F_μ of approximately 2.2 is indicated. For the remaining earthquakes in the magnitude 4.5 to 6.0 range, the average value is about 2.9 for ductilities of about 4.3.

Using the Riddell-Newmark formulation for F_μ given above together with the 4.27 ductility ratio and 7 percent of critical damping used in Reference 3

$$F_\mu = [(q+1)\mu - q]^r$$

where: $\mu \quad = 4.27$

$q \quad \doteq 3.0\gamma^{-0.30} = 1.67$

$r \quad = 0.48\gamma^{-0.08} = 0.41$

or $F_\mu \approx 2.55$

For earthquakes in the magnitude 4.5 to 6.0 range, an effective ductility can be obtained

$$F_\mu = 2.9 = [(q+1)\mu^\star - q]^r$$

$$= [2.67\,\mu^\star - 1.67]^{0.41}$$

or $\mu^\star = 5.62$

$$\mu^\star = 1.0 + C_D\,(\mu - 1.0)$$

or

$$C_D = \frac{\mu^\star - 1.0}{\mu - 1.0} = \frac{5.62-1.0}{4.27-1.0} = 1.4$$

Similarly, for earthquakes in the magnitude 6.5 to 7.5 range

$$F_\mu = 2.2 = [2.67\,\mu^\star - 1.67]^{0.41}$$

or $\mu^\star = 3.2$

and $C_D = \dfrac{3.2-1.0}{4.27-1.0} = 0.7$

The magnitude 5.3 to 6.3 range expected to contribute to the majority of the risk for the Millstone 3 plant is less than the magnitude 6.5 to 7.5 range and slightly greater than the magnitude 4.5 to 6.0 range. Consequently, the ductility correction factor, C_D, used in the Millstone 3 seismic fragility evaluation was taken to be 1.3.

Thus, based on the limited research available to date, the value of C_D is considered to be independent of frequency and the value of 1.3 used in the fragilities evaluation (Reference 1) is considered to be appropriate for the Millstone site. Any variations in this factor which could reasonably be expected are covered in the variability associated with F_μ in Reference 1.

Although the value of 1.3 is considered to be appropriate for Millstone, the effects of other assumptions were evaluated by means of seismic sensitivity studies for the plant (Reference 4). The approach adopted was to substitute a value of C_D of 1.0 and compare the median and high confidence, low probability of failure fragilities for important affected components as well as the overall plant fragilities for those plant damage states contributing the majority of seismic risk. The components affected are the Emergency Generator Enclosure Building, Control Building Engineered Safety Features Building, Containment Crane Wall, and the Control Rod Drive System. Of most interest is the effects on four plant damage states contributing the majority of seismic risk. Shown in Table 720.92-3 are the median and high confidence, low probability of failure seismic-induced core melt frequencies for the four dominant plant damage states for both the base case (C_D = 1.3) and the case for C_D = 1.0. As is readily observed, only minor differences result in the use of C_D = 1.0.

Response (4)

The value of C_D = 1.3 is not changed from the original evaluation (Reference 1).

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. 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.

3. Kennedy, R. P., et al, "Engineering Characterization of Ground Motion", NUREG/CR-3805, 1984.

4. Ravindra, M. K., et al, "A Program to Determine the Capability of the Millstone 3 Nuclear Power Plant to Withstand Seismic Excitation Above the Design SSE", prepared for Northeast Utilities, Structural Mechanics Associates Report No. NTS/SMA 20601.01, November, 1984.

TABLE 720.92a
MEAN MAGNITUDES FOR EXCEEDANCES OF 0.60 g

		W-W 1981 Attenuation						Campbell Attenuation					
		W-W Conversion			Weston Conversion			W-W Conversion			Weston Conversion		
Zonation	Best Est. $m_{b,max}$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$
Decollement, Version 1	6.3	6.19	6.44	6.11	6.31	6.48	6.25	6.14	6.42	5.96	6.23	6.47	6.28
Decollement, Version 2	7.0	6.45	6.51	6.19	6.50	6.56	6.22	6.47	6.51	6.19	6.51	6.55	6.21
Piedmont	5.8	5.75	6.09	0	5.77	6.11	0	5.75	6.08	0	0	6.11	0
Geologic Provinces	6.0	6.01	6.30	5.75	6.01	6.32	0	5.94	6.26	5.75	5.95	6.24	0
Mesozoic Rift, Version 1	6.0	5.90	0	0	5.92	0	0	5.91	0	0	5.92	0	0
Mesozoic Rift, Version 2	6.0	6.47	0	0	6.57	0	0	6.44	0	0	6.54	0	0
Mesozoic Intersection, Version 1	6.0	5.87	0	0	5.92	0	0	5.81	0	0	5.92	0	0
Mesozoic Intersection, Version 2	6.0	6.40	0	0	6.54	0	0	6.40	0	0	6.54	0	0
Tectonic Provinces	6.3	6.21	6.43	6.13	6.42	6.51	6.25	6.18	6.43	6.07	6.38	6.50	6.28
Northern Appalachian	6.3	6.09	6.43	5.75	6.11	6.46	5.75	6.09	6.42	5.75	6.11	6.45	5.75
U.S. Geological Survey	5.8/ 6.3	6.06	6.26	5.75	6.12	6.33	5.75	6.05	6.23	5.75	6.12	6.29	0

	AI Attenuation						AID Attenuation						
	W-W Conversion			Weston Conversion			W-W Conversion			Weston Conversion			
Zonation	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Average Magnitude
Decollement, Version 1	6.30	6.51	6.18	6.43	6.56	6.32	6.27	6.48	6.18	6.41	6.53	6.28	6.32
Decollement, Version 2	6.57	6.61	6.24	6.60	6.63	6.26	6.49	6.55	6.21	6.54	6.58	6.2[illegible]	6.43
Piedmont	5.77	6.15	0	6.09	6.18	0	5.75	6.11	0	5.86	6.14	0	5.98
Geologic Provinces	6.00	6.36	5.75	6.00	6.36	0	6.03	6.34	5.75	6.03	6.35	0	6.08
Mesozoic Rift, Version 1	5.92	0	0	5.93	0	0	5.91	0	0	5.92	0	0	5.92
Mesozoic Rift, Version 2	6.56	0	0	6.64	0	0	6.52	0	0	6.60	0	0	6.56
Mesozoic Intersection, Version 1	5.85	0	0	5.93	0	0	5.88	0	0	5.92	0	0	5.89
Mesozoic Intersection, Version 2	6.50	0	0	6.62	0	0	6.43	0	0	6.56	0	0	6.50
Tectonic Provinces	6.34	6.53	6.22	6.58	6.60	6.31	6.30	6.49	6.20	6.51	6.56	6.28	6.36
Northern Appalachian	6.12	6.50	5.75	6.13	6.53	0	6.11	6.47	5.75	6.13	6.50	5.75	6.15
U.S. Geological Survey	6.10	6.34	5.75	6.14	6.40	0	6.09	6.31	5.75	6.13	6.37	5.75	6.08
												OVERALL	6.20

TABLE 720.92b
MEAN MAGNITUDES FOR EXCEEDANCES OF 0.70 g

		N-H 1981 Attenuation						Campbell Attenuation						A1 Attenuation						A1D Attenuation						
		N-H Conversion			Weston Conversion			N-H Conversion			Weston Conversion			N-H Conversion			Weston Conversion			N-H Conversion			Weston Conversion			
Zonation	Best Est. $m_{b,max}$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Average Magnitude
Decollement, Version 1	6.3	6.28	6.50	6.28	6.45	6.56	6.32	6.21	6.49	6.31	6.35	6.54	6.34	6.37	6.55	6.34	6.57	6.61	6.35	6.36	6.53	6.31	6.54	6.59	6.33	6.42
Decollement, Version 2	7.0	6.54	6.58	6.26	6.61	6.63	6.30	6.55	6.58	6.26	6.61	6.63	6.30	6.63	6.65	6.29	6.67	6.69	6.32	6.56	6.60	6.28	6.65	6.64	6.31	6.51
Plutons	5.8	0	6.15	0	0	6.18	0	0	6.15	0	0	6.18	0	0	6.19	0	0	6.25	0	0	6.16	0	0	6.20	0	6.18
Geologic Provinces	6.0	6.11	6.37	0	6.19	6.40	0	6.04	6.33	0	6.19	6.36	0	6.09	6.41	0	6.20	6.43	0	6.12	6.40	0	6.19	6.43	0	6.27
Mesozoic Rift, Version 1	6.0	5.95	0	0	0	0	0	5.95	0	0	0	0	0	5.96	0	0	0	0	0	5.95	0	0	0	0	0	5.95
Mesozoic Rift, Version 2	6.0	6.56	0	0	6.65	0	0	6.53	0	0	6.63	0	0	6.62	0	0	6.69	0	0	6.58	0	0	6.67	0	0	6.62
Mesozoic Intersection, Version 1	6.0	5.95	0	0	0	0	0	5.95	5	0	0	0	0	5.96	0	0	0	0	0	5.95	0	0	0	0	0	5.95
Mesozoic Intersection, Version 2	6.0	6.50	0	0	6.63	0	0	6.50	0	0	6.63	0	0	6.57	0	0	6.68	0	0	6.52	0	0	6.64	0	0	6.58
Tectonic Provinces	6.3	6.31	6.50	6.27	6.56	6.58	6.32	6.26	6.49	6.29	6.52	6.57	6.34	6.43	6.57	6.32	6.67	6.65	6.35	6.39	6.56	6.29	6.62	6.62	6.36	6.45
Northern Appalachian	6.3	6.15	6.49	0	6.19	6.54	0	6.15	6.48	0	6.18	6.53	0	6.16	6.54	0	6.19	6.58	0	6.16	6.52	0	6.19	6.56	0	6.35
U.S. Geological Survey	5.8/ 6.3	6.15	6.35	0	6.19	6.45	0	6.15	6.31	0	6.19	6.42	0	6.17	6.47	0	6.20	6.50	0	6.16	6.39	0	6.19	6.48	0	6.29
																									OVERALL	6.37

TABLE 720.92c
MEAN MAGNITUDES FOR EXCEEDANCES OF 0.80 g

Emanation	Best Est. $m_{b,max}$	N-N 1981 Attenuation: N-N Conversion, Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Weston Conversion, Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Campbell Attenuation: N-N Conversion, Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Weston Conversion, Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$
Decollement, Version 1	6.3	6.38	6.55	6.33	6.65	6.62	6.38	6.28	6.54	6.35	6.57	6.61	6.39
Decollement, Version 2	7.0	6.61	6.63	6.33	6.69	6.69	6.38	6.61	6.64	6.32	6.68	6.70	6.38
Plutons	5.8	0	6.20	0	0	6.25	0	0	6.20	0	0	6.25	0
Geologic Provinces	6.0	6.20	6.44	0	6.25	6.50	0	6.20	6.38	0	6.25	6.44	0
Mesozoic Rift, Version 1	6.0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Rift, Version 2	6.0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Intersection, Version 1	6.0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Intersection, Version 2	6.0	0	0	0	0	0	0	0	0	0	0	0	0
Tectonic Provinces	6.3	6.42	6.55	6.32	6.69	6.65	6.38	6.35	6.54	6.34	6.69	6.63	6.39
Northern Appalachian	6.3	6.20	6.56	0	6.25	6.60	0	6.20	6.53	0	6.25	6.59	0
U.S. Geological Survey	5.8/ 6.3	6.20	6.44	0	6.25	6.57	0	6.20	6.40	0	6.25	6.56	0

Emanation	AI Attenuation: N-N Conversion, Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Weston Conversion, Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	AID Attenuation: N-N Conversion, Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Weston Conversion, Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Average Magnitude
Decollement, Version 1	6.47	6.59	6.36	6.72	6.65	6.40	6.46	6.57	6.35	6.69	6.64	6.39	6.50
Decollement, Version 2	6.67	6.69	6.34	6.72	6.73	6.38	6.63	6.64	6.33	6.70	6.70	6.38	6.57
Plutons	0	6.23	0	0	6.36	0	0	6.21	0	0	6.28	0	6.25
Geologic Provinces	6.21	6.46	0	6.25	6.50	0	6.20	6.46	0	6.25	6.52	0	6.36
Mesozoic Rift, Version 1	0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Rift, Version 2	0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Intersection, Version 1	0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Intersection, Version 2	0	0	0	0	0	0	0	0	0	0	0	0	0
Tectonic Provinces	6.53	6.60	6.35	6.76	6.68	6.40	6.50	6.58	6.34	6.72	6.67	6.39	6.52
Northern Appalachian	6.20	6.58	0	6.25	6.62	0	6.20	6.56	0	6.25	6.61	0	6.40
U.S. Geological Survey	6.20	6.48	0	6.25	6.60	0	6.20	6.47	0	6.25	6.59	0	6.37
												OVERALL	6.42

TABLE 720.92d
MEAN MAGNITUDES FOR EXCEEDANCES OF 1.00 g

		N-H 1981 Attenuation						Campbell Attenuation						AI Attenuation						AID Attenuation						
		N-H Conversion			Weston Conversion			N-H Conversion			Weston Conversion			N-H Conversion			Weston Conversion			N-H Conversion			Weston Conversion			
Zonation	Best Est. $m_{b,max}$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Best Est. $m_{b,max}$	$m_{b,max}+.5$	$m_{b,max}-.5$	Average Magnitude
Decollement, Version 1	6.3	6.75	6.63	6.42	6.78	6.69	0	6.78	6.62	6.42	6.80	6.66	0	6.80	6.65	6.42	6.82	6.69	0	6.76	6.64	6.42	6.79	6.69	0	6.66
Decollement, Version 2	7.0	6.72	6.72	6.41	6.76	6.76	0	6.72	6.72	6.41	6.75	6.76	0	6.75	6.75	6.42	6.77	6.78	0	6.72	6.72	6.42	6.76	6.76	0	6.68
Plutons	5.8	0	0	0	0	0	0	0	0	0	0	0	0	0	6.70	0	0	6.71	0	0	6.71	0	0	6.71	0	6.71
Geologic Provinces	6.0	0	6.56	0	0	6.66	0	0	6.69	0	0	6.67	0	0	6.56	0	0	6.68	0	0	6.58	0	0	6.67	0	6.61
Mesozoic Rift, Version 1	6.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Rift, Version 2	6.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Intersection, Version 1	6.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Mesozoic Intersection, Version 2	6.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Tectonic Provinces	6.3	6.73	6.64	6.41	6.78	6.71	0	6.75	6.63	6.42	6.80	6.69	0	6.78	6.66	6.42	6.82	6.72	0	6.75	6.65	6.42	6.79	6.72	0	6.66
Northern Appalachian	6.3	0	6.67	0	0	6.66	0	0	6.62	0	0	6.66	0	0	6.63	0	0	6.67	0	0	6.63	0	0	6.66	0	6.64
U.S. Geological Survey	5.8/ 6.3	0	6.62	0	0	6.66	0	0	6.62	0	0	6.66	0	0	6.64	0	0	6.67	0	0	6.63	0	0	6.66	0	6.65
																									OVERALL	6.66

TABLE 720.92e

SENSITIVITY STUDY FOR v/a RATIOS

Plant Damage State*		v/a Ratio		
		28 in/sec/g (Base Case)	[illegible]7 in/sec/g	36 in/sec/g
V3	Median	3×10^{-9}	-	-
	95%	7×10^{-7}		
AE	Median	5×10^{-8}	4×10^{-8}	5×10^{-8}
	95%	2×10^{-6}	2×10^{-6}	2×10^{-6}
SE	Median	3×10^{-7}	3×10^{-7}	3×10^{-7}
	95%	6×10^{-6}	6×10^{-6}	6×10^{-6}
TE	Median	2×10^{-6}	1×10^{-6}	2×10^{-6}
	95%	2×10^{-5}	2×10^{-5}	2×10^{-5}

* The description of the Plant Damage States is contained in the Millstone 3 PSS. The four Plant Damage States investigated here comprise the majority of risk resulting from seismic events.

TABLE 720.92-1

STATISTICAL EVALUATION OF SCALE FACTOR DATA (Reference 3)

(a) Scale Factors (F_{μ}) for High Ductility Ratio (μ = 4.27)

	Earthquake Record (Comp)	Model Structure Frequency 8.54 Hz	5.34 Hz	3.20 Hz	2.14 Hz	Mean <F>	Std. Dev. σ	C.O.V. σ/<F>
1	Olympia, WA., 1949 (N86E)	1.56	1.54	2.61	3.75	2.37	1.05	0.44
2	Taft, Kern Co., 1952 (S69E)	1.25	1.65	2.05	3.38	2.08	0.92	0.44
3	El Centro Array No. 12 Imperial Valley, 1979, (140)	1.56	2.29	2.10	2.14	2.02	0.32	0.16
4	Artificial (R.G. 1.60)	1.89	1.88	2.84	2.75	2.34	0.53	0.23
5	Pacoima Dam San Fernando, 1971 (S14W)	1.70	1.86	2.67	3.89	2.53	1.00	0.40
6	Hollywood Storage PE Lot, San Fernando, 1971 (N90E)	1.94	2.50	2.60	2.05	2.27	0.33	0.15
7	El Centro Array No. 5, Imperial Valley, 1979, (140)	2.38	2.66	2.33	3.45	2.71	0.52	0.19
8	UCSB Goleta Santa Barbara, 1978 (180)	1.52	2.05	2.05	1.96	1.90	0.25	0.13
9	Gilroy Array No. 2, Coyote Lake, 1979, (050)	1.56	3.85	4.36	3.03	3.20	1.22	0.38
10	Cholame Array No. 2, Parkfield 1966 (N65E)	1.55	1.29	1.48	2.65	1.74	0.61	0.35
11	Gavilan College Hollister, 1974 (S67W)	2.84	2.97	2.71	8.49	4.25	2.83	0.67
12	Melendy Ranch Barn, Bear Valley, 1972 (N29W)	1.89	5.48	5.16	3.36	3.97	1.67	0.42
	Mean, <F>	1.8	2.5	2.75	3.41	Overall:		
	Std. Dev., σ	0.43	1.17	1.03	1.73	<F> = 2.62	σ = 1.28	C.O.V. = 0.49
	C.O.V., σ/<F>	0.24	0.47	0.37	0.51			

TABLE 720.92-2

DATA USED IN DEVELOPING THE DUCTILITY FACTORS FOR

THE MILLSTONE 3 FRAGILITIES (Ref. 3)

Scale Factors (F_H) for High Ductility Ratio ($\mu = 4.27$)

Earthquake Record (Comp)	Model Structure Frequency				Weighted Average
	8.54 Hz	5.34 Hz	3.20 Hz	2.14 Hz	
1 Olympia, WA., 1949 (N86E)	1.56	1.54*	2.61	3.75	2.0
2 Taft, Kern Co., 1952 (S69E)	1.25	1.65*	2.05*	3.38*	2.0
3 El Centro Array No. 12 Imperial Valley, 1979, (140)	1.56	2.29*	2.10	2.14*	2.1
4 Artificial (R.G. 1.60)	1.89*	1.88*	2.84*	2.75*	2.3
5 Pacoima Dam San Fernando, 1971 (S14W)	1.70*	1.86*	2.67*	3.89	2.2
6 Hollywood Storage PE Lot, San Fernando, 1971 (N90E)	1.94*	2.50*	2.60	2.05	2.5
7 El Centro Array No. 5, Imperial Valley, 1979, (140)	2.38*	2.66*	2.33	3.45	2.5
8 UCSB Goleta Santa Barbara, 1978 (180)	1.52	2.05	2.05	1.96	—
9 Gilroy Array No. 2, Coyote Lake, 1979, (050)	1.56*	3.85*	4.36	3.03*	3.0
10 Cholame Array No. 2, Parkfield 1966 (N65E)	1.55	1.29	1.48	2.65	—
11 Gavilan College Hollister, 1974 (S67W)	2.84*	2.97*	2.71	8.49	2.8
12 Melendy Ranch Barn, Bear Valley, 1972 (N29W)	1.89*	5.48*	5.16	3.36	> 3

* Values which result from relatively flat portions of the response spectra

TABLE 720.92-3

SENSITIVITY STUDY FOR $C_D = 1.0$

Plant Damage State*		Base Case $C_D = 1.3$	Increased Magnitude Case, $C_D = 1.0$
V3	Median	3×10^{-9}	4×10^{-9}
	95%	7×10^{-7}	8×10^{-7}
AE	Median	5×10^{-8}	5×10^{-8}
	95%	2×10^{-6}	2×10^{-6}
SE	Median	3×10^{-7}	4×10^{-7}
	95%	6×10^{-6}	7×10^{-6}
TE	Median	2×10^{-6}	2×10^{-6}
	95%	2×10^{-5}	2×10^{-5}

* The description of the Plant Damage States is contained in the Millstone 3 PSS. The four Plant Damage States investigated here comprise the majority of risk resulting from seismic events.

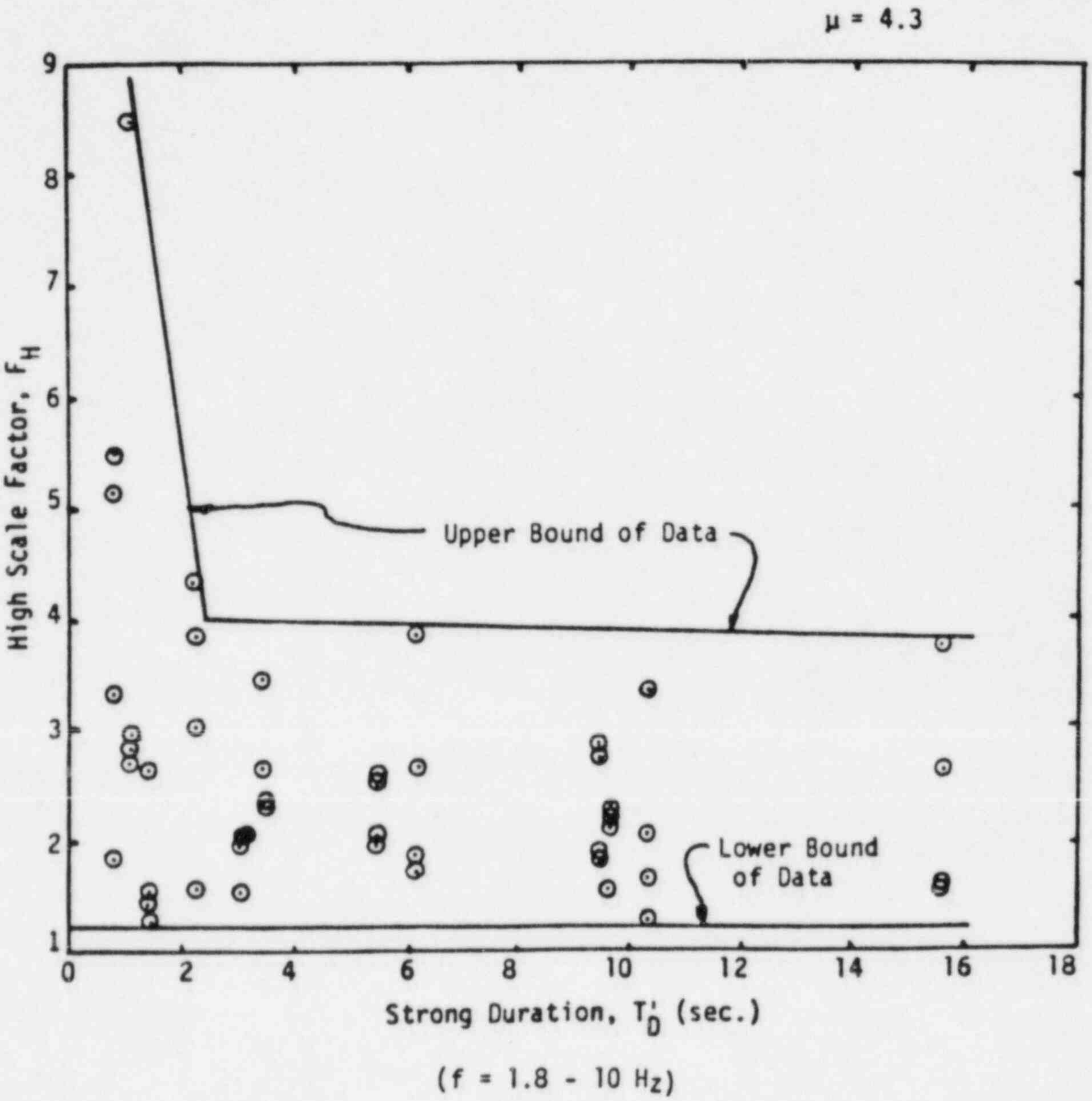

FIGURE 720.92-1. SCALE FACTOR, F_H VERSUS DURATION

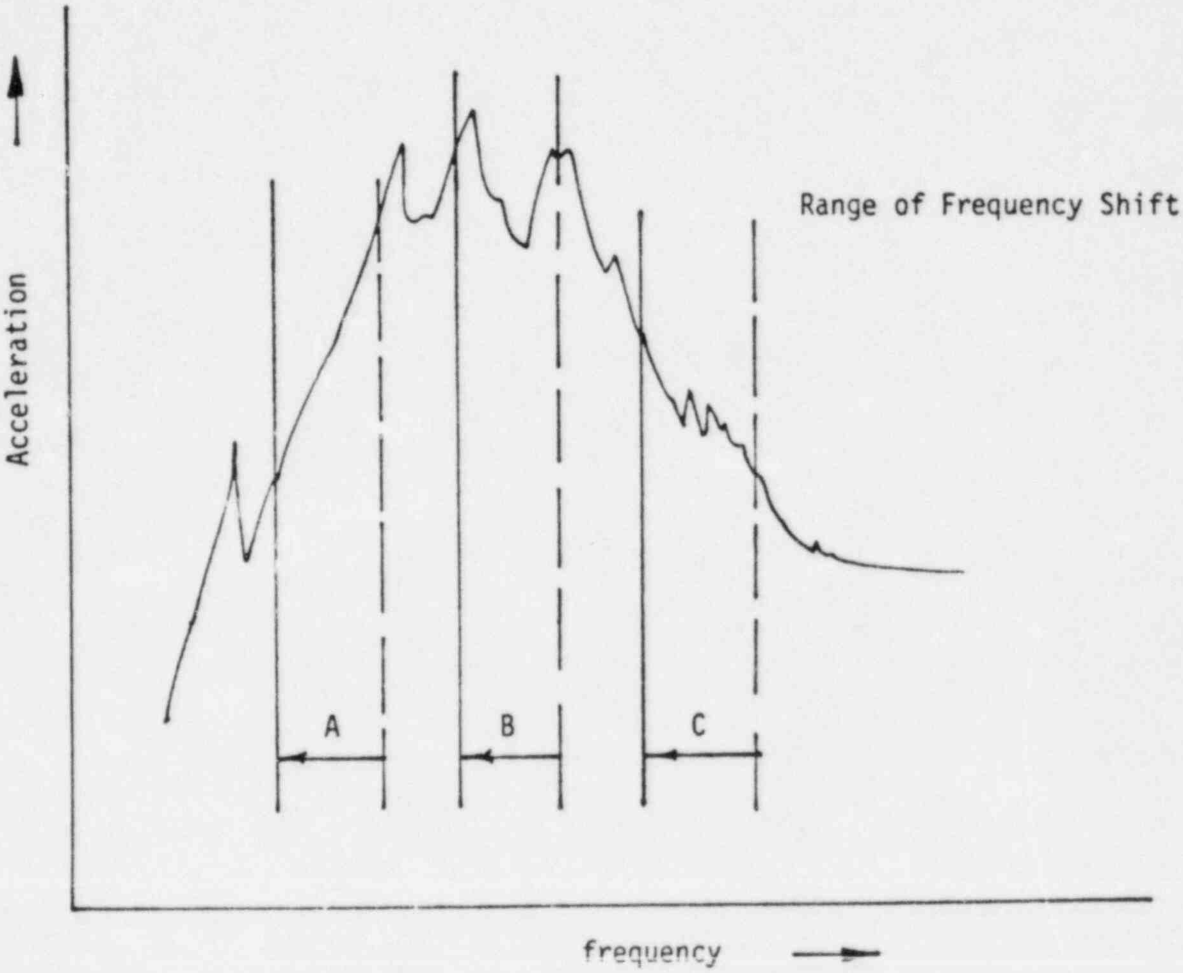

FIGURE 720.92-2. ELASTIC RESPONSE SPECTRUM SHOWING EFFECT OF FREQUENCY SHIFT ON RESPONSE

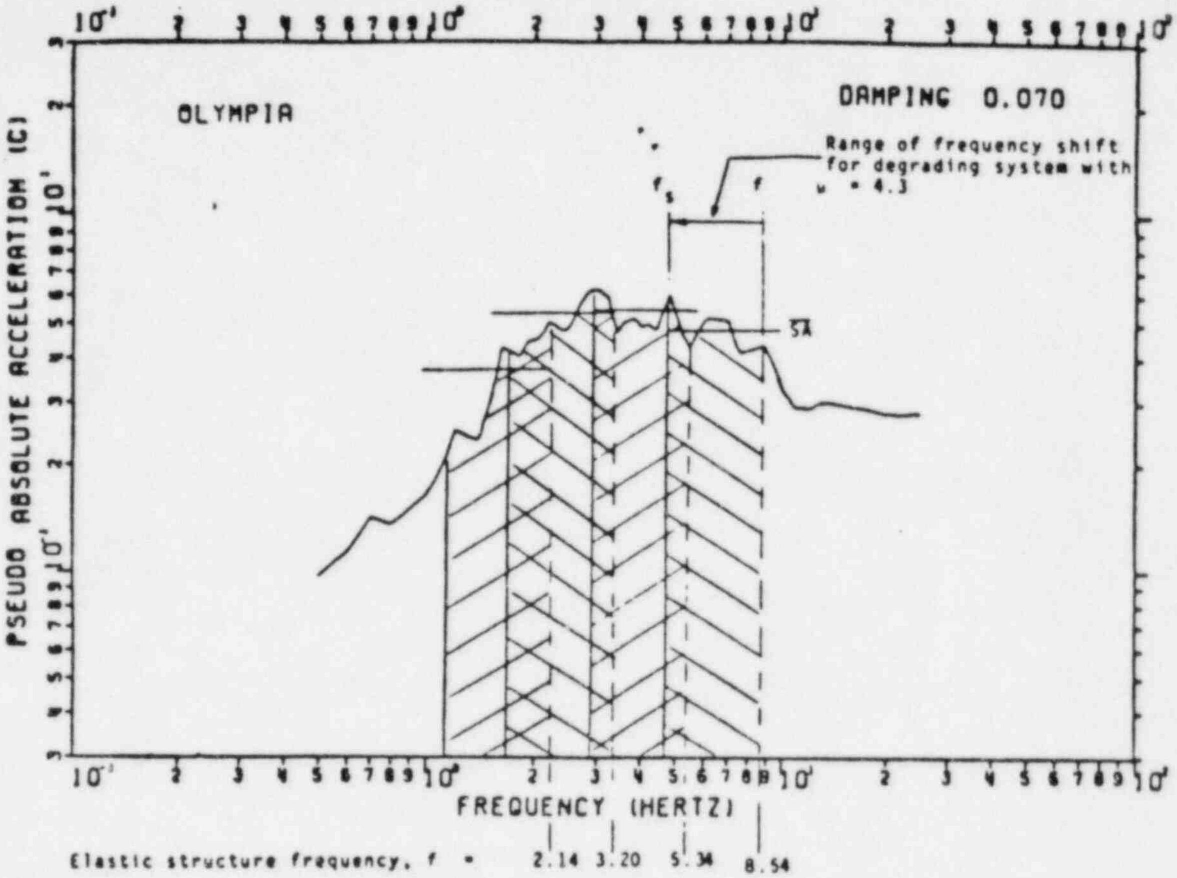

FIGURE 720.92-3

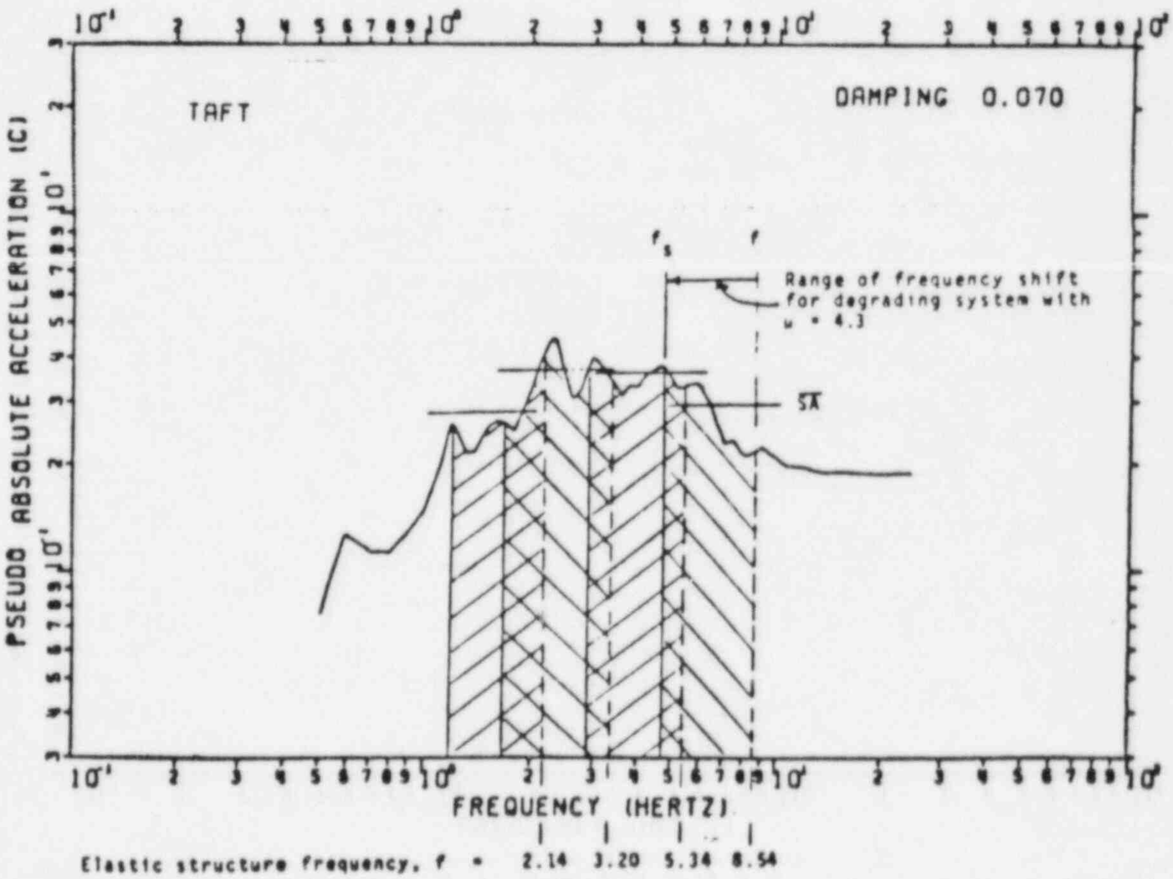

FIGURE 720.92-4

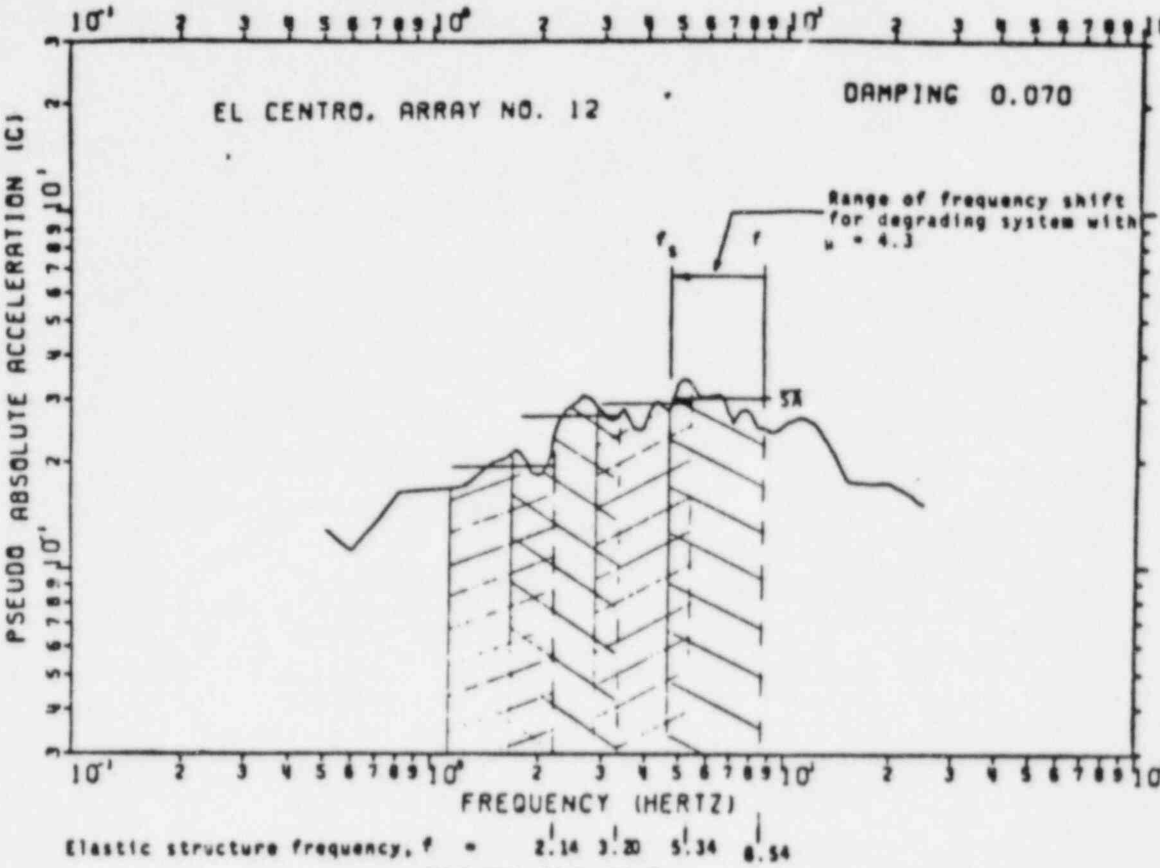

FIGURE 720.92-5

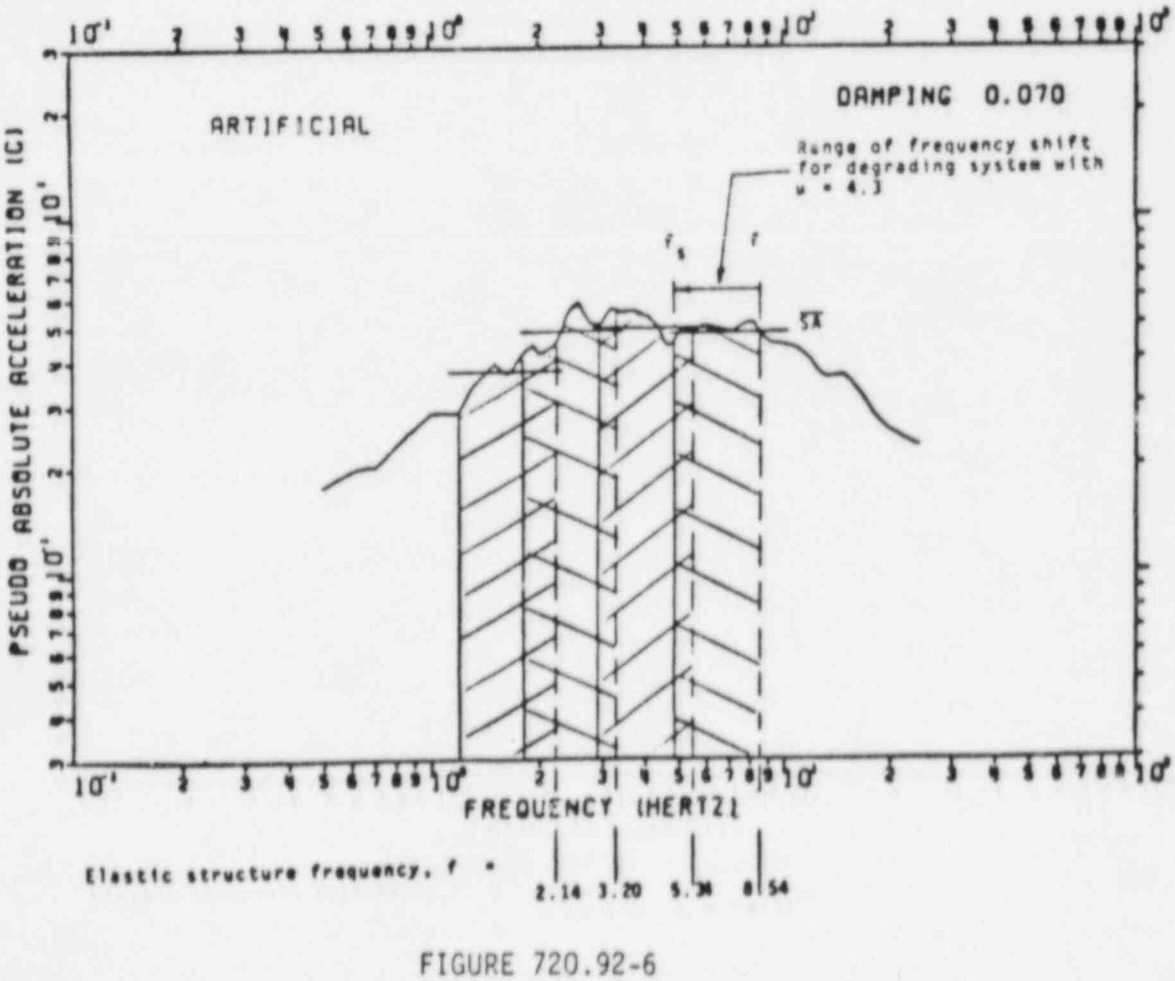

FIGURE 720.92-6

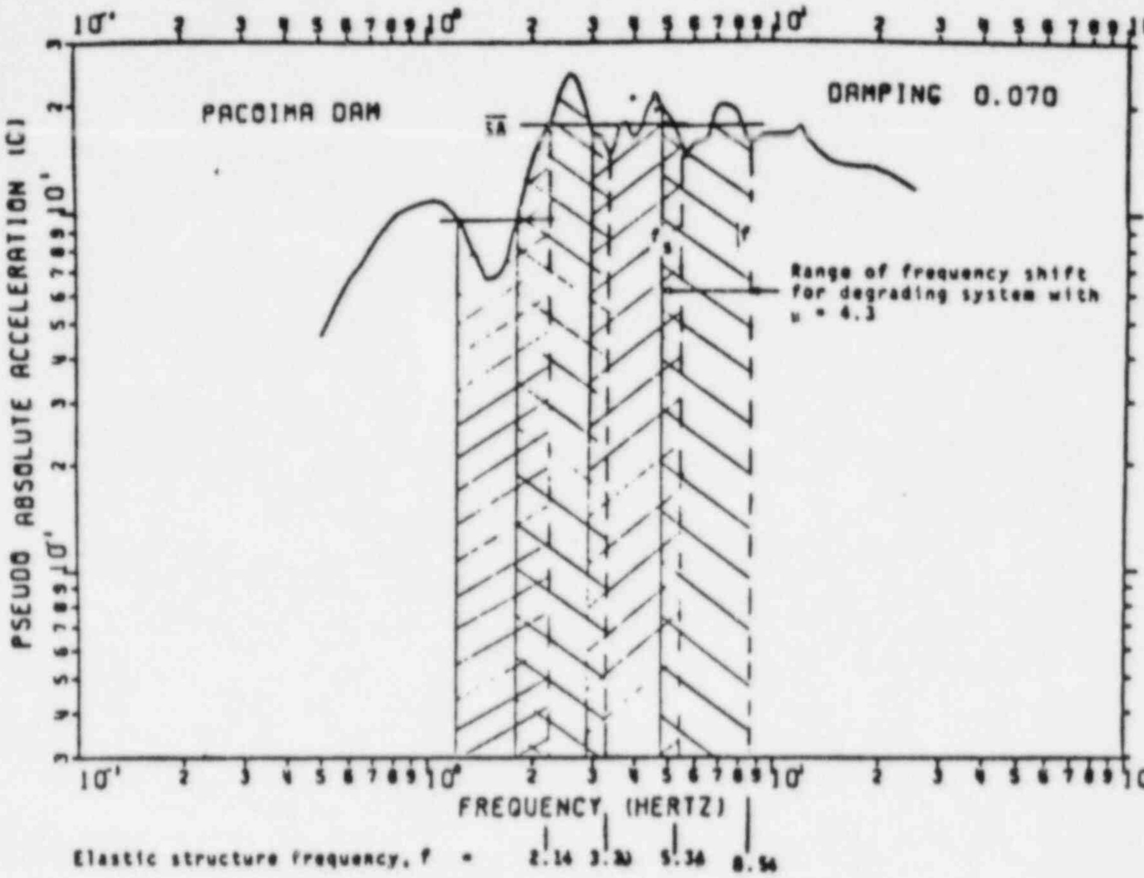

FIGURE 720.92-7

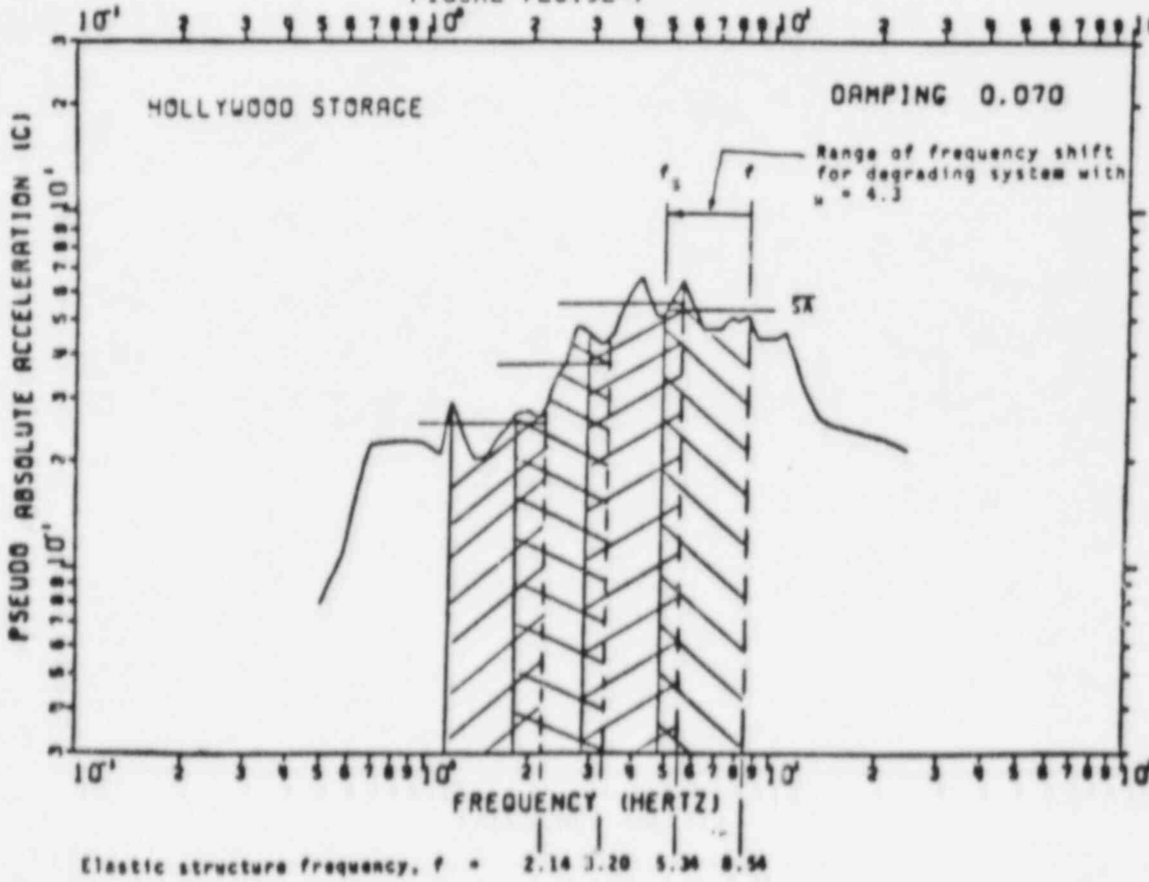

FIGURE 720.92-8

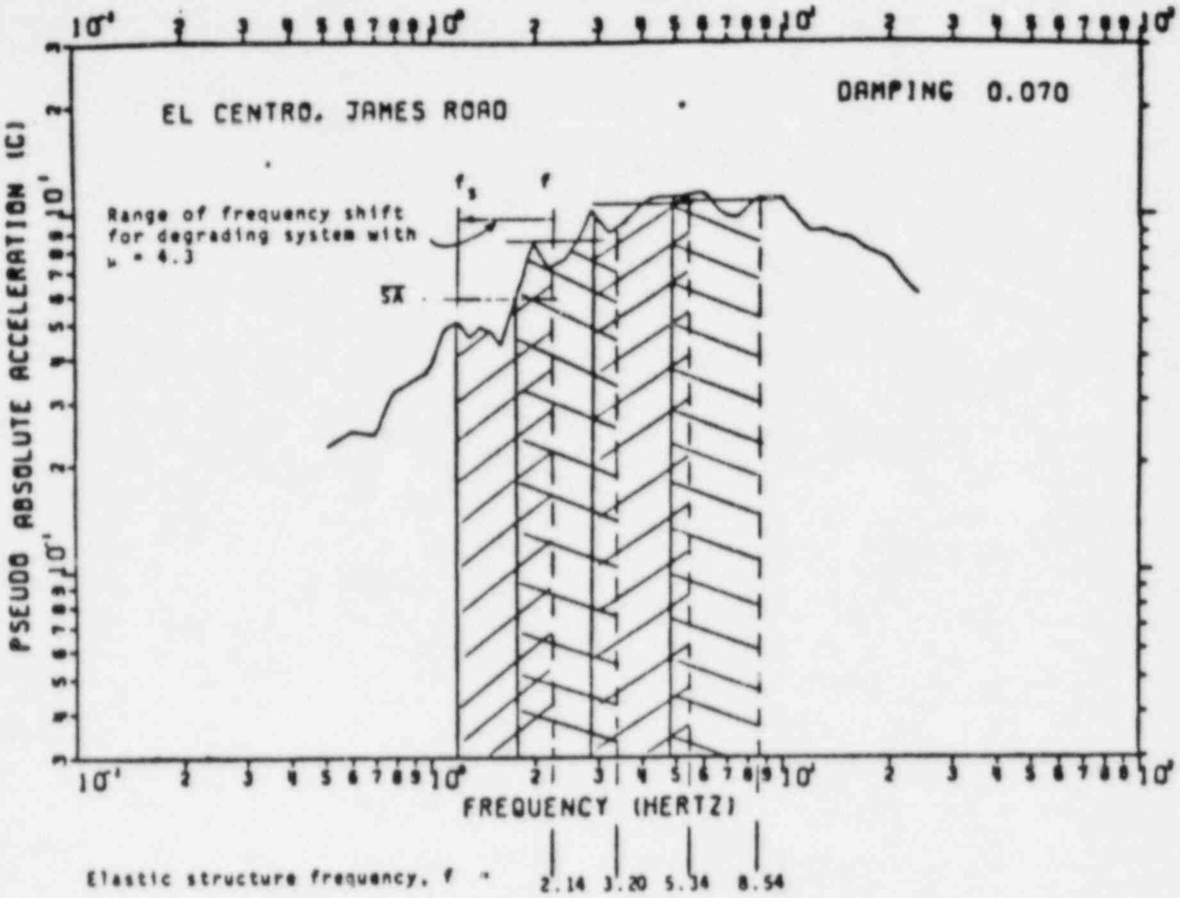

FIGURE 720.92-9

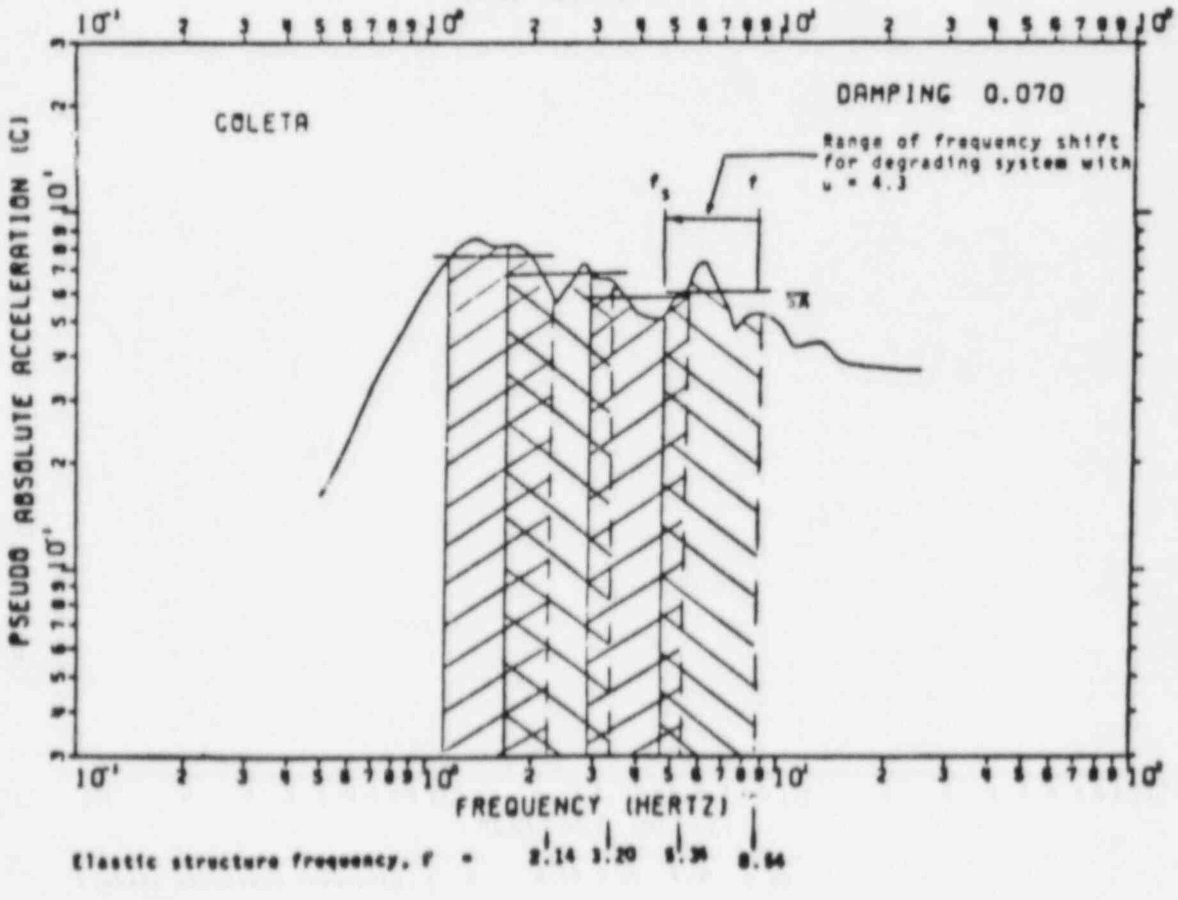

FIGURE 720.92-10

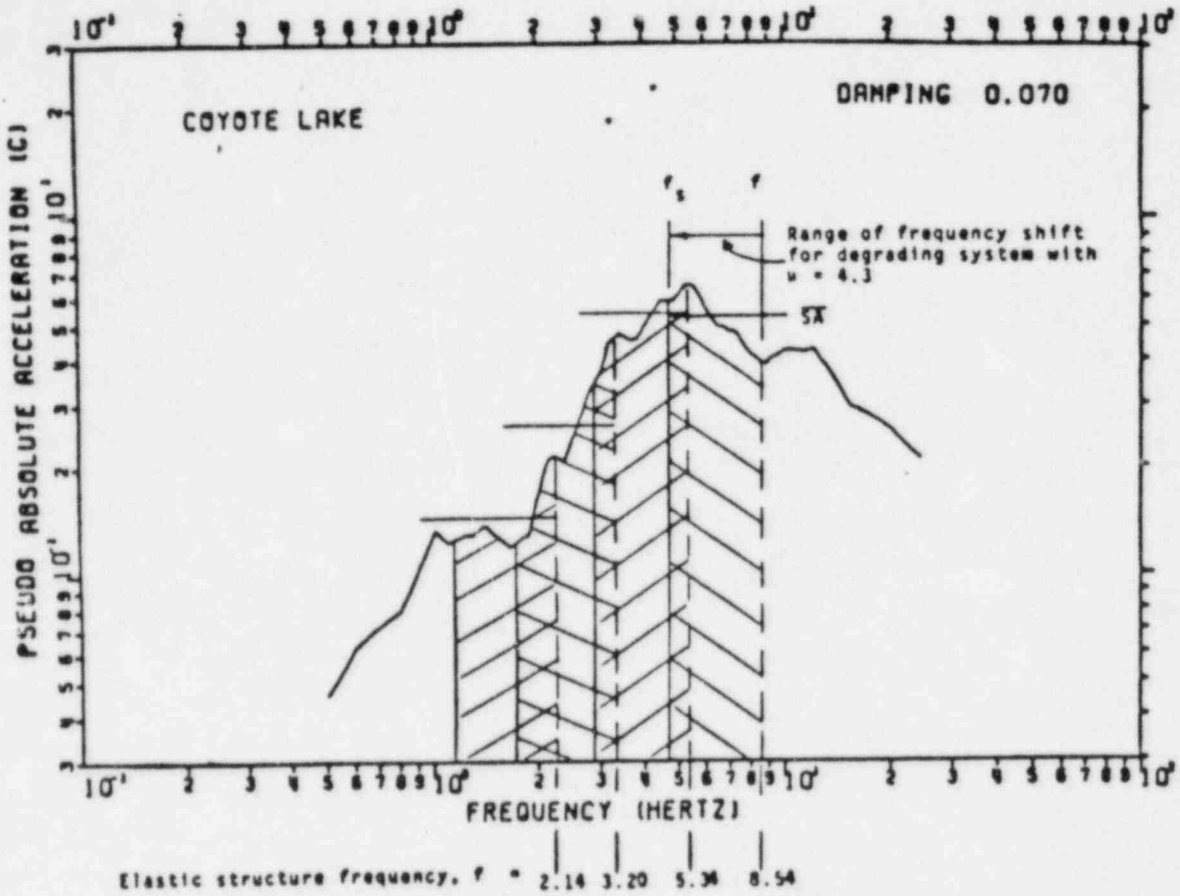

FIGURE 720.92-11

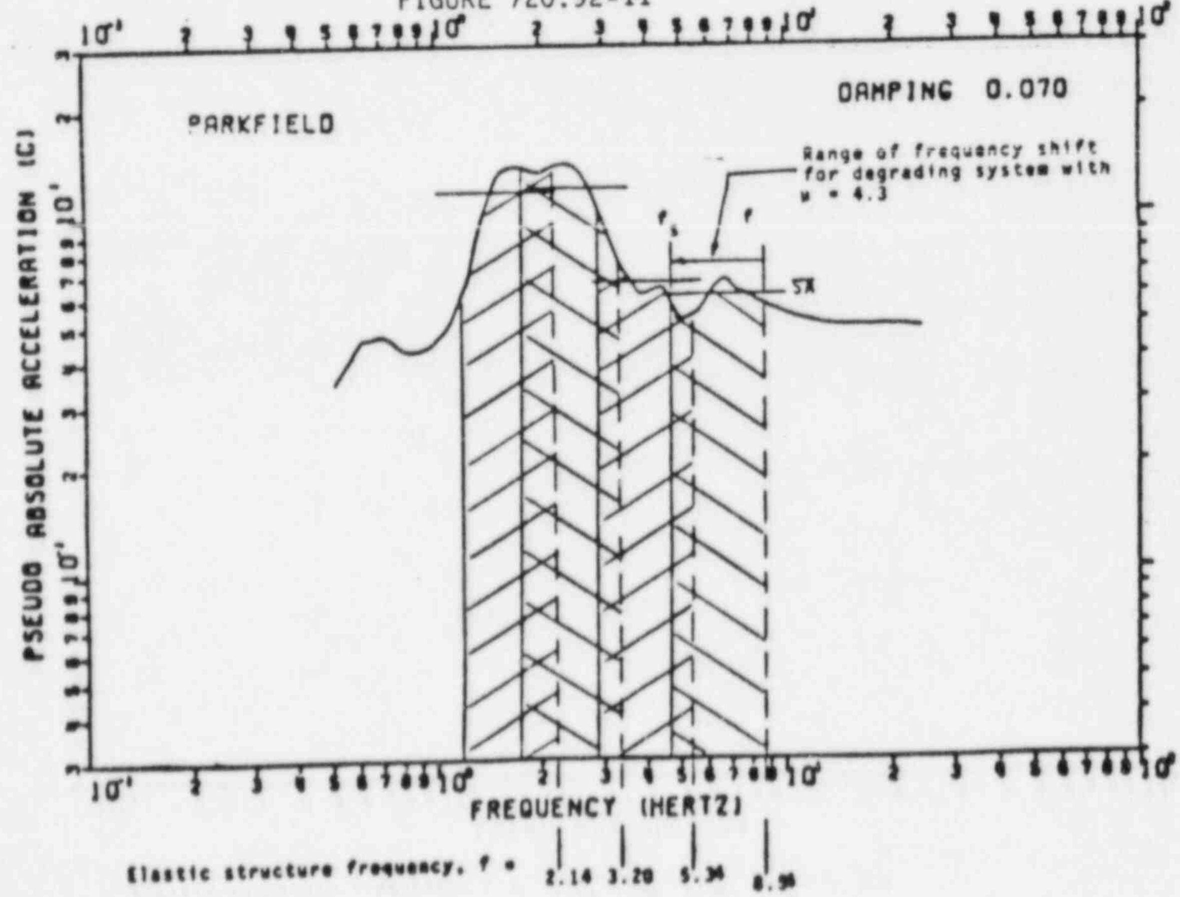

FIGURE 720.92-12

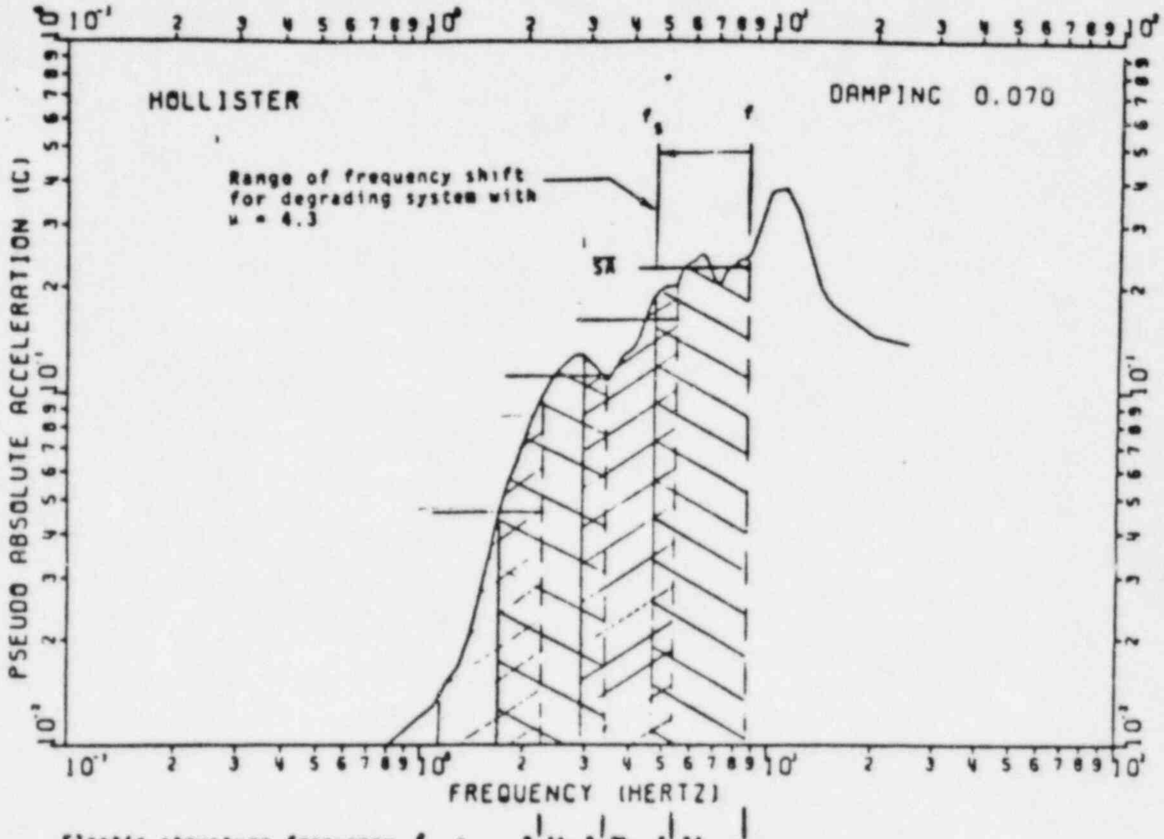

FIGURE 720.92-13

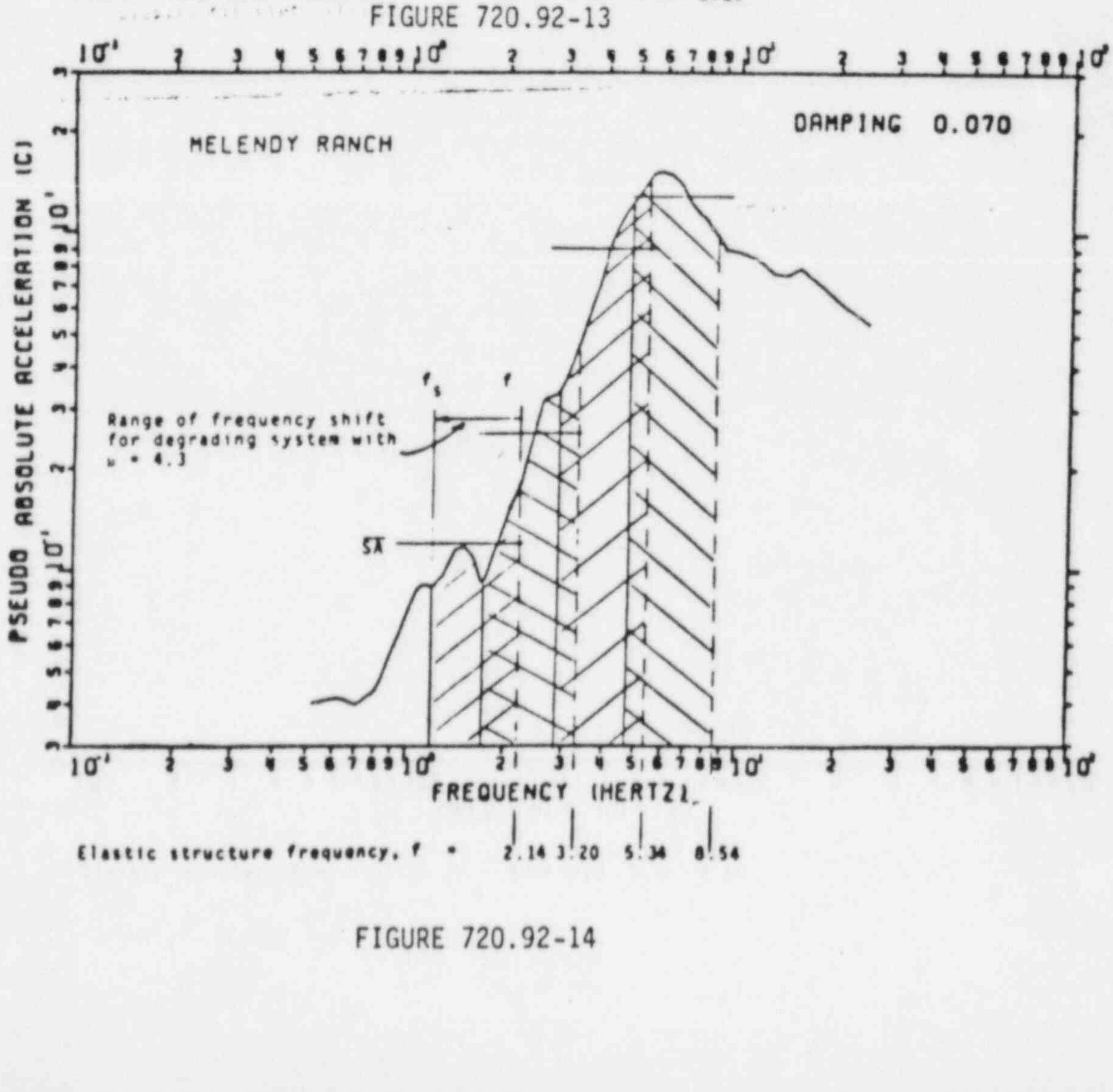

FIGURE 720.92-14