KEY TECHNICAL ISSUES WHEN DEVELOPING PROBABILISTIC FOUNDATION INPUT RESPONSE SPECTRA FOR SEISMIC ANALYSIS

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Abstract

Many existing nuclear facilities have undertaken updated Probabilistic Seismic Hazard Analysis(PSHA) in response to the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami. For nuclear facilities located in eastern North America the updated PSHA must accurately account for high-frequency (> 10 Hz spectral frequency) ground motions and may include the assessment of site response resulting from site-specific geotechnical conditions. Recent experience at a site in eastern Canada suggests that the assessment of high frequency ground motion critically depends on the site response model. This experience will be summarized including (1) the development of a site geotechnical model that reflects available site data, accounting for epistemic uncertainty and aleatory variability, (2) the derivation of frequency-dependent site amplification factors, and (3) combining the amplification factors with thePSHA results to develop Foundation Input Response Spectra (FIRS), with emphasis on modeling spectral frequencies above 10 Hz.

Keywords: Seismic hazard, ground motion, site response, Foundation Input Response Spectra.

1. Introduction

Many existing nuclear facilities have undertaken updated Probabilistic Seismic Hazard Analysis (PSHA) as part of licensing renewal and ensuring that the assessment of natural hazards reflects up-to-date data, models, and methods. Additionally many facilities are also updating the assessment of natural hazards in response to the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami. For nuclear facilities located in eastern North America the updated PSHA must account for high-frequency (> 10 Hz spectral frequency) ground motions associated with hard rock site conditions (defined as having a shear wave velocity of 2,800 m/s) and may include the assessment of site response resulting from site-specific geotechnical conditions. Recent experience at the Point Lepreau Generating Station (PLGS), a site in eastern Canada, suggests that the assessment of high-frequency ground motion critically depends on the site response model.

The PSHA completed for the PLGS was for generic hard-rock site conditions. The PSHA results incorporated up-to-date inputs related to seismic source modeling, earthquake recurrence rates, maximum magnitudes for each seismic source, and ground motion prediction equations for hard-rock site conditions. The PSHA results are displayed on Figure 1 which shows the hard-rock Uniform Hazard Response Spectra (UHRS) at a Mean Annual Frequency of Exceedance (MAFE) of $1x10^{-3}$ and $1x10^{-4}$.

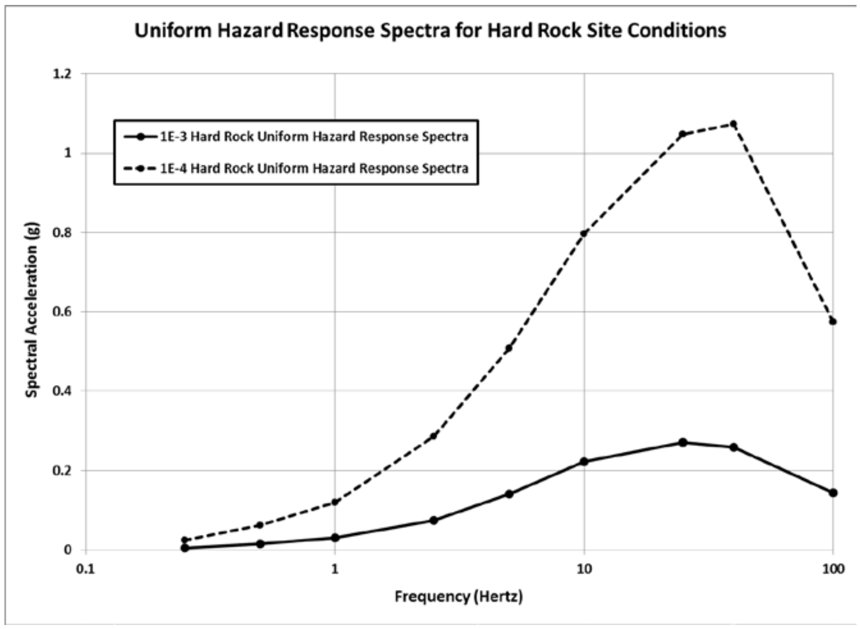

Figure 1 Uniform Hazard Response Spectra for Hard-Rock Site Conditions at a Mean Annual Frequency of Exceedance of $1x10^{-3}$ and $1x10^{-4}$.

The hard-rock PSHA results need to be adjusted to represent the site-specific geologic and geotechnical conditions for both surface and foundation conditions. The focus of this paper is to summarize the work completed to derive foundation level ground motions. The foundation level ground motions having a MAFE of $1x10^{-4}$ are labeled as Foundation Input Response Spectra (FIRS).

Recent guidance developed by the Electric Power Research Institute (EPRI) was used to quantify both aleatory variability and epistemic uncertainty in site response [1]. This guidance was developed to ensure consistency in site response analysis as part of updating nuclear site PSHAs in the United States. The available geotechnical information forms the basis for defining geotechnical model inputs (Section 2) that are used to perform a site response analysis (Section 3). The site response analysis provides site amplification functions specific to both surface and foundation conditions at the site.Results of the site response analysis are then combined probabilistically with the hard-rock PSHA results to obtain corresponding response spectra at the foundation elevation control points of interest (Section 4).

When the FIRS are compared with the hard rock UHRS having a MAFE of $1x10^{-4}$ it is seen that the site response model developed for the site reduces the high frequency ground motions. This is discussed in Section 5 with conclusions of this work described in Section 6.

2. Site geotechnical model

The site geotechnical model is comprised of a soil profile model represented by a set of horizontal layers with associated shear wave velocities (V_S) and a set of shear modulus and damping curves to model the strain degradation behaviour of this layered system. Each of these is summarized below.

2.1 Soil profile model

The geotechnical model for the PLGS is developed from available geologic and geotechnical information. The geotechnical model consists of horizontal layers characterized by V_S, density, and dynamic properties. The geotechnical model includes an assessment of uncertainty represented by alternative base-case V_S profiles and alternative sets of dynamic property curves. Development of the geotechnical model considered the guidance from [1] with respect to modeling aleatory variability and epistemic uncertainty in site response inputs.

The local site stratigraphy is based on site-specific geotechnical investigations where the site geology consists of a thick sequence of Triassic sedimentary rocks overlain by soil deposits that include peat, sand, gravels, clay, and glacial till. The focus of this discussion is on modeling the geologic layers below the soils. Based on available geotechnical investigations the elevation for the top of bedrock is taken as Elevation (EL) +10.7 meters (m), an average value at the location of critical structures, based on visual inspection of the top of rock. Given the geophysical measurements at the site, it is reasonable to assume that there is a layer of weathered rock with an average thickness of approximately 3.1 m at the location of critical structures. Given this interpretation, the elevation for the top of competent bedrock is taken at EL +7.6 m, which is the foundation elevation of interest for critical structures at the site.

The full geotechnical model includes layers above the bedrock to the ground surface. To account for uncertainty in site properties including layer thickness variation, three soil layer models were developed. Soil$V_S$values were developed using the available soil penetration test results at the site [2]. For the weathered rock layers, V_S values were determined from geophysical surveys documented in the geotechnical site investigation reports. For competent rock the $V_S$values for the upper layers were also based on the available site-specific measurements. The three soil profiles account for uncertainty in thickness of the soil and weathered rock layers.

The site response analysis required that the$V_S$model for competent rock extend to a depth where hard-rock $V_S$values would be reached. A generic V_S gradient for eastern North America of 0.5 mps/m [2], is usedwhich implies that the depth to hard rock is >1,100 m. The available site geologic and geotechnical data suggest that the thickness of the Triassic rocks is at least 610 m.

The final site profile model is displayed on Figure 2. The modeled V_S profile with depth is displayed on Figure 3

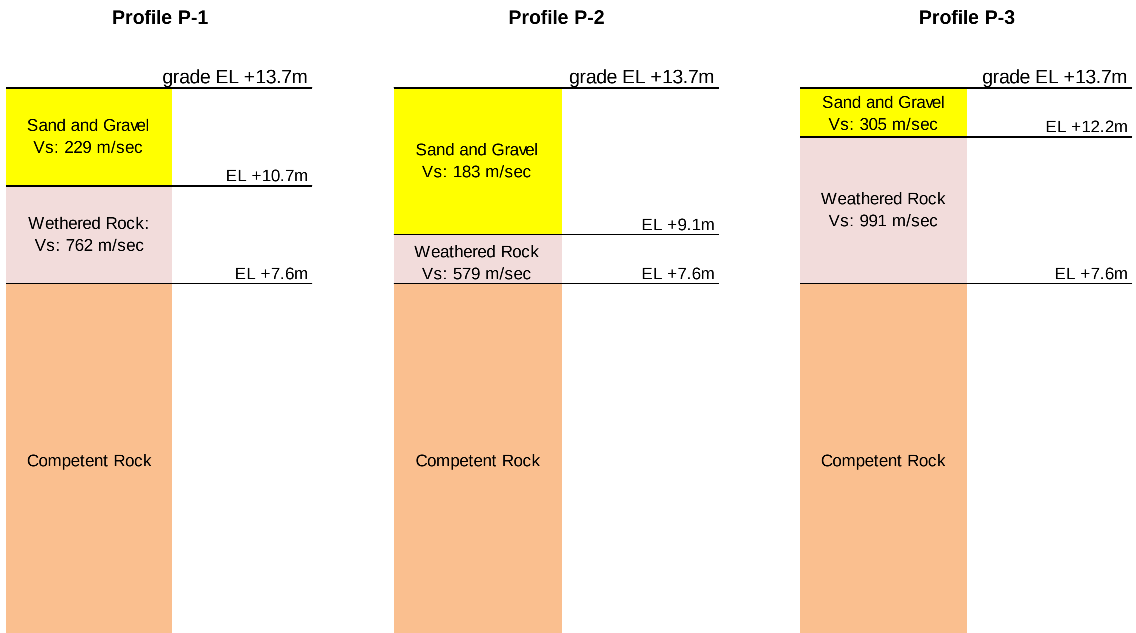

Figure 2 Geotechnical Model Used for the Site Response Analysis at the Point Lepreau Generating Station.

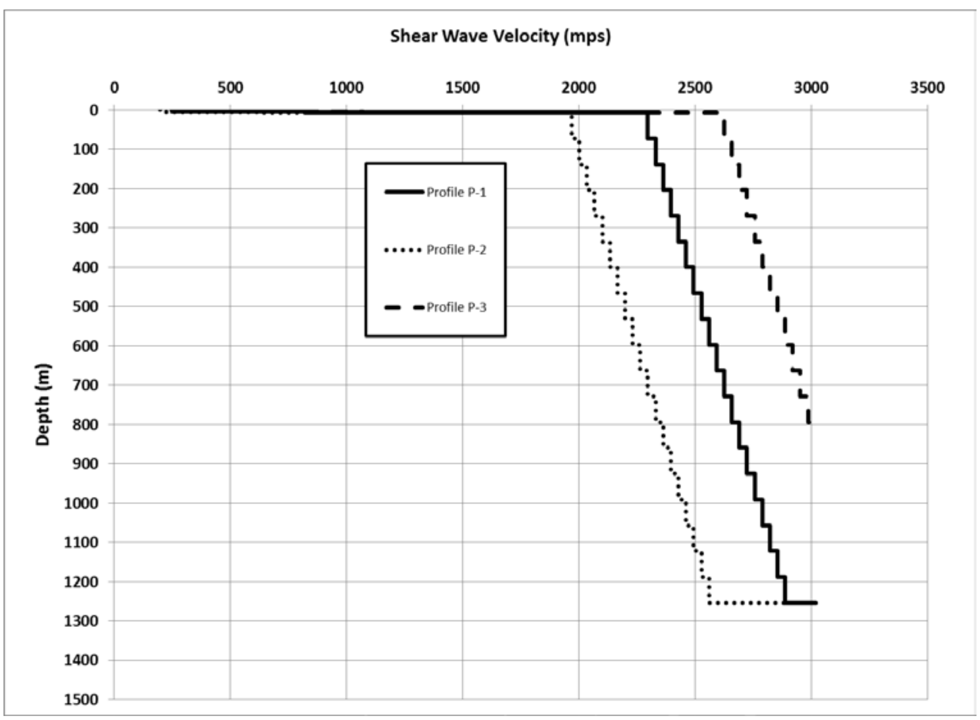

Figure 3Shear Wave Velocity Profiles Used for the Site Response Analysis at the Point Lepreau Generating Station.

2.2 Site geotechnical model

Assignment of dynamic properties considers the relationships identified in [1], and the authors' experience at other sites with thick sedimentary rock sections, and the minor amounts of clay in the soil. As part of the evaluation, normalized shear modulus (G/Gmax) and damping curves for soil were compared; these are labeled as "EPRI Soil" and "Peninsular Range" curves from [1]. Selection of the EPRI Soil and Peninsular Range dynamic property curves provides a reasonable representation of the range of uncertainty in G/Gmax and damping for the PLGS Site and addresses the minor amounts of clay at the Site.

For weathered rock, uncertainty in dynamic properties is represented by using the "EPRI Rock" curves from [1][3] and curves developed for shale [4]. For competent sedimentary rock, the shale dynamic property curves from [4] and the assumption that the material behaves linearly with the low-strain damping from [4] characterize the uncertainty in G/Gmax and damping.

3. Site response analysis

The derivation of the ground motions used to perform the site response analysis and the resulting site amplification factors are each summarized below.

3.1 Site response analysis input ground motions

To determine the input control motion for the site response analysis, a comparison was made between the normalized hard-rock UHRS and the single-corner seismological model recommended in [1]. A generic earthquake moment-magnitude (M_W) 6.5 is specified in [1] and is comparable to the site-specific controlling earthquakes (M_W 6.3 to M_W 6.6) identified in the hard-rock PSHA at a MAFE of 1×10^{-4}. The spectral shape comparison shows a reasonable agreement, and as a result, the single-corner seismological parameters from [1] are considered appropriate for developing input motions.

Because the PSHA report only provided controlling earthquakes for one MAFE, epistemic uncertainty is considered by introducing two sets of input motions in the logic tree; the first set denoted as "single-corner, constant distance" assumes that the site-specific controlling earthquake magnitude and distance at other MAFEs are the same as those for a MAFE of 1×10^{-4}, while the second set denoted as "single-corner, variable distance" uses the generalized controlling earthquakes for the Central and Eastern United States [1]. In general, the spectral shape for the range of ground motions for the "single-corner, constant distance" case are more consistent with the hard rock 1×10^{-4} MAFE UHRS. As a result, the "single-corner, constant distance" case is assessed a relative weight of 0.7 and the alternative "single-corner, variable distance" case is assessed a weight of 0.3.For the three site profiles a higher weight was given to the best-estimate profile P-1 from Figure 2. The two options for shear modulus and damping curves were given equal weight.The site response logic tree is displayed onFigure 4.

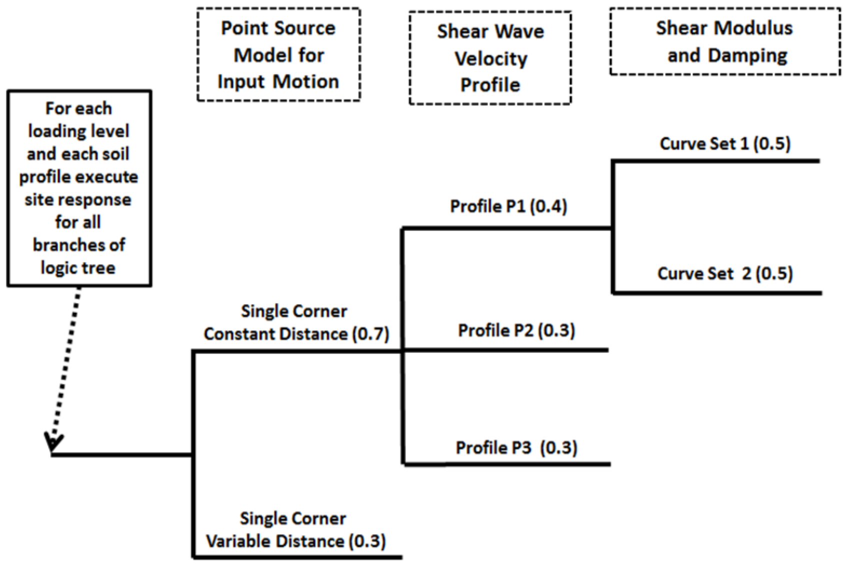

Figure 4 Logic Tree of Site Response Inputs Used to Perform Site Response Analysis at the Point Lepreau Generating Station.

3.2 Site response analysis

The site response analysis accounts for the effects of surficial geologic deposits and site geotechnical properties. Both aleatory variability and epistemic uncertainty in the critical site parameters are accounted for in the analysis. Aleatory variability in V_S profile and dynamic properties (G/Gmax and Damping curves) is modeled in the site response analysis through a randomization process. Guidance from [1] is used for representing aleatory variability of the V_S and layer thickness. A total of 60 randomized profiles are developed for each property.

Site response is modeled using an equivalent-linear approach in which free-field peak responses at the top of sublayers are solved by using the random vibration theory technique. The site materials are represented by semi-infinite horizontal layers overlying a half-space. The input ground motion is applied at the base of the model as an outcrop motion and assumed to consist of vertically propagating shear waves. Nonlinearity is accounted by using an iterative procedure to obtain values for shear modulus and damping compatible with the effective shear strains in each layer.

The amplification factors determined in the site response analysis account for both surface and embedded conditions. Site amplification factors are determined for 11 control motions (PGA from 0.01g to 1.5g) based on a set of 60 site response analyses [1] at each end branch of the logic tree shown on Figure 3. For each control motion, an analysis is performed for each of the 60 combinations of a randomized V_S profile and randomized G/Gmax and damping curves. For each analysis, the response spectrum of the computed motion at the ground surface or foundation level is divided, frequency by frequency, by the response spectrum of the rock input motion to obtain the site amplification factor. The median amplification factors for the foundation El

+7.6m are shown on Figure 5. A suite of amplification factors for 6 out of the 11 control motions are shown on Figure 6.

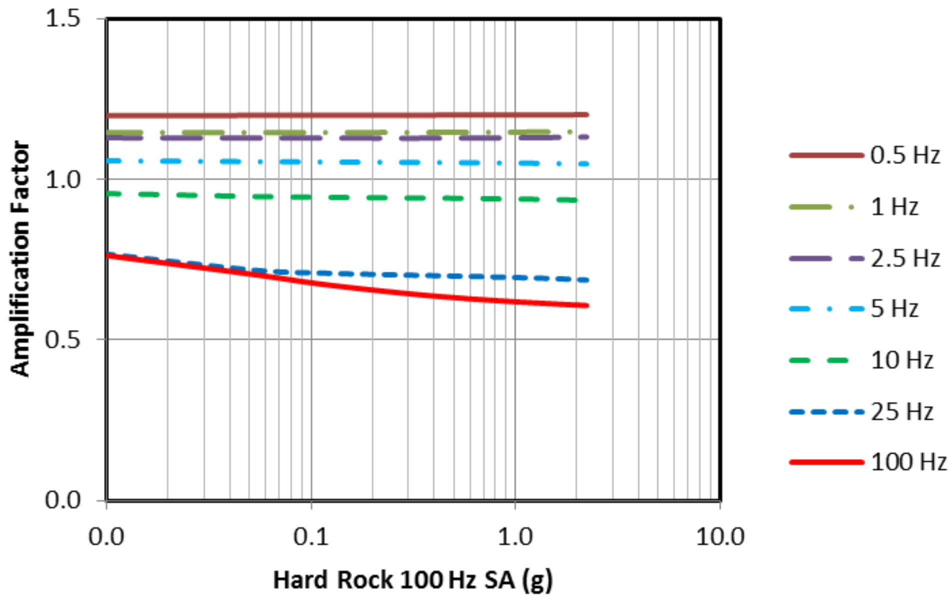

Figure 5 Median Total Amplification Factors versus Input Ground Motions for Foundation Elevation + 7.6 m.

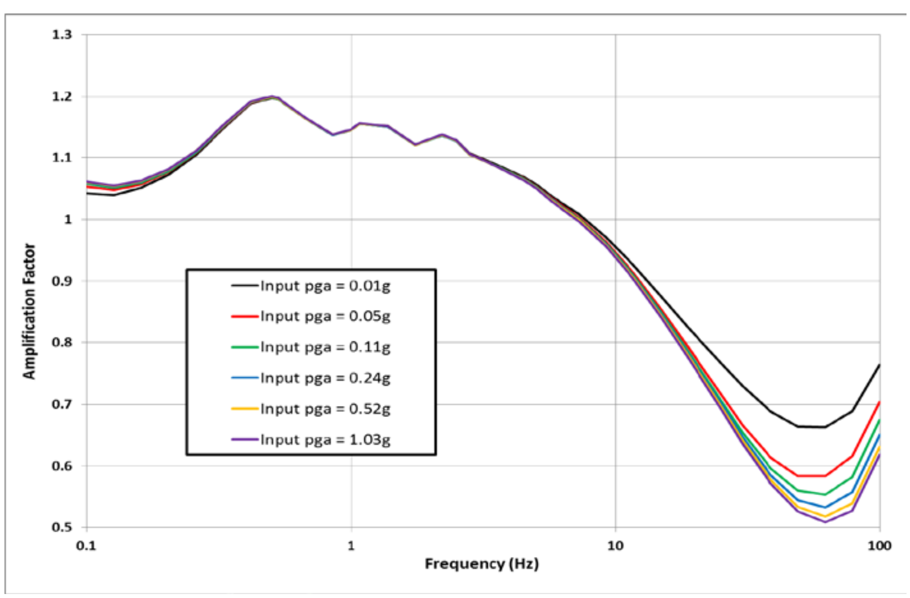

Figure 6Suite of Median Amplification Factors for Six Input Control Motions for Foundation Elevation + 7.6 m.

4. Derivation of foundation input response spectra

The FIRS is developed for ground motion with a MAFE of 1×10^{-4}. The FIRS is derived from hazard curves where the hazard curves are determined by convolving the hard-rock hazard curves from the PSHA with the site-specific amplification factors from the site response analysis [5][6].

Mathematically, the approach is given as:

$$\lambda(a_S) = \int P(AF > \frac{a_s}{a_R} | a_R)\, \lambda(a_R)\, \mathrm{d}a_R$$

in which

$\lambda(a_S)$	is the annual frequency of exceeding acceleration a_S at the control point of interest
AF	is amplification factor
$P(AF > \frac{a_s}{a_R} \| a_R)$	is the probability that AF is greater than $\frac{a_s}{a_R}$, given hard-rock acceleration a_R, and
$\lambda(a_R)$	is the annual frequency of occurrence of acceleration a_R at rock.

The frequency-dependent site AFs are defined as:

$$AF(f) = \frac{SA_{Control\ \ Point}\,(f)}{SA_{Rock}\,(f)}$$

in which *f* is response frequency, and $SA_{Control\ \ Point}\,(f)$ and $SA_{Rock}\,(f)$ are the 5%-damped SA at the foundation level control point and hard-rock, respectively.

$P\left(AF > \frac{a_s}{a_R} \middle| a_R\right)$is computed by taking *AF* as lognormally distributed and a function of a_R:

$$P\left(AF > \frac{a_s}{a_R} \middle| a_R\right) = 1 - \phi\left(\frac{\ln\left(\frac{a_s}{a_R}\right) - \mu_{\ln(AF)|a_R}}{\sigma_{\ln(AF)|a_R}}\right)$$

in which $\mu_{\ln(AF)|a_R}$ is the mean value of ln(*AF*) given rock SA = a_R, and $\sigma_{\ln(AF)|a_R}$ is the standard deviation of ln(*AF*) given rock SA = a_R. The term ϕ is the standard normal cumulative distribution function.

The resulting FIRS at the foundation elevation of +7.6 m isdisplayed on Figure 7.

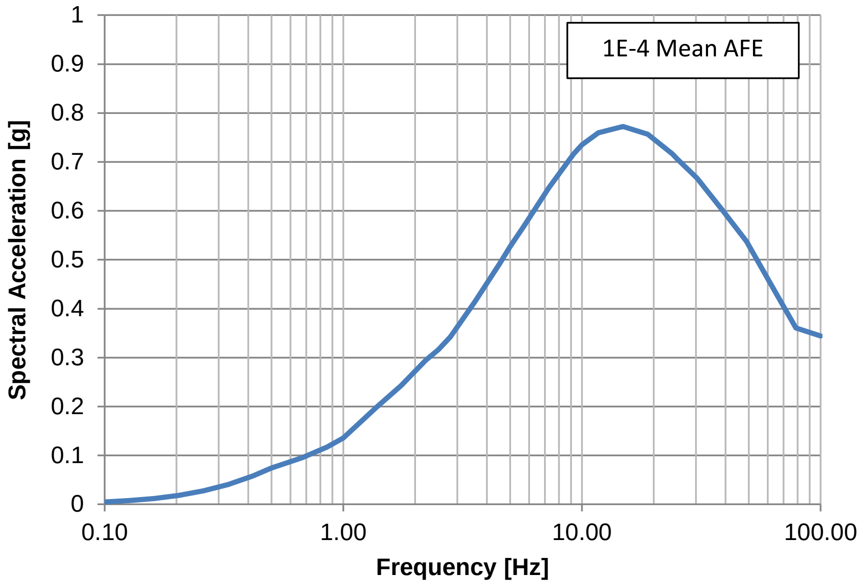

Figure 7 Foundation Input Response Spectra.

5. Discussion

As displayed on Figures5 and 6 the resulting amplifications factors at the top of competent rock are less than 1.0 for spectral frequencies above about 10 Hz. This indicates that the thick section of sedimentary rocks attenuates high-frequency ground motion relative to hard-rock site conditions. The magnitude of this attenuation is a function of ground motion input amplitude as shown in Figure 6. As the ground motion inputs increase the amount of damping within the sedimentary rock layers increases, resulting in more attenuation of high-frequency ground motion. Review of the site response computations indicates that strain-iterated damping values between about 0.5% and 1.5% within the sedimentary rock column are consistent with the FIRS. This finding is important for the PLGS Site given that the sedimentary rocks have a thickness that exceeds 610 m before reaching hard rock, allowing for damping of high-frequency ground motions. Small (~1%) strain iterated damping values result in large reductions in high frequency ground motion.

This finding can also be displayed by comparing the hard-rock UHRS with the FIRS. This comparison is shown in Figure 8 where the FIRS is less than the hard rock UHRS for higher spectral frequencies. If it had been assumed that the hard rock boundary for the site was at the top of the sedimentary rock, high-frequency ground motions would be over-estimated, possibly leading to an inaccurate assessment of facility seismic safety.

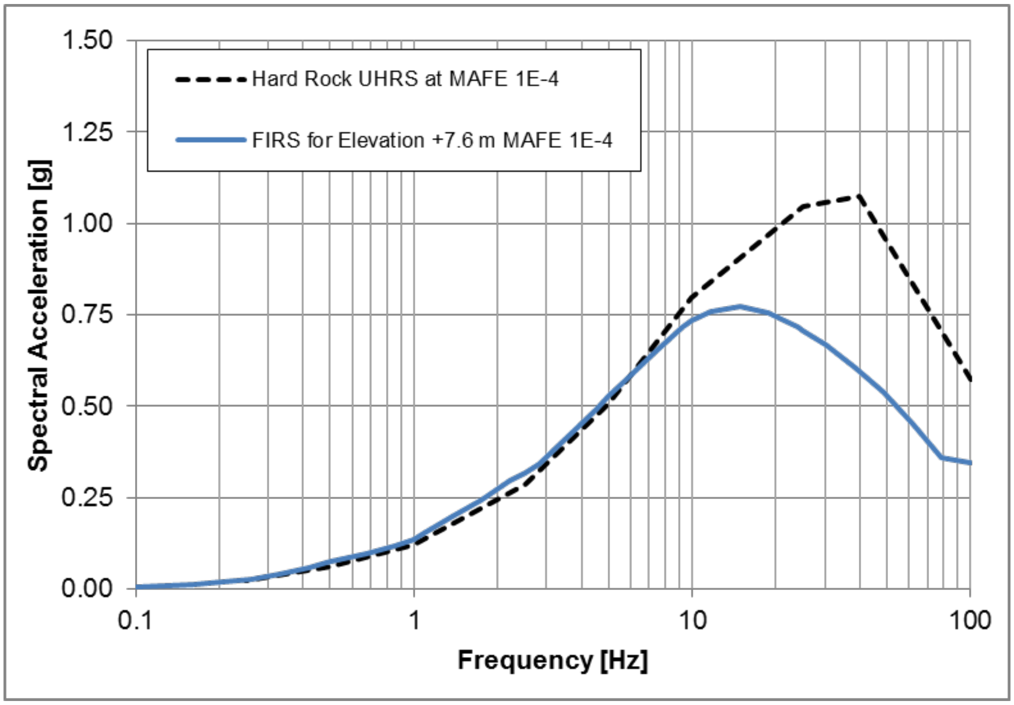

Figure 8 Comparison of the Hard-Rock Uniform Hazard Response Spectra with the Foundation Input Response Spectra.

The work completed is used to identify key technical issues for developing FIRS and recommended actions to address these issues. These are listed below.

Key Technical Issues for Developing Foundation Inputs Response Spectra and Recommended Actions to Address these Issues

1. *Develop Site Geotechnical Model*
    a. Compile relevant site geotechnical data;
    b. Evaluate soil and rock Vs and layer thickness, including uncertainty;
    c. Assess depth to hard-rock (Vs = 2800 m/s); and
    d. Assess dynamic properties, including uncertainty, for each soil and rock layer (e.g., shear modulus reduction versus shear strain and damping versus shear strain).

2. *Perform Site Response Analysis*
    a. Develop site response analysis logic tree which models epistemic uncertainty in site response inputs (alternative base-case site profiles, including Vs, thickness, and dynamic properties; alternative input motions);
    b. Develop set of randomized site profiles and dynamic properties (30 to 60) which models aleatory variability in site properties;

c. Develop site response input motions, including uncertainty (based on which combination of magnitude and distance controls the seismic hazard at the site); and
d. Execute site response computational software and develop mean and standard deviation site amplification factors for each branch of logic tree and each input motion.

3. *Integrate Site Response Analysis into PSHA*
    a. Execute PSHA for control points of interest by taking site amplification factors and mathematically combine these with hard-rock probabilistic ground motions and
    b. Document PSHA results.

4. *Develop Foundation Input Response Spectra*
    a. Define relevant foundation elevations (control points) for critical structures;
    b. Define approach being used for structural analysis (how any embedment is being modeled);
    c. Establish criteria to be used to define foundation input response spectra; and
    d. Derive foundation input response spectra at each relevant foundation elevation.

6. Conclusion

A PSHA was completed for the PLGS for generic hard rock site conditions. Based on reviewing available site geotechnical data a decision was made to develop a site-specific geotechnical model to be used as input to performing site response analysis. The PLGS site is represented by a 3 m thick shallow soil layer about 3.1 m of weathered rock, over a thick section of competent sedimentary rock that has a thickness exceeding 610 m. The top of competent sedimentary rockis the foundation elevation of interest for critical structures at the site. The geotechnical model includes an assessment of both aleatory variability and epistemic uncertainty with respect to defining appropriate site profiles, associated V_S values for geologic layers, and assignment of dynamic properties.

A site response analysis was completed for a wide range of input ground motions. The results from the site response analysis were integrated with the hard-rock PSHA results to quantify probabilistic ground motions at the site which are used to derive FIRS at the top of competent sedimentary rock. The amplifications factors from the site response analysis at the top of competent rock are less than 1.0 for spectral frequencies above about 10 Hz. This indicates that the thick section of sedimentary rocks attenuates high-frequency ground motion relative to hard rock site conditions. As the ground motion inputs increase the amount of damping within the sedimentary rock layers increase, resulting in more attenuation of high-frequency ground motion.

This finding is important for the PLGS Site given that the sedimentary rocks have a thickness that exceeds 610 m before reaching hard-rock, allowing for damping of high-frequency ground motions.This finding results in the FIRS being less than the hard rock UHRS for higher spectral frequencies. If it had been assumed that the hard rock boundary for the site was at the top of the sedimentary rock, high-frequency ground motions would be over-estimated, possibly leading to an inaccurate assessment of facility seismic safety.

7. References

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