RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

1/31/2013

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 821-5984 REVISION 3**
SRP SECTION:	**03.07.01 – Seismic Design Parameters**
APPLICATION SECTION:	**3.7.1**
DATE OF RAI ISSUE:	**09/01/11**

QUESTION NO. RAI 03.07.01-18:

In the supplemental response of June 2011 to RAI No. 659-5133 Revision 2, Question No. 3.7.1-17, the applicant provided a description and justification for the decrease in shear wave velocity with increase in depth. The applicant is requested to provide additional justification to the following observations from the supplement response to ensure that the soil profiles used in the standard design realistically represent typical site conditions in the US and the approaches used in the development of strain compatible soil profiles are conservative in order for the staff to fully evaluate the adequacy of the US-APWR design in accordance with the requirements of 10 CFR Part 50, Appendix A, General Design Criteria 2; 10 CFR Part 50 Appendix S; and 10 CFR Part 100.23.

1. The staff observed that the behavior of increase in shear wave velocity with depth in not limited to the 560-500 profile but is evident in all strain compatible profiles for postulated soil sites, namely 270 m/s and 560 m/s soil profile series. Please explain why the shear wave velocity in the initial profiles never decreases but increase with depth as defined in the standard design.

2. It is well known that when shear wave propagates from bedrock up to the surface horizontally through a softer media, the ground motion at the surface normally will be higher than the bedrock input motion, or the soil column acts as an amplifier. The strain compatible shear wave velocity at deeper depth normally will be higher than that at a shallow depth if the whole soil column consists of the same type of material. Even if the increased motion at the softer-stiffer material interface may increase shear strain and thus results in the decrease of shear wave velocity, why does the strain compatible shear wave velocity not only decrease at the depth close to the interface but also at depths way above the interface for all 270 m/s and 560 m/s soil profiles series?

3. There is no reliable data to verify the actual soil shear wave velocity at deep depth when under seismic loading. Please explain how the strain compatible soil profiles where the shear wave velocity decreases as depth increases, as presented in this design, can be considered realistic.

4. Although the soil profiles with decreased shear wave velocity can result in higher peak responses; will similar results be obtained over all frequency range in the soil-structure interaction (SSI) analysis?

5. The shear wave velocity in some profiles decreases, then increases, and then decreases again within certain depth ranges. This behavior appears to have nonphysical attributes caused by the numerical instabilities in the computer code that was used to develop the soil profiles. Please

provide justification for the behavior of the shear wave velocity and details of the computer code used including its validation and verification performed for this application.

ANSWER:

1. The reference to the "initial" or low-strain profiles is shown in Figure 01.4.2-1 of MUAP-10006, Rev. 3. The increases in shear wave velocity with depth for the profiles shown in Figure 01.4.2-1 are a consequence of averaging multiple profiles (Reference 1). This approach is used to create generic site conditions taken to reflect a range in site-specific site conditions through randomization about each initial profile. The set of generic profiles are determined to be adequate for design because the suite of profile resonances (column frequencies) span the frequency range of interest for the standard plant structures.

2. Shear-waves (horizontal motion for S_H waves and vertical and horizontal motion for inclined S_V waves) propagate vertically from depth to the surface. High-frequency surface motions may be greater or lower than the corresponding bedrock motions, depending upon loading level and nonlinear dynamic material properties. For profiles of uniform material type with a monotonic increase in velocity with depth, the distribution in cyclic shear-strains and resulting strain-compatible properties can be complex depending on loading level, steepness of the velocity gradient, and distribution of G/G_{max} and hysteretic damping curves with depth. Depending on the velocity gradient, downgoing wavefields may be reflected upward above the bedrock resulting in peaks and troughs in cyclic shear strains with depth due to superposition of upgoing and downgoing wavefields. Superimposed on this behavior is the variation in nonlinear dynamic material properties with depth to accommodate confining pressure effects on nonlinearity (Reference 2). This interaction can result in cyclic shear-strain concentrations with depth and a non-monotonic change in strain-compatible shear-wave velocity and damping with depth. As the non-monotonic pattern in strain-compatible properties with depth becomes more pronounced, the more nonlinear are the suite of G/Gmax and hysteretic damping curves.

   It should be noted that the change in G/G_{max} and hysteretic damping curves to a more linear suite in Technical Report MUAP-10006 Rev. 3 reduced the non-monotonic change in velocity and hysteretic damping, and therefore peak cyclic shear-strain, with depth compared to MUAP-10001 Rev. 1, which was the basis for the original RAI No. 659-5133 Revision 2, Question 3.7.1-17.

3. As discussed in parts (1) and (2) above, the generic profiles in Technical Report MUAP-10006 exhibit reduced non-monotonic changes in velocity and hysteretic damping, and therefore peak cyclic shear-strain, with depth. For the equivalent-linear approach to site response, the controlling parameters for the soil motions are the frequency independent shear-wave velocity and hysteretic damping at the final iteration. Provided the site response achieves response spectral levels in overall agreement with recorded motions over a wide frequency range in general and at vertical arrays in particular, where the control motion is specified, the variation of strain-compatible properties with depth must be consistent with the in-situ shear-wave velocity and hysteretic damping during strong shaking, and therefore can be considered realistic. The technical literature contains numerous examples of favorable comparisons of equivalent-linear site response estimates with recorded motions validating the equivalent-linear approach and, consequently, the resulting strain-compatible dynamic material properties (for example, References 1 and 3).

4. Peak responses in the soil profiles will yield similar results in soil-structure interaction (SSI) responses over the frequency range of the standard plant SSI analyses. Amplification of the responses will occur at those frequencies where the structural frequencies are resonant with the soil column frequencies. The selection of generic profiles with a wide variety of dynamic

properties (as discussed in Sections 01.3.2 and 01.4.2.1 of Technical Report MUAP-10006 and the broad-band content of the certified seismic design response spectra (CSDRS) (as discussed in Section 01.4.2.2 of Technical Report MUAP-10006, Rev 3) assure that seismic responses captured in the SSI analyses and used for design are appropriate for the standard plant. Further, COL Item 3.7(25) requires site-specific SSI be performed, which provides assurance that any potential site-specific resonance effects are adequately captured and enveloped by standard plant design ISRS.

5. The equivalent-linear approach fundamentally does not suffer from numerical instabilities as it is fundamentally an exact linear process in terms of wave propagation for a system comprised of constant velocity and constant damping layers. As such, there are no stability conditions and the solution is exact to arbitrary low- as well as high-frequency. The only numerical issue is the possible lack of convergence in peak cyclic shear strains with shear modulus and damping at high loading levels for initially soft sites that have highly nonlinear G/G_{max} curves. However, for these cases an equivalent-linear solution does not exist at those loading levels and other analysis approaches (e.g., nonlinear) must be pursued.

   The computer code RASCALS, which employs the Random Vibration Theory approach is used in conducting the site response analyses. The site response analyses are described in Section 01.5.2.1 of Technical Report MUAP-10006, Rev 3.

   The RASCALS SET V1.1 software suite is used to develop seismic input for pre- and post-closure design and analysis. The software suite includes two primary computational modules (RASCALS V5.5, RASCALP V2.2), two preprocessing modules (VELAVG V2.0, RANPAR V2.2), and one postprocessing module (SCP V1.0). The entire RASCALS software suite was developed by Pacific Engineering & Analysis. The test standards against which the V&V test cases were compared include hand calculations, computerized computations using different methodology, graphical results comparisons, and subjective visual comparison.

References:

1. I.M. Friedland, M.S Power and R.L. Mayes eds., W.J. Silva, "Characteristics of Vertical Strong Ground Motions for Applications to Engineering Design," NCEER-97-0010, Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities, 1997.
2. TR-102293, Electric Power Research Institute (EPRI), Palo Alto, CA, 1993, "Guidelines for Determining Design Basis Ground Motions,"
   - Vol.1: Methodology and Guidelines for Estimating Earthquake Ground Motion in Eastern North America
   - Vol. 2: Appendices for Ground Motion Estimation
   - Vol. 3: Appendices for Field Investigations
   - Vol. 4: Appendices for Laboratory Investigations
   - Vol. 5: Quantification of Seismic Source Effects
3. N. Abrahamson, G. Toro, C. Costantino, and W.J. Silva, "Description and Validation of the Stochastic Ground Motion Model," Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc., Upton, New York, 1996.

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.

Impact on Technical/Topical Report

There is no impact on the Technical/Topical Report.

This completes MHI's response to the NRC's question.