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TOKYO, JAPAN

June 23, 2011

Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Attention: Mr. Jeffery A. Ciocco

Docket No. 52-021
MHI Ref: UAP-HF-11186

Subject: MHI's Supplemental Responses to US-APWR DCD RAI No. 659-5133 (SRP 03.07.01)

References:
1) "Request for Additional Information No. 659-5133 Revision 2, SRP Section: 03.07.01 – Other Seismic Category I Structures," dated 11/15/2010.
2) "Response to Request for Additional Information No. 659-5133 Revision 2, SRP Section: 03.07.01 – Other Seismic Category I Structures," (MUAP-HF-10047), dated 12/29/2010.

With this letter, Mitsubishi Heavy Industries, Ltd. ("MHI") transmits to the U.S. Nuclear Regulatory Commission ("NRC") a document entitled "Supplemental Responses to Request for Additional Information No. 659-5133, Revision 2."

Enclosed is the supplemental response to clarify one of the previous responses submitted in Reference 2 to Question 03.07.01-17 contained within Reference 1. This transmittal completes the response to this RAI.

Please contact Dr. C. Keith Paulson, Senior Technical Manager, Mitsubishi Nuclear Energy Systems, Inc. if the NRC has questions concerning any aspect of this submittal. His contact information is provided below.

Sincerely,

Y. Ogata

Yoshiki Ogata,
General Manager- APWR Promoting Department
Mitsubishi Heavy Industries, LTD.

Enclosure:

1. Supplemental Responses to Request for Additional Information No. 659-5133, Revision 2

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NRO

CC: J. A. Ciocco
C. K. Paulson

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Enclosure 1

UAP-HF-11186
Docket No. 52-021

Supplemental Responses to Request for Additional Information No. 659-5133, Revision 2

June, 2011

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

06/23/2011

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 659-5133 REVISION 2**
SRP SECTION:	**03.07.01 – Seismic Design Parameters**
APPLICATION SECTION:	**3.7.1**
DATE OF RAI ISSUE:	**11/15/10**

QUESTION NO. RAI 03.07.01-17:

This request for additional information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR Part 50, Appendix A, General Design Criteria 2; 10 CFR Part 50 Appendix S; and 10 CFR Part 100; as well as the guidance in NUREG-0800, 'Standard Review Plan for the Review of Safety Analysis for Nuclear Power Plants,' Chapter 3.7.1, "Seismic Design Parameters."

Section 3.2 of MHI's Topical Report, MUAP-10006 (R0), addresses site conditions and states that Tables 3-3A through 3-3H present the input material properties of the subgrade. However, the basis for the values in these tables is not discussed. In order to conduct a technical evaluation of the supporting media used for the seismic analysis, the staff requests that the applicant provide the following information:

1. The origin of the information in the tables
2. A description of how and to what extent the information does or does not relate to the data shown in Tables 5.2-3 through 5.2-11 of MUAP-10001 (R1)
3. A statement as to whether the properties shown are low-strain or strain-iterated properties
4. If the properties are strain-iterated properties, a description of which time histories (i.e. horizontal H1, horizontal H2, vertical, or some combination) were used to generate the properties
5. A description of how the compressional wave speeds and damping used in the vertical seismic analysis were developed.

ANSWER:

1. MHI Technical Report MUAP-10001 Section 5.2 is the origin of the information contained in Technical Report MUAP-10006 Tables 3-3A through 3-3H. Please note that the strain compatible properties for generic profiles 270 and 560 listed in Tables 5.2-3 through 5.2-8 of MUAP-10001(R1) have been revised in Revision 2 of the report to show the updated results of the site response analyses.
2. The median strain-compatible soil properties listed in Tables 5.2-3 through 5.2-11 and shown in Figures 5.2-6 through 5.2-14 of the updated revision of MUAP-10001(R2) Section 5.2 are used as input for the site-independent soil-structure interaction (SSI)

analyses of US-APWR standard plant Category I buildings documented in MUAP-10006(R0). The site independent SSI analyses consider the foundations of the Category I buildings to be supported on the surface of the subgrade that is located at depth approximately 40 ft below the surface of the finished grade of the plant. Since the top 40 ft of the soil are excavated, the eight generic subgrade profiles used for the site independent SSI analyses of surface mounted foundations are obtained by removing the top 40 ft of soil from the generic profiles with median strain-compatible properties developed in Technical Report MUAP-10001. The layering of the profiles are adjusted in order to ensure that the ACS SASSI models are capable of transmitting seismic waves with frequencies equal to or lower than the cut off frequency of SSI analyses. The figures below show comparison of the profiles of the median shear wave velocities obtained from the site response analyses in MUAP-10001 and the profiles of the shear velocities used as input for the SSI analyses presented in MUAP-10006. Please note that in response to RAI 625-4924, it has been explained that one soil profile, the "270-100" profile and its associated Table 5.2-3 and Figure 5.2-8, is to be deleted from suite of profiles considered in the standard design.

3. Refer to MUAP-10001 Section 4.2.2. The properties are equivalent to strain-iterated properties. The approach for the US-APWR CSDRS strain compatible properties is that the properties are developed in a fully probabilistic manner, in which each base-case profile is randomized in velocity as well as nonlinear dynamic material properties. Thirty realizations were generated for each profile category and depth to hard or soft rock. Random vibration theory (RVT) equivalent-linear site response analyses were then performed on each random profile for horizontal motions while for vertical motions, linear analyses were used by assuming that the soil compressional velocities are not strain dependent. Section 5.2.1 of MUAP-10001 provides more information regarding the approach used for site response analyses for vertical motions and how the results of these analyses relate to the vertical CSDRS design spectrum which is based on the RG 1.60 V/H ratio. For the horizontal component site response analyses, modulus reduction and hysteretic damping curves from EPRI TR-102293 are used. The curves are appropriate for generic soils comprised of gravels, sands, and low PI clays. For the soft and firm rock profiles (560m/sec and 900m/sec), an unpublished suite of curves appropriate for soft and firm rock conditions were used. The rock curves were developed during the EPRI project (refer to TR-102293) assuming soft and firm rock exhibits a nonlinear dynamic material behavior similar to gravels. The rock curves were not included in TR-102293 as the final suite of amplification factors was based on soil profiles intended to capture the behavior of soils ranging from gravels to low plasticity sandy clays at CENA nuclear power plants.

4. Random vibration theory (RVT) was used with equivalent-linear site response (EPRI, 1993; Silva et al., 1996) to develop the strain compatible properties (MUAP-10001 (R1), Section 5.2.1). In this approach time histories are not required as random process theory is used to estimate peak cyclic shear strains as well as oscillator response (5% damped response spectra; MUAP-10001 (R1), Figure 5.2-3). Please refer to Section 5.2.1 of MUAP-10001 for more information on the point-source model used to generate the ground motions used as input for the RVT based site-response analyses.

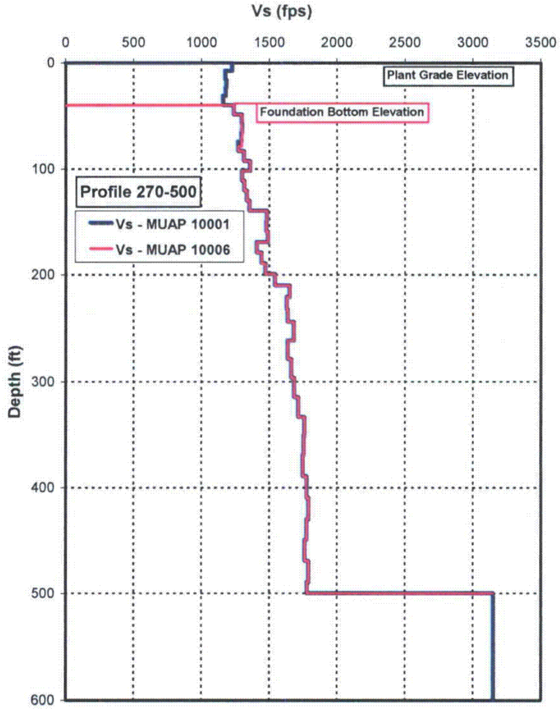
5. Because far fewer measured compressional-wave velocity profiles are available compared to shear-wave velocity profiles, due principally to surface geophysical techniques, significantly more judgment had to be used in developing the generic compressional-wave profiles. The general approach involved averaging available measured shear- and compressional-wave profiles binned into similar surficial geology or site category such as Geomatrix (Silva, 1997). For example, profile 270m/sec is close to shear-wave velocity averages for Geomatrix categories C and D or alternatively Quaternary Alluvium. Similarly profile 560m/sec is close to Geomatrix A and B or Tertiary Bedrock (Silva et al., 1999). For each profile bin reflecting a category (e.g. firm soil, close to 270m/sec), median velocity and Poisson ratio profiles were computed which were then

smoothed to produce smooth generic profiles. At this point, as the number of available measured profiles falls off rapidly beyond about 100 ft, the smoothed profiles were extrapolated to the required depths using the shallower more well constrained portions as guides along with the remaining deep measured profiles. To achieve the desired $\overline{V_S}$ (30m), the closest measurement driven smooth generic shear-wave velocity profile was adjusted typically by the addition or subtraction of a constant factor. To generate a corresponding or companion smooth generic compressional-wave profile, the corresponding Poisson ratio was applied to the adjusted shear-wave profile. This process would typically not result in a smooth compressional-wave profile increasing with depth in a manner consistent with the companion shear-wave velocity profile. At this point the derived (from Poisson ratio) compressional-wave profile was adjusted followed by a computation of the corresponding Poisson ratio. This process was iterated upon to achieve both a smooth and realistic compressional-wave profile (e.g. generally mirroring the gradient in the shear-wave velocity profile) as well as a smooth and realistic Poisson ratio profile. Consistency in Poisson ratios between profile categories also provided a constraint such that the Poisson ratio profiles either decreased or remained the same with increasing category stiffness.

References:

Silva, W.J. (1997). "Characteristics of vertical strong ground motions for applications to engineering design." *Proc. Of the FHWA/NCEER Workshop on the Nat=l Representation of Seismic Ground Motion for New and Existing Highway Facilities,* I.M. Friedland, M.S Power and R. L. Mayes eds., Technical Report NCEER-97-0010.

Silva, W. J.,S. Li, B. Darragh, and N. Gregor (1999). "Surface geology based strong motion amplification factors for the San Francisco Bay and Los Angeles Areas."A PEARL report to PG&E/CEC/Caltrans, Award No. SA2120-59652.



Vs (fps)
0
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3000
3500
0
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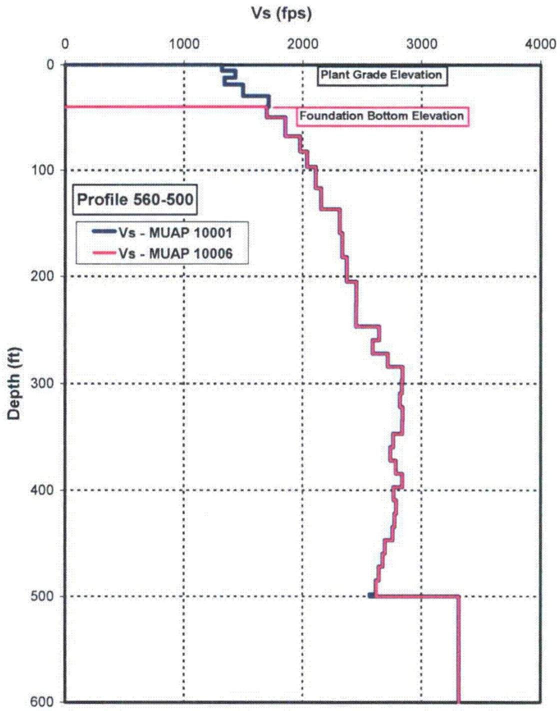
Depth (ft)
Plant Grade Elevation
Foundation Bottom Elevation
Profile 270-500
Vs - MUAP 10001
Vs - MUAP 10006



Vs (fps)
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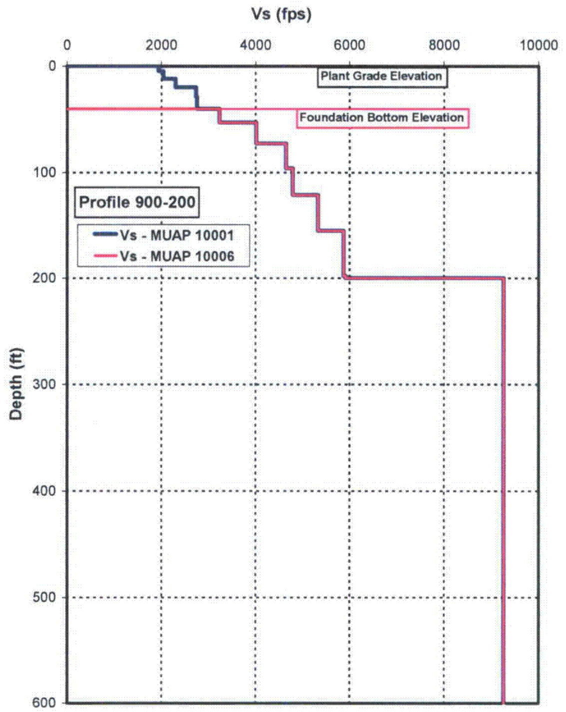
Depth (ft)
Plant Grade Elevation
Foundation Bottom Elevation
Profile 560-500
Vs - MUAP 10001
Vs - MUAP 10006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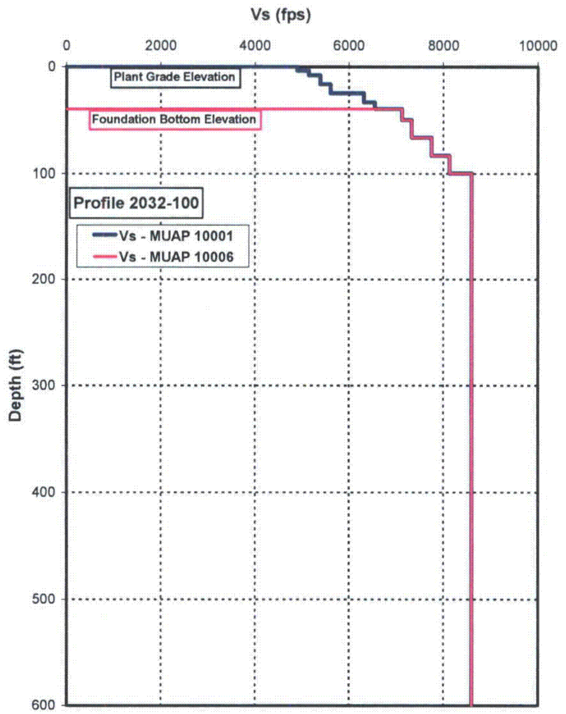
Depth (ft)
Plant Grade Elevation
Foundation Bottom Elevation
Profile 900-200
Vs - MUAP 10001
Vs - MUAP 10006


Vs (fps)
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300
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600
Depth (ft)
Plant Grade Elevation
Foundation Bottom Elevation
Profile 2032-100
Vs - MUAP 10001
Vs - MUAP 10006

SUPPLEMENTAL RESPONSE:

On the February 7, 2011 US-APWR NRC Weekly DCD Chapter 3 conference call, the NRC requested the following additional information: (1) description and justification of a decrease in shear wave velocity with increase in depth, as observed for some of the standard plant profiles whose strain-compatible properties are presented in Section 5.2.2 of MHI Technical Report MUAP-10001, and (2) provide copy of the referenced unpublished curves in Part 3 of 5 of the original response for the NRC staff's review.

(1) Description and Justification of Shear Wave Velocity Decrease versus Increase in Depth

The decrease in velocity with depth from about 400 ft to the base of the 560-500 profile at 500 ft (refer to MUAP-10001 Table 5.2-8) is due to equivalent-linear softening in the soft rock profile. Figure 1 below shows the median and $\pm 1\sigma$ estimates for the strain-compatible shear-wave velocities compared to the initial low-strain profile. The initial profile reflects a slow increase in velocity with depth beyond about 300 ft; the increase is attributed to additional crack closure due to increasing confining pressure at high velocity (> 2,500 ft/sec). The corresponding peak cyclic shear strains (median and $\pm 1\sigma$ estimates) are shown in Figure 2 below and show a decrease near a depth of 300 ft with generally increasing shear strains below that depth. The trend in the cyclic shear strains is reflected in the strain-compatible shear-wave velocities in Figure 1 with little reduction at a depth of about 300 ft and an increasing reduction in velocity as depth increases. As expected, the opposite trend is evident in the strain-compatible damping shown in Figure 3 below.

These trends are not unusual for strain-compatible properties, particularly for cases where a soil or soft rock profile is underlain by stiff (e.g. basement) material which is assumed to behave with significantly more linear response. The presence of relatively stiff basement material results in an increase of motions, as the interface is approached, over motions that would result if the basement material were absent. Similarly, the presence of the basement material at the analyzed depth results in an increase of motions over motions that would result if the basement material were at a much greater depth.

The 270-500 profile shows a similar but much more subdued trend, as illustrated in Figure 4 for the shear-wave velocities (see also Table 5.2-5 in MUAP-10001). In Figure 4, the shear-wave velocities just above 500 ft show a light decrease resulting from the increase in cyclic shear strains at the same depth, illustrated in Figure 5. The corresponding increase in shear-wave damping is shown in Figure 6.

In terms of directly measuring or validating strain-compatible properties, no data or analysis procedures exist that can provide reliable measurements or estimates of cyclic shear strains during high loading conditions, especially for soft rock profiles. Vertical array data, which have been analyzed extensively, can provide estimates of shear strains, but the resolution is poor because instruments are spaced widely in depth. Also, because in-situ estimates of cyclic shear strains require differences in displacements, noise contamination is also a significant issue.

In lieu of direct in-situ validation of strain-compatible properties, it is preferable to rely on validations of recorded motions (5% damped response spectra) using equivalent-linear site response. For the equivalent-linear site response, the only controlling parameters are the shear-wave velocity and damping at the levels of strains of interest. If vertically propagating shear-waves dominate motions (at least for periods of interest, ≤ 2 to 4 seconds) and the model reflects predictions with little bias and a low variability, the equivalent-linear approach of vertically propagating shear-waves must necessarily result in shear-wave properties appropriate for the specific loading level. This approach was considered at several sites in which the equivalent-linear methodology provided matches to recorded motions (5% damped spectra) that were considered of sufficient accuracy for engineering design (EPRI 1993).

It is worth noting that the higher degradation of shear velocities at the interfaces between the softer and harder subgrade materials results in strain-compatible profiles that minimize the radial dissipation of energy (geometric damping) in the soil-structure interaction (SSI) system due to sharper contrasts in the shear wave velocities at these interfaces. The lower damping of the SSI system results in higher peak responses and higher seismic design demands which are enveloped as part of the standard plant design process. In light of these considerations, it can be concluded that the generic soil profiles used as input for the standard design of US APWR will result in seismic responses that will envelope the responses across a wide range of sites within the central and eastern US.

In summary, although decreases in shear-wave velocities occur as depth increases for some soil profiles, this phenomenon is not unusual for strain-compatible properties, particularly for cases where a soil or soft rock profile is underlain by stiff material. The effects of this phenomenon are appropriately captured in the resulting SSI responses used for design of US-APWR standard plant structures.

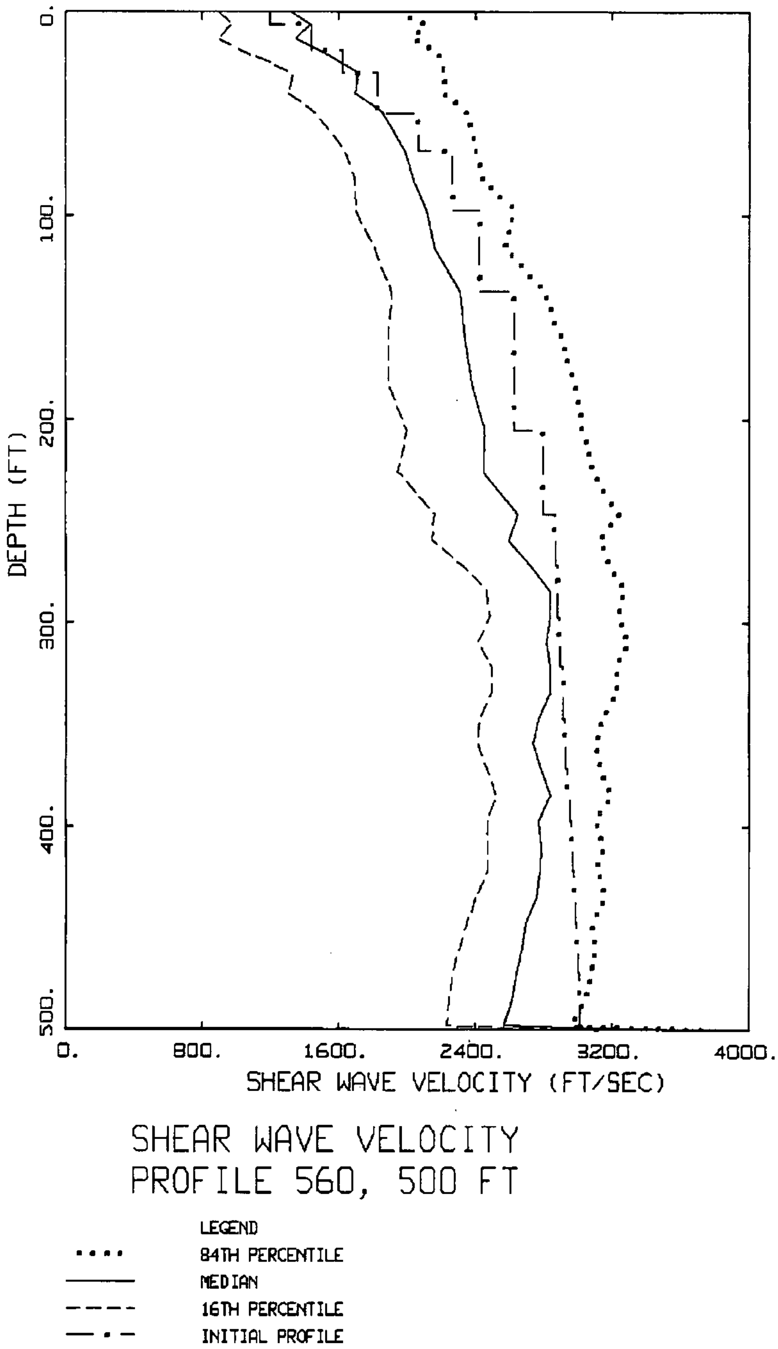


Figure 1. Strain-compatible shear-wave velocity developed for profile 560m/sec and 500 ft depth along with the initial profile.

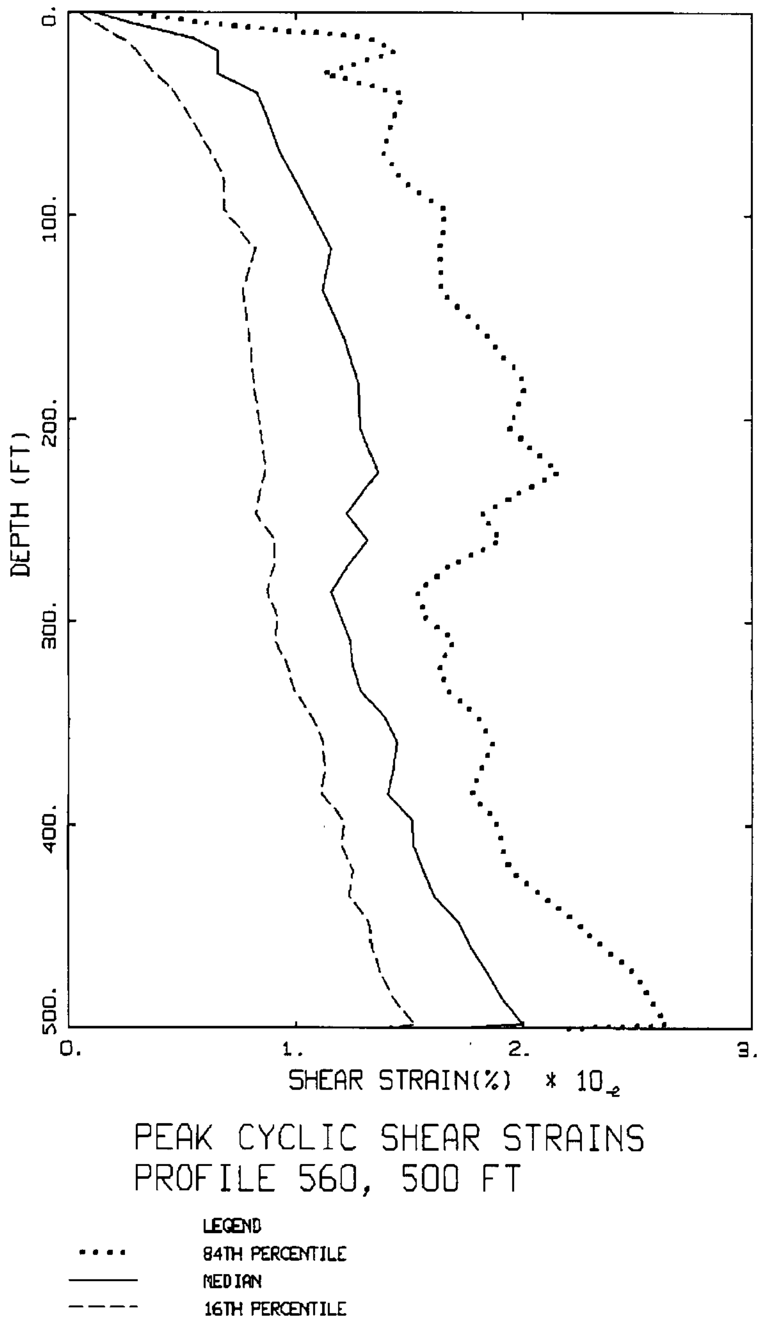


Figure 2. Peak cyclic shear strains developed for profile 560m/sec and 500 ft depth.

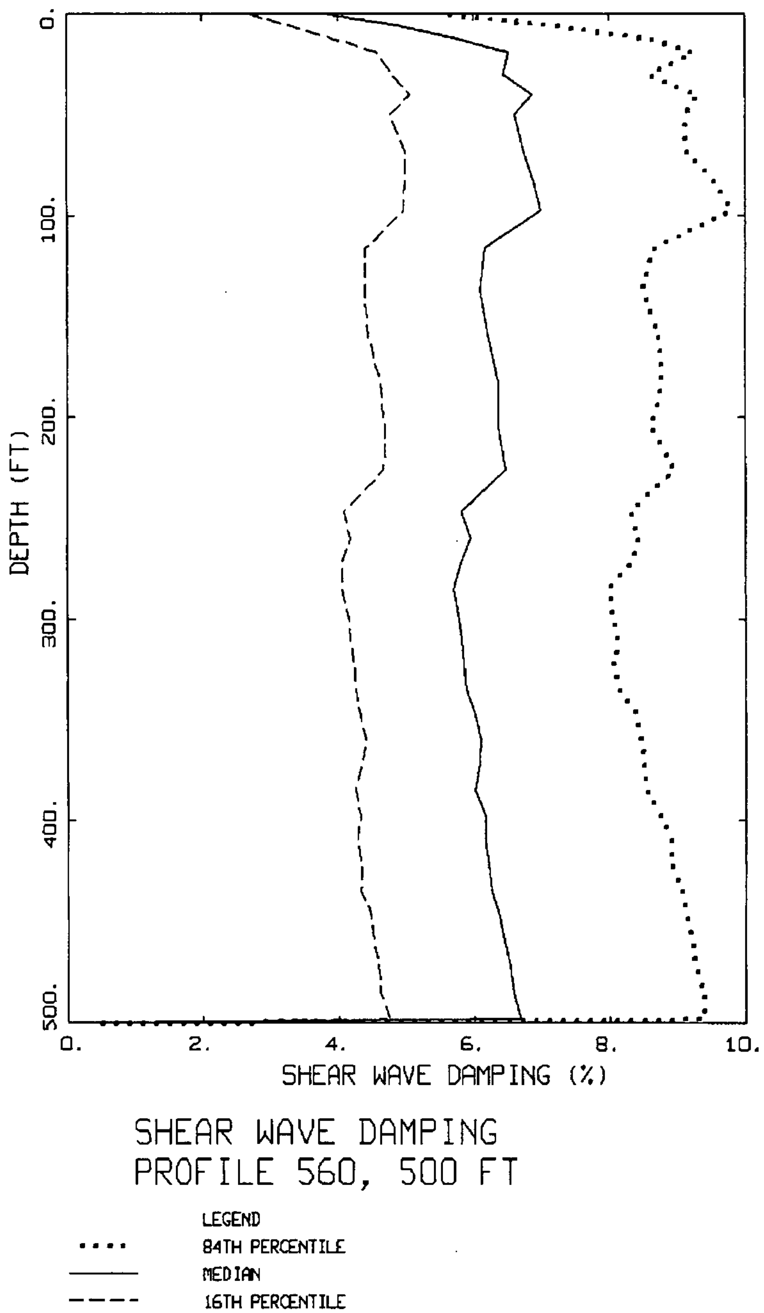


Figure 3. Strain-compatible damping developed for profile 560m/sec and 500 ft depth.

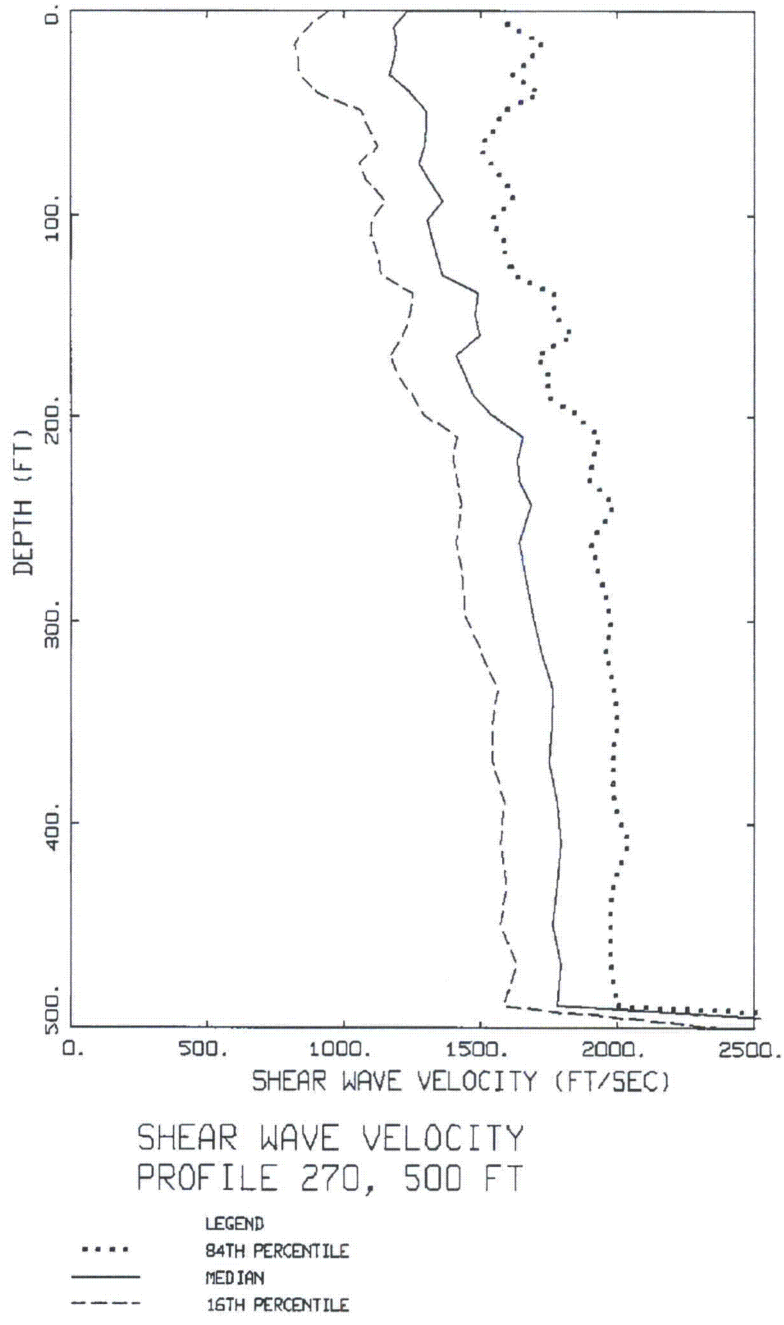


Figure 4. Strain-compatible shear-wave velocity developed for profile 270m/sec and 500 ft depth.

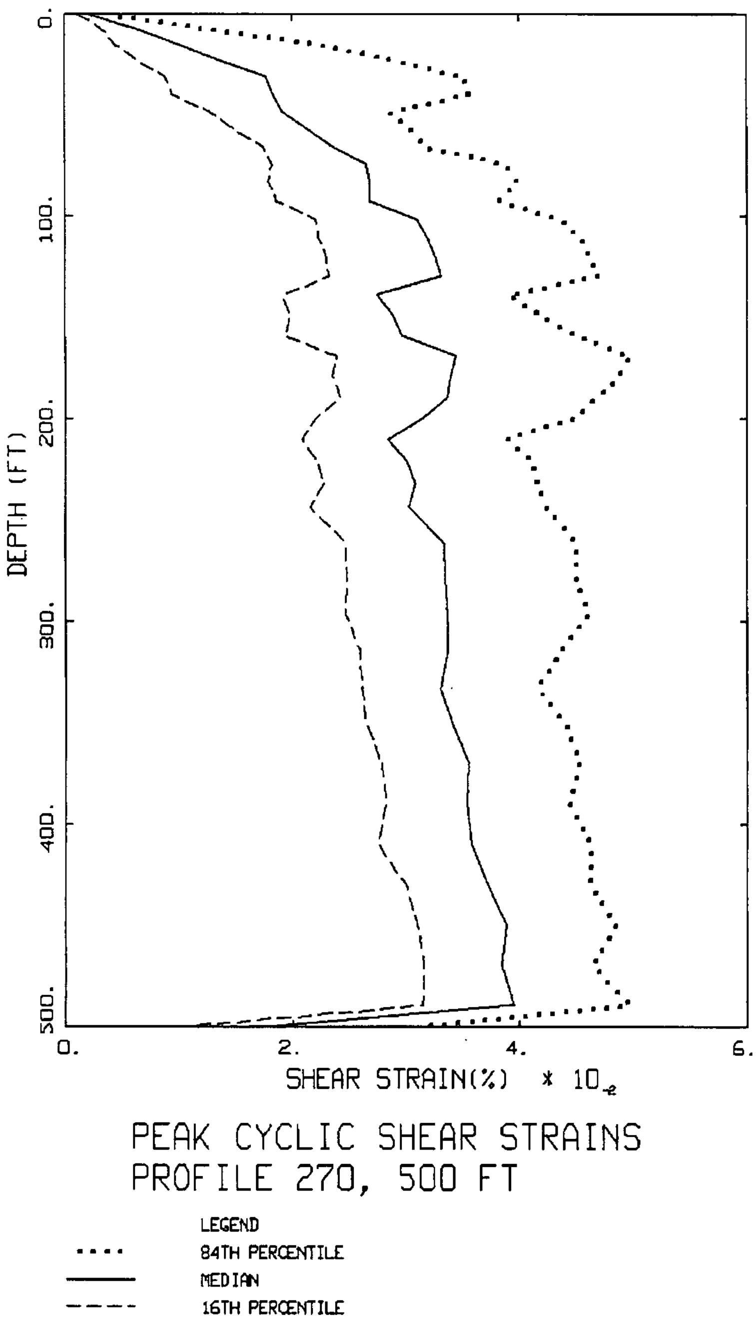


Figure 5. Peak cyclic shear strains developed for profile 270m/sec and 500 ft depth.

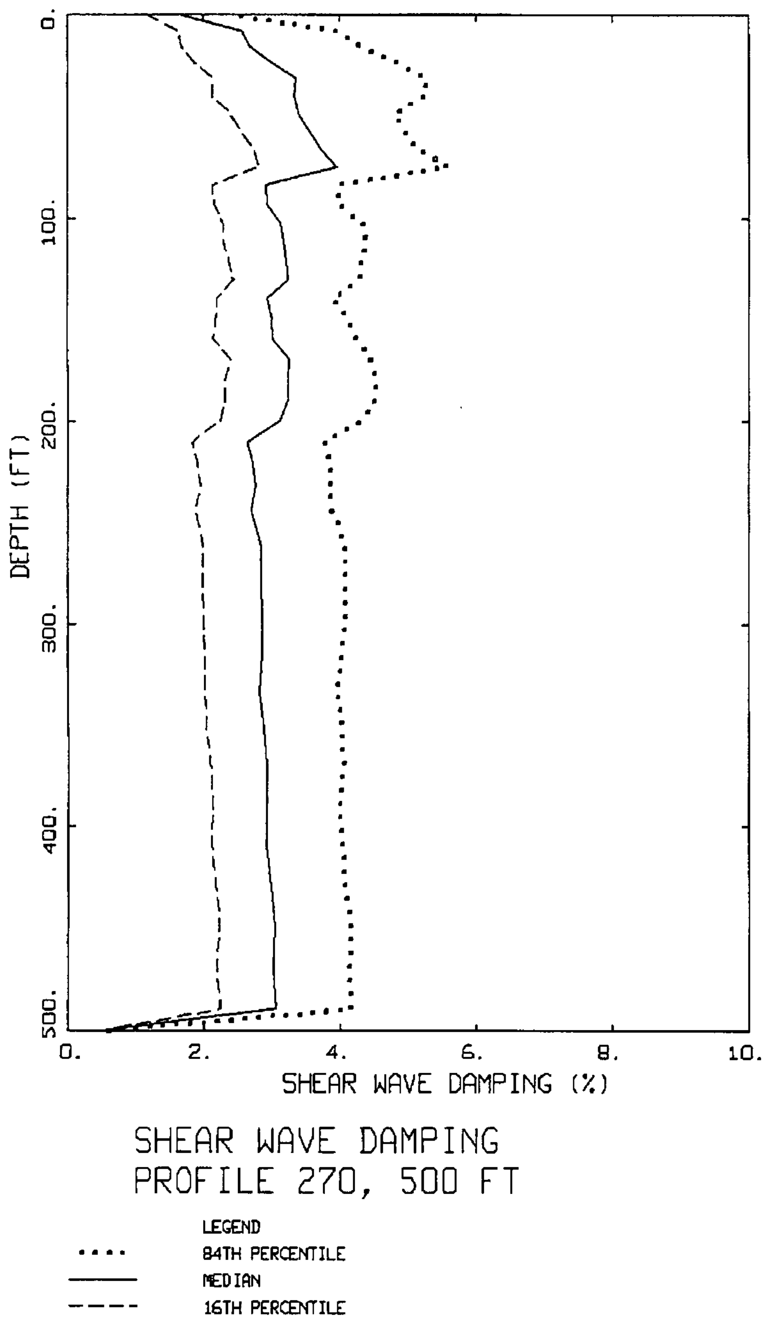


Figure 6. Strain-compatible damping developed for profile 270m/sec and 500 ft depth.

(2) Provide Unpublished Curves Referenced in Part 3 of 5 of initial response

The response to item (3) in the original RAI stated that an unpublished suite of modulus reduction and hysteretic damping curves appropriate for soft and firm rock conditions were used for development of the horizontal component site response analyses. The rock curves were developed during the EPRI project (refer to TR-102293) assuming soft and firm rock exhibits a nonlinear dynamic material behavior similar to gravels. The rock curves were not included in TR-102293 as the final suite of amplification factors was based on soil profiles intended to capture the behavior of soils ranging from gravels to low plasticity sandy clays at CENA nuclear power plants. Those curves are presented and explained as follows.

The curves used for the soft and firm rock sites (560m/sec and 900m/sec), which are shown in Figure 7 below and also shown in Figure 4.2-2 of MUAP-10001, were an outgrowth of the EPRI Ground Motion Study (EPRI, 1993), which, amongst many other things, included development of the depth dependent generic modulus reduction and damping curves for sandy materials that are shown in Figure 8 below. The "rock" curves are not shown in Appendix 7A of the EPRI (1993) report, which details the development of the "sand" curves, but are based on the range suggested for modulus reduction and damping of gravels. The general positioning of the "sand" and "gravel" curves has subsequently been confirmed by the work of Darendeli (2001) and Menq (2003) at the University of Texas. Formal development of modulus reduction and damping curves for "rock" is difficult because not only does this designation cover a wide range of materials but all these material are effectively impossible to sample and test in the laboratory and attempts to determine modulus reduction and damping from field measurements are still in their infancy. At shallow depths, however, say less than 200-300 feet, rock-like materials are often weathered and almost invariably fractured. Weathering and fractures affect the properties of the rock-like materials such that it is not unreasonable to assume modulus reduction and damping curves that are similar to those for gravels, since it is now well established that material behavior becomes more nonlinear as the particle size increases – as one goes from clays to sands to gravels and rock mass. The sensitivity of the results of site response analyses to the selection of modulus reduction and damping curves for rock-like material is in any case limited by the fact that they generally have higher stiffnesses and thus develop lower shear strains than do soil-like materials for the same level of loading. At much greater depths, where weathering is absent and fractures are likely to remain closed, even under strong shaking, the behavior of rock-like materials can safely be assumed to remain largely elastic so that modulus reduction and damping curves may no longer be applicable. It should be pointed out that the increase in velocity with depth at the rock profiles (560m/sec and 900m/sec) is such that for depths exceeding several hundred feet the shear-wave velocities exceed about 2,500 ft/sec, resulting in relatively low strains at the moderate loading levels associated with the 0.3g zero period acceleration of the CSDRS. For example, the soft rock 560m/sec profile has a shear-wave velocity of about 2,900 ft/sec (883m/sec) at a depth of about 300 ft (91m). At a 0.3g loading level the corresponding peak cyclic shear strains are about 0.012% with a strain-compatible velocity of about 2,800 ft/sec, reflecting a modest reduction of about 4%.

References:

Electric Power Research Institute (1993). "Guidelines for determining design basis ground motions." Palo Alto, Calif: Electric Power Research Institute, vol. 1 5, EPRI TR 102293.

Menq, Farn-Yuh (2003). "Dynamic properties of sandy and gravelly soils" Presented to the faculty of the graduate school of The University of Texas at Austin in Partial Fulfillment of the Requirements for the degree of Doctor of philosophy.

Darendeli, Mehmet B. (2001). "Development of a new family of normalized modulus reduction and material damping curves." Presented to the faculty of the graduate school of The University of Texas at Austin in Partial Fulfillment of the Requirements for the degree of Doctor of philosophy.

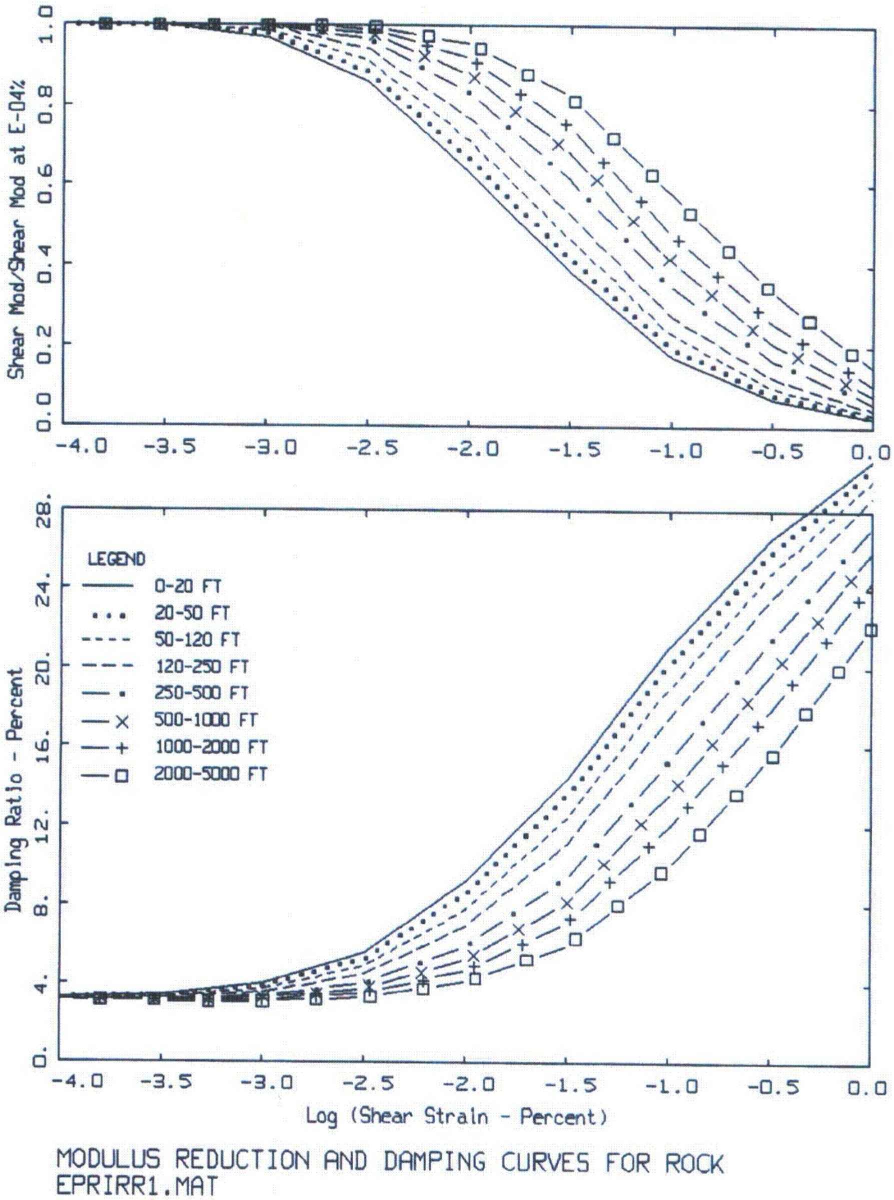
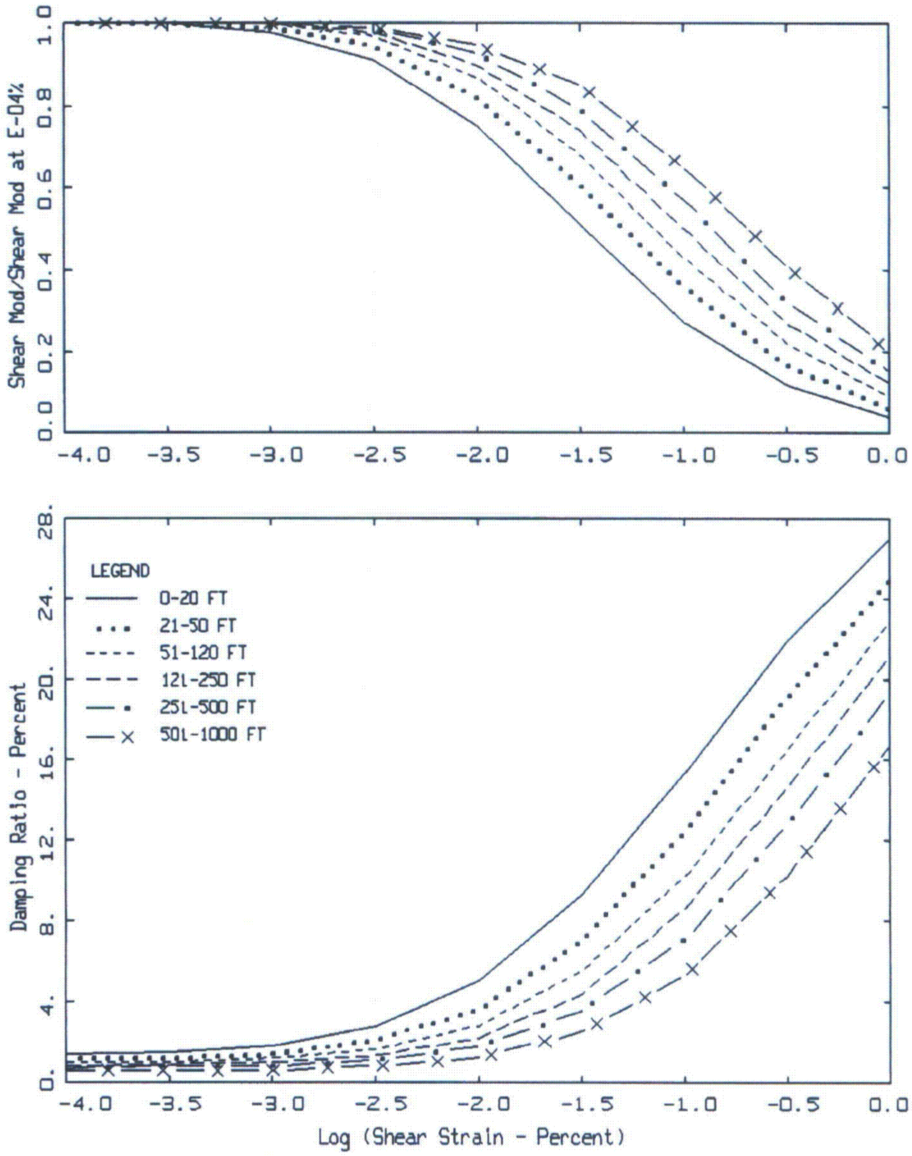


Figure 7. Rock modulus reduction and hysteretic damping curves developed for rock site conditions.



MODULUS REDUCTION AND DAMPING CURVES FOR COHESIONLESS SOILS
EPRISR1.MAT

Figure 8. EPRI (1993) modulus reduction and hysteretic damping curves developed for cohesionless soils.

Impact on DCD

There is no impact on the DCD.

Impact on R-COLA

There is no impact on the R-COLA.

Impact on S-COLA

There is no impact on the S-COLA.

Impact on PRA

There is no impact on the PRA.

This completes MHI's response to the NRC question.