Appendix B: Development of Horizontal Site-Specific Amplification Factors

1.0 Introduction

It has long been recognized that the amplitude and frequency content of ground motions at a site are strongly influenced by the characteristics of the near-surface materials. For most sites, however, the properties of the near-surface materials and the parameters that control the dynamic response are not known with certainty. The uncertainty in these parameters needs to be accounted for when developing site specific hazard curves. Ultimately, the goal or objective of the site response analysis is to produce site-specific hazard curves and response spectra which reflect the desired exceedance frequencies, that is, preserve the reference site annual exceedance frequency (AEF) thereby maintaining hazard consistency for results produced at any elevation in the profile. However, the uncertainty in characterizing the soil profile and dynamic properties of the near-surface materials presents a challenge to preserving hazard levels for sites that differ from some specified reference condition.

Previously, the state of practice in calculating a site-specific ground motion has been to calculate probabilistic reference rock ground motions and then multiply them by deterministic site-amplification factors [17, 18]. However, as stated above, there is uncertainty in the layering, spatial distribution and dynamic properties of near-surface materials. This leads to uncertainty in the estimation of site amplification functions. To alleviate this problem it is necessary to calculate the effects of uncertainty on the estimate of the site-amplification functions and use the resulting site-amplification distribution within a probabilistic methodology [24, 16, 8].

The first step in developing site-specific seismic hazard curves and response spectra consists of performing a Probabilistic Seismic Hazard Analysis (PSHA) that reflects an outcropping reference site condition. The reference site condition is usually hard or firm rock and is consistent with assumptions made in the development of the most recent ground motion prediction equations. For central and eastern North America (CENA) this represents a site with a theoretical shear wave velocity over the top 1 km of the crust of 2.83 km/sec with a specified shallow crustal damping parameter [18]. The shear-wave velocity is based on the empirical Mid-continent compressional-wave velocity model of Pakiser and Mooney (1989) [28], taken by EPRI (1993) to represent the CENA, and an assumed Poisson ratio. For western U.S. (WUS) sites an appropriate reference condition should be selected that is well-constrained by observational data in the ground motion prediction equations (GMPEs). Site-specific amplification functions are then developed relative to the reference site condition.

After completing PSHA calculations for reference rock site conditions, the development of hazard consistent, site specific horizontal seismic hazard results may be considered as involving two independent analyses. The first is the development of frequency and amplitude dependent relative amplification factors (for 5% damped response spectra, Sa) between the site of interest and the reference site $(Sa_{SITE}\ (f)/Sa_{REFERENCE}\ (f))$ that accommodates potential linear or nonlinear site response. Currently the state-of-practice approach involves vertically propagating shear-waves and approximations using equivalent-linear analysis with either a time domain method (e.g. SHAKE) or a more computationally efficient frequency domain random vibration theory (RVT) method [32].

Subsequent to the development of the amplification factors, site-specific motions are computed by scaling the reference site motions with the transfer functions. As suggested above, probabilistic methods have been developed [24, 8] that accurately preserve the reference site hazard level and result in full site-specific hazard curves. These fully probabilistic approaches represent a viable and preferred mechanism to properly incorporate the site-specific aleatory (randomness) and epistemic (uncertainty) variabilities of the dynamic properties and achieve desired hazard levels and performance goals. The following sections describe the specific steps in the development of the site-specific amplification functions.

B-2.0 Description of Sites Requiring Response Analyses and Basis for Alternative Models

The level of detail and scope of the geological and geotechnical investigations conducted during the licensing of the currently operating NPPs was consistent with the state of the practice at the time of the plants design and licensing. However, the state of the practice in earthquake engineering has evolved over the last several decades. As a result, some of the detailed information required to perform modern site response analyses (consistent with the request in the March 12, 2012 50.54(f) letter [27]) are lacking for some of the older plants. This lack of information results in increased levels of uncertainty in the site response analyses. The following sections describe how this uncertainty will be accommodated in the site response analyses. The amount, quality and applicability of the available data will determine the analysis procedures.

The information available to develop estimates of site properties and characteristics will be primarily based on readily available sources (FSAR and other regional data) for most locations. However, for sites with recent COL and ESP submittals, the co-located operating plants would be expected to utilize any applicable information developed in the ESP and COL site characterization process to the maximum extent practicable.

Site response analyses will be required for sites in the central and eastern U.S. (CEUS) (i.e., those sites located east of the Rocky Mountains) when available information suggests surficial materials will impact design motions at frequencies below about 50 Hz. The conservative criteria used to determine if site-specific amplification functions are required is more than 7.5m (25ft) of material with an average shear-wave velocity less than 2286m/s (7,500ft/s) over hard rock. Site-specific response analyses will be required for all sites in the western U.S. (sites west of the Rocky Mountain front).

Based on the need to determine if a facility requires detailed site response analyses (the combined stiffness and velocity criteria described above), the first step in the process is the compilation and evaluation of site geotechnical and geophysical characteristics. This information should be summarized consistent with the documentation described in Section 4 of the main report. The available site-specific information will be highly variable in terms quantity and applicability and this range in available site-characterization data and information will necessitate several different approaches to developing site amplification functions. The different approaches are described more fully below.

B-2.1 Background on the Treatment of Uncertainties

There are two different types of uncertainty in the development of site-specific amplification functions (AF(*f*)). First, at any given site, at the spatial dimensions of typical nuclear facilities (100-200m (~325-650ft) scale dimensions) there is expected to be some variability in important site response parameters such as shear-wave velocities, non-linear dynamic material properties at any depth across the footprint of the facility, and the overall thickness of soil/soft rock above firm rock site conditions. It is important to attempt to capture this uncertainty in the final AF(*f*) estimates. This is treated as an aleatory (randomness) type of variability. Current practice represents this variability by developing a candidate shear-wave velocity profile, depth and overall thickness of soil/soft rock and associated non-linear dynamic material properties (shear modulus reduction and damping curves). This is referred to as a “base case” model. Subsequently, potential variations in shear-wave velocity and layer thickness are represented by correlated random perturbations to the base-case values. This is frequently referred to as a randomization process. A sufficient number of realizations (30 or more) are used to develop statistical estimates (log median and log standard deviation) of the amplification functions.

The second type of uncertainty is epistemic or lack of knowledge uncertainty. This represents the uncertainty in the development of the base-case models for site profile, dynamic properties, and seismological parameters. For well-characterized sites with abundant high-quality data this uncertainty would be reduced, possibly eliminating the need to vary some of the site parameters such as the site profile. This epistemic uncertainty would increase with decreasing confidence in the available data and information. This uncertainty is evaluated through the development of alternative base-case models. The approach applied for the development of alternative base-case models (epistemic uncertainty) is discussed in more detail in the following sections.

The following information is required to perform the site-specific response analyses: site shear-wave velocity profiles, non-linear dynamic material properties, estimates of low-strain site damping (parameterized through the parameter, kappa), and input or control motions (including relevant seismological parameters). These various factors are discussed individually in the following sections.

B-3.0 Development of Base-Case Profiles and Assessment of Epistemic Uncertainty in Site Profiles and Dynamic Material Properties

Epistemic uncertainty in depth to hard rock site conditions and dynamic material properties, which includes shear-wave velocity profiles, site material damping at low strain (parameterized through kappa), and modulus reduction and hysteretic damping curves, should be accommodated through the development of alternative mean cases. The specific methodology utilized to develop the alternative cases will depend on the amount of information available at a given site. Conceptually in this context, for poorly characterized sites with few if any measured dynamic material properties, multiple cases should be developed with broad ranges of epistemic uncertainty applied in the development of the parameters of the alternative cases. For sites that have more complete site characterization information available, smaller epistemic uncertainty factors can be employed in the development of the alternative cases.

For those cases where limited or no at-site information is available, a minimum of three profile estimates combined with three kappa estimates and two sets of modulus reduction and hysteretic damping curves should be developed. If significant uncertainty exists in the thickness of soil above firm or hard rock conditions, this thickness should be treated as an epistemic uncertainty. The three cases for shear-wave velocity profiles and kappa are referred to hereafter as base-case, and upper-range and lower-range models. A general set of guidelines should be employed to develop these cases for dynamic material properties and associated weights and is described more fully below. The general computational framework for developing the mean site amplification functions and associated standard deviations is illustrated in Figure B-1.

B-3.1 Development Process for Base-Case Shear-Wave Velocity Profiles

In order to predict site response as accurately as possible, and ultimately prevent error from propagating into other engineering calculations, it is important to define a detailed shear wave velocity (V_S) profile that represents the known or inferred in-situ velocity structure as realistically as possible. The following discussion describes the development of the mean or base-case V_S profile. The alternative (upper-range and lower-range) models are derived from the base-case model utilizing an information-informed epistemic factor. The development of the upper-range and lower-range models is discussed in Section B-3.2 after the base-case development.

For sites with sparse or very limited information regarding dynamic material properties (e.g., a measured shear-wave velocity profile was unavailable), typically an estimate based on limited surveys (e.g., compressional-wave refraction) is available over some shallow, limited depth range. For such cases, as well as to provide a basis for extrapolating profiles specified over shallow depths to hard rock basement material, a suite of profile velocity templates has been developed, parameterized with V_{S30} (time averaged shear-wave velocity over upper 30m of the profile) ranging from 190m/s to 2,032m/s (620ft/s to 6670ft/s). The suite of profile templates is shown in Figure B-2 to a depth of 305m (1,000ft). The templates are from [40] supplemented for the current application with profiles for V_{S30} values of 190m/s, 1,364m/s, and 2,032m/s. The latter two profiles were added to accommodate cases where residual soils (saprolite) are present and overly hard rock. For both soil and soft rock sites, the profile with the closest velocities over the appropriate depth range should be adopted from the suite of profile templates and adjusted by increasing or decreasing the template velocities or, in some cases, stripping off material to match the velocity estimates provided.

For sites with no direct velocity measurements of any type and limited geological information it may be necessary to utilize a proxy to estimate near-surface shear-wave velocity. There are four currently employed V_{S30} proxies, surficial geology [42, 41], Geomatrix site category [14], topographic slope [39], and terrain [43]. Analysis of these proxies suggests a relatively constant variability of measured V_{S30} about the predicted value amongst the various methods [34]. If proxy methods are used to infer near-surface shear-wave velocities, additional levels of epistemic uncertainty may need to be considered (see Section B-3.2).

For intermediate cases, such as when only the upper portion of a deep soil profile is constrained with measured velocities, the V_S template profile with velocities closest to the observed velocity at the appropriate depth should be identified. This template can then be used to provide a rational basis to extrapolate the profile to the required depth.

For soft or firm rock sites, which are often composed of Cenozoic or Paleozoic sedimentary rocks such as shales, sandstones, siltstones, or similar rock types, a constant shear-wave velocity gradient of 0.5m/s/m (0.5ft/s/ft) should be used as a template and used to estimate the velocities over the appropriate depth range. This gradient is based on deep measurements in similar rock types in Japan [20]. The 0.5m/s/m velocity gradient is also consistent with measurements in sedimentary rocks of similar type at the Varian well in Parkfield, California [22]. It is recognized that the soil or firm rock gradients in the original profiles are primarily driven by confining pressure and may not be strictly correct for each adjusted profile template at each site. However, any shortcoming in the assumed gradient is not expected to be significant as the range in multiple base-case profiles accommodates the effects of epistemic uncertainty in the profile gradient on the resulting amplification functions.

For all sites where limited data exists, or only exists for very shallow depths, it is necessary to fully evaluate and integrate all existing geological information into the development of the base-case profile. For sites with soil or soft rock at the surface and much stiffer materials at relatively shallow depths (less than approximately 60m (200ft)) the potential for strong resonance in the frequency range of engineering interest exists. All relevant geological information should be assessed to ensure this condition is identified.

An example is provided in Figure B-3 to schematically illustrate how a combination of geological information and geophysical measurements may be used to develop a base-case profile. The data available at this hypothetical site consists of shallow shear-wave velocity measurements (a single S-wave refraction profile) over only about the upper ~30m (100ft) of the profile with a V_{S30} of approximately 450m/s. There are also geologic profiles and regional data available in the FSAR that indicate firm rock is present at a depth about 45m (150ft) beneath the site. A shear-wave velocity of approximately1525m/s (5000ft/s) is inferred for the firm rock based on velocity measurements on comparable material elsewhere. Regional data indicates the firm, sedimentary rocks extend to a depth of at least 1 km before crystalline basement rock is encountered. The information is combined in the following manner to construct a base case profile. The closest template profile to the 450m/s V_{S30} estimate is the 400m/s profile. The velocities in the 400m/s template are scaled by a factor of 1.125 (450/400) to adjust to the desired V_{S30} value. At the 45m (150ft) depth, a velocity discontinuity is inserted with a velocity of 1525m/s (5000ft/s). Below this depth the firm rock gradient model of 0.5m/s/m is used to estimate velocities. This gradient is extended to a sufficient depth such that 2830m/sec is reached or the depth is greater than the criteria for no influence on response for frequencies greater than 0.5Hz. The uncertainty in the depth to the soil-firm rock interface is incorporated in the treatment of epistemic uncertainty as discussed below.

B-3.2 Capturing Epistemic Uncertainty in Velocity Profiles

There are basically two approaches for constructing shear wave velocity profiles, either through inference from geotechnical/geologic information or through the use of geophysical measurements. Each approach will inherently have some level of uncertainty associated with its ability to accurately represent the in-situ velocity structure. The level of uncertainty will depend on the amount of information available along with how well the information is correlated with shear-wave velocity. By adopting the general mean based approach outlined in Section B-3.1, a level of uncertainty can be assigned to a template velocity profile, commensurate with the available information, in order to account for the epistemic uncertainty associated with the in-situ velocity structure.

For sites where geophysical information such as very limited shear-wave velocity data exists or compressional-wave velocities are used to infer shear-wave velocities, the estimate for uncertainty in shear-wave velocity is to be taken as:

$$\sigma_{\mu} \ln = 0.35$$

This value is similar to a Coefficient of Variation (COV) of 0.25 which is consistent with Toro (1997) [37] for observed spatial variability over a structural footprint of several hundred meters. The profile epistemic uncertainty factor of 0.35 (σ_μ ln) is to be applied throughout the profile and is based on the estimates of epistemic uncertainty in V_{S30} developed for stiff profiles [14]. The logarithmic factor assumes shear-wave velocities are lognormally distributed and was originally developed to characterize the epistemic uncertainty in measured V_{S30} at ground motion recording sites where measurements were taken within 300m (900ft) from the actual site. The uncertainty accommodates spatial variability over maximum distances of 300m, and is adopted here as a reasonable and realistic uncertainty assessment reflecting a combination of: (1) few velocity measurements over varying depth ranges, (2) shear-wave velocities inferred from compressional-wave measurements, and (3) the spatial variability associated with observed velocities. While velocities are undoubtedly correlated with depth beyond 30m, which forms the basis for the use of V_{S30} as an indicator of relative site amplification over a wide frequency range, clearly the correlation is neither perfect nor remains high over unlimited depths [11]. An example of the resulting mean ± σ_μ ln shear-wave profiles for the 760m/s template is shown in Figure B-4.

For sites where site-specific velocity measurements are particularly sparse (e.g., based on inference from geotechnical/geologic information rather than geophysical measurements) a conservative estimate of the uncertainty associated with the template velocity is to be taken as:

$$\sigma_{\mu} \ln = 0.5$$

For sites where multiple, detailed shear-wave velocity profiles are available, the level of uncertainty may be taken as zero if justified. For sites with an intermediate level of information available, such as a single shear-wave velocity profile of high-quality, a reduced σ_μ (ln) value may be applied if justified.

B-3.2.1 Epistemic Uncertainty in Final Hazard Calculations

It is necessary to represent the epistemic uncertainty in the distribution of potential shear-wave velocity profiles (mean base-case and a σ_μ (ln) of 0.35, for example) in the final site-specific hazard results. Practicality requires this be accomplished with the minimum number of cases. The recommended approach for this application is to utilize three cases, the mean base-case and upper range and lower range base-cases with relative weights applied. An accurate three point approximation of a normal distribution which preserves the mean utilizes the 50th-percentile (median) and 10th and 90th percentiles, with relative weights of 0.30 for the 10th and 90th percentiles and 0.40 for the median applied [23]. These values are summarized in Table B-1. The 10th and 90th fractiles correspond to a profile scale factor of 1.28 σ_μ. When $\sigma_\mu \ln = 0.35$ the 10th or 90th percentiles are obtained by subtracting or adding 0.45 in natural log units to the shear wave velocity. For $\sigma_\mu \ln = 0.5$, a value of 0.64 is subtracted or added to the natural log of the shear wave velocity for the 10th and 90th percentile values. This is equivalent to an absolute factors of 1.57 or 1.90 applied to the mean base-case profile for $\sigma_\mu \ln = 0.35$ or $\sigma_\mu \ln = 0.5$, respectively. Figure B-5 illustrates application of these two factors applied to the 760 m/s (1525 ft/s) V_{S30} template. Figure B-6 illustrates the same type of curves for the firm rock template derived using the empirical gradient of [20]. For some individual sites it may be necessary to deviate from these standard weights if application of the standard factors results in velocities that are not deemed credible.

Figure B-7 illustrates the development of Upper Range and Lower Range profiles to accommodate epistemic uncertainties for the hypothetical example shown in Figure B-3. A σ_μ ln = 0.35 has been used to develop the 10th and 90th-percentile curves in the upper portion of the profile where sparse Vs measurements were available. A σ_μ ln = 0.50 was applied to the lower portion of the profile where the Vs of the Base Case was inferred from geological information. The 90th-percentile curve was capped at a value equal to the 2830m/s Vs value assumed for the hard rock basement. This example illustrates the broad range of velocities encompassed by the Upper Range, Mean, and Lower Range profiles for sites lacking in good data.

For sites where the depth to firm rock conditions is poorly constrained, that depth should be treated as a separate epistemic uncertainty as illustrated in Figure B-1.

B-3.3 Nonlinear Dynamic Material Properties

The potential nonlinear response of near-surface materials to input ground motions is an important element of the site that needs to be characterized in a proper site response analysis. To characterize the epistemic uncertainty in nonlinear dynamic material properties for both soil, and soft/firm rock sites, the use of two sets of modulus reduction and hysteretic damping curves is suggested.

For soils, the two sets of proposed curves are the EPRI (1993) and Peninsular Range [32, 40] results. The two sets of generic curves are appropriate for cohesionless soils comprised of sands, gravels, silts, and low plasticity clays. The EPRI (1993) curves, illustrated in Figure B-8, were developed for application to CENA sites and display a moderate degree of nonlinearity. The EPRI (1993) curves are depth (confining pressure) dependent as shown in Figure B-8. The Peninsular Range curves reflect more linear cyclic shear strain dependencies than the EPRI (1993) curves [40] and were developed by modeling recorded motions as well as empirical soil amplification in the Abrahamson and Silva WNA (Western North America) GMPE [32, 2]. The Peninsular Range curves reflect a subset of the EPRI (1993) soil curves with the 51 to 120 ft (15 to 37 m) EPRI (1993) curves applied to the 0 to 50 ft (0 to 15 m) depth range and the EPRI (1993) 501 to 1,000 ft (153 to 305 m) curve applied to the 51 to 500 ft (15 to 152 m) depth range, below which linear behavior is assumed.

The two sets of soil curves are considered to reflect a realistic range in nonlinear dynamic material properties for cohesionless soils. The use of these two sets of cohesionless soil curves implicitly assumes the soils considered do not have response dominated by soft and highly plastic clays or coarse gravels or cobbles. The presence of relatively thin layers of hard plastic clays are considered to be accommodated with the more linear Peninsular Range curves while the presence of gravely layers are accommodated with the more nonlinear EPRI (1993) soil curves, all on a generic basis. The potential impact on the amplification functions of the use of these two sets of nonlinear dynamic property curves was evaluated and is shown in Figure B-9. The results indicate that above 1 Hz the difference can be significant and the resulting epistemic uncertainty needs to be included in the development of the final amplification functions.

The two sets of soil curves are given equal weights (Table B-1 and Figure B-1) and are considered to represent a reasonable accommodation of epistemic uncertainty in nonlinear dynamic material properties for the generic types of soils found at most in CEUS sites which include:

1. Glaciated regions which consist of both very shallow Holocene soils overlying tills as well as deep soils such as the Illinois and Michigan basins, all with underlying either firm rock (e.g., Devonian Shales) and then hard basement rock or simply hard basement rock outside the region of Devonian Shales,
2. Mississippi embayment soils including loess,
3. Atlantic and Gulf coastal plain soils which may include stiff hard clays such as the Cooper Marl,
4. Residual soils (saprolite) overlying hard metamorphic rock along the Piedmont and Blue Ridge physiographic regions.

For soft or firm rock site conditions, taken generally as Paleocene sedimentary rocks, such as shale, sandstones, or siltstones, two alternative expressions of nonlinear dynamic material behavior are proposed: the EPRI “rock curves” (Figure B-10) and linear response. The EPRI rock curves were developed during the EPRI (1993) project by assuming firm rock, with nominal shear-wave velocities in the range of about 914m/s to 2134m/s (3,000ft/s to about 7,000ft/s, about 5,000ft/s on average), behaves in a manner similar to gravels [18] being significantly more nonlinear with higher damping than more fine grained sandy soils. The rock curves were not included in the EPRI report 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 CEUS nuclear power plants. With the stiffness typically associated with consolidated sedimentary rocks, cyclic shear strains remain relatively low compared to soils. Significant nonlinearity in the soft-to-firm rock materials is largely confined to the very high loading levels (e.g. ≥ 0.75g).

As an alternative to the EPRI rock curves, linear response should be assumed. Implicit in this model is purely elastic response accompanied with damping that remains constant with cyclic shear strain at input loading levels up to and beyond 1.5g (reference site). Similar to the two sets of curves for soils, equal weights were given to the two sets of nonlinear properties for soft/firm rock sites as summarized in Table B-1.

B-3.4 Densities

Because relative (soil surface/reference site) densities play a minor role in site-specific amplification, a simple model based on the shear-wave velocity of the mean base-case profile is proposed for those sites where a profile density is not available. This model relating estimated shear-wave velocity and density is summarized in Table B-2.

Due to the square root dependence of amplification on the relative density, a 20% change in soil density results in only a 10% change in amplification and only for frequencies at and above the column resonant frequency. As a result only an approximate estimate of profile density is considered necessary with the densities of the mean base-case profile held constant for the upper and lower range base-case profiles. This approach provides a means of accommodating epistemic uncertainty in both density as well as shear-wave velocity (Section B-3.1) in the suite of analyses over velocity uncertainty.

B-4.0 Representation of Aleatory Variability in Site Response

To accommodate the aleatory variability in dynamic material properties that is expected to occur across each site (at the scale of the footprint of a typical nuclear facility), shear-wave velocity profiles as well as G/G_{MAX} and hysteretic damping curves should be randomized. The aleatory variability about each base-case set of dynamic material properties should be developed by randomizing (a minimum of thirty realizations) shear-wave velocities, layer thickness, depth to reference rock, and modulus reduction and hysteretic damping curves. For all the sites considered, where soil and firm rock extended to depths exceeding 150m (500ft), linear response can be assumed in the deep portions of the profile [32, 33, 35, 36].

B-4.1 Randomization of Shear-Wave Velocities

The velocity randomization procedure makes use of random field models [37] to generate V_S profiles. These models assume that the shear-wave velocity at any depth is lognormally distributed and correlated between adjacent layers. The layer thickness model also replicates the overall observed decrease in velocity fluctuations as depth increases. This realistic trend is accommodated through increasing layer thicknesses with increasing depth. The statistical parameters required for generation of the velocity profiles are the standard deviation of the natural log of the shear-wave velocity (σ_{lnVs}) and the interlayer correlation (ρ_{IL}). For the footprint correlation model, the empirical σ_{lnVs} is about 0.25 and decreases with depth to about 0.15 below about 15m (50ft) [32]. To prevent unrealistic velocity realizations, a bound of $\pm 2\sigma_{lnVs}$ should be imposed throughout the profile. In addition, randomly generated velocity should be limited to 2.83km/s (9200ft/s). All generated velocity profiles should be compared to available site-specific data as a check to ensure that unrealistic velocity profiles are removed (and replaced) from the set of velocity profiles used to develop site response amplification functions. This process should be documented as part of the site response analysis.

B-4.2 Aleatory Variability of Dynamic Material Properties

The aleatory variability about each base-case set of dynamic material properties (EPRI depth dependent vs. Peninsular for example) will be developed by randomizing modulus reduction and hysteretic damping curves for each of the thirty realizations. A log normal distribution may be assumed with a σ_{ln} of 0.15 and 0.30 at a cyclic shear strain of 3×10^{-2}% for modulus reduction and hysteretic damping respectively [32]. Upper and lower bounds of +/-2σ should be applied. The truncation is necessary to prevent modulus reduction or damping models that are not physically realizable. The distribution is based on an analysis of variance of measured G/G_{MAX} and hysteretic damping curves and is considered appropriate for applications to generic (material type specific) nonlinear properties [32]. The random curves are generated by sampling the transformed normal distribution with a σ_{ln} of 0.15 and 0.30 as appropriate, computing the change in normalized modulus reduction or percent damping at 3×10^{-2}% cyclic shear strain, and applying this factor at all strains. The random perturbation factor is then reduced or tapered near the ends of the strain range to preserve the general shape of the base-case curves [18, 32]. Damping should be limited to a maximum value of 15% in this application.

B-5.0 Development of Input Motions

The ground motion used as input to site response analyses is commonly referred to as the “control motion.” This can be reflected in time histories matched or scaled to a response spectrum or, in the case of Random Vibration Theory, a power spectral density (PSD). Because of the very large number of cases that will need to evaluated to capture the range of epistemic uncertainty and aleatory variability in this application (See Figure B-1 and Table B-3) the following discussion will assume that the much more efficient random vibration theory (RVT) approach to performing site response analyses will be utilized as opposed to a time series (TS) based technique. Recent studies [29] have confirmed that the two approaches yield similar results. The following sections describe the model used in the development of the control motions and the parameters of that model that require an assessment of uncertainty.

B-5.1 Simple Seismological Model to Develop Control Motions

The methodology suggested for developing the input or control motions relies on a widely used, simple seismological model to represent earthquake source, propagation path and site characteristics ([10] and references therein). The ground motions recorded at a given site from an earthquake can be represented in the frequency domain as:

$$Y(M_0, R, f) = E(M_0, f) \cdot P(R, f) \cdot G(f).$$

Where $Y(M_0,R,f)$ is the recorded ground motion Fourier amplitude spectrum, $E(M_0,f)$ is the Brune point-source seismic spectrum, $P(R,f)$ represents the propagation path effects, and $G(f)$ represents the modification due to site effects. In this equation M_0 is the seismic moment of the earthquake, R is the distance from the source to the site, and f is frequency. The seismic moment and the earthquake magnitude are related through the definition of the moment magnitude, M [21]:

$$M = 2/3 Log_{10} M_0 - 10.7.$$

The $P(R, f)$ term accounts for path effects, geometrical spreading and frequency dependent deep crustal damping and can be expressed as:

$$P(R,f) = S(R)\ exp((-\pi f R)/(Q(f)V_S)).$$

Where $S(R)$ is the geometrical spreading function, $Q(f)$ is the seismic quality factor, and V_S is the shear-wave velocity in the upper crust.

The $G(f)$ term accounts for upper crustal amplification and frequency-independent shallow crustal damping:

$$G(f) = A(f) \cdot D(f).$$

Where $A(f)$ is the amplification function relative to source depth velocity conditions and $D(f)$ represents the frequency-dependent damping term ($D(f) = exp(-\pi\kappa_0 f)$). The $A(f)$ term may be calculated using simplified square-root impedence methods ($A(f) = (Zsource/Zavg)^{0.5}$), where Z is the product of density and velocity (ρV_S), for example) or using more detailed full resonant techniques.

Kappa (κ_0) is an upper crustal site ground motion attenuation parameter that accounts for the overall damping in the basement rock immediately beneath a site. The properties and behavior of the upper few hundreds of meters of the crust has been shown to produce as much as 50% or more of the total attenuation ($Q(f)$) of the high-frequency portion of the ground motion spectrum [1, 4]. The value of kappa influences the shape of the ground motion spectrum observed at a given site. High values of kappa result in enhanced attenuation of the high-frequency portion of the spectrum.

The factors in the simple seismological model that affect the spectral shape of the input motions are kappa, magnitude, attenuation model and source model. These factors are discussed below. An example of the potential effect of these parameters on the spectral shape of the input ground motions (Fourier amplitude spectra and 5%-damped response spectra) is shown in Figure B-11 (input parameters are summarized in Table B-7).

B-5.1.1 Magnitude

Conditional on reference site peak acceleration, amplification factors depend, to some extent, upon control motion spectral shape due to the potential nonlinear response of the near-surface materials. For the same reference site peak acceleration, amplification factors developed with control motions reflecting **M** 5.5 will differ somewhat with those developed using a larger or smaller magnitude, for example.

Figure B-12 shows amplification factors developed for the 400m/s V_{S30} template profile (Figure B-2) using the single-corner source model for magnitudes **M** 5.5, 6.5, and 7.5. For this sensitivity analysis the more nonlinear EPRI G/G_{max} and hysteretic damping curves (Figure B-8) were used. The dependence on control motion spectral shape is observed to decrease with degree of nonlinearity becoming independent for linear analyses. As Figure B-12 illustrates, the largest amplification reflects the lowest magnitude (**M** 5.5). Over the frequency range of about 5 to 10 Hz, and the ground motion amplitude range of most engineering interest (between 0.1 g and 0.75 g), the difference in the derived amplification functions between the magnitudes is minor. The largest difference in amplification is about 20% and at the highest loading levels (≥ 0.75g). The largest difference in amplification is between **M** 5.5 and **M** 6.5 with little difference (< 10%) between **M** 6.5 and **M** 7.5. Given the most current source characterization model in CENA [26] and the distribution of existing NPP sites, the dominant contribution for the annual exceedance frequencies (AEF) of 10^{-4} and below are from magnitudes in the range of about **M** 6 to **M** 7+. Given these factors, and the large number of analyses required (Table B-3) a single magnitude (**M** 6.5) is proposed for development of the control motions. This is felt to adequately characterize the amplification, with tacit acceptance of slight conservatism for magnitude contributions above about **M** 7.

B-5.1.2 Attenuation (*Q(f)*) Model

As illustrated in Figure B-11, major differences in the assumed crustal attenuation model will influence the spectral shape of the control motions. However, within a given tectonic region, the CENA or the WUS for example, changes in the crustal attenuation model do not contribute significantly to changes in the derived amplification functions. Appropriate, widely referenced crustal attenuation models are proposed for the CEUS and WUS sites (Table B-4).

B-5.1.3 Kappa

In the context of this discussion, the kappa referred to here is the profile damping contributed by both intrinsic hysteretic damping as well as scattering due to wave propagation in heterogeneous material. Both the hysteretic intrinsic damping and the scattering damping within the near-surface profile and apart from the crust are assumed to be frequency independent, at least over the frequency range of interest for Fourier amplitude spectra (0.33 to about 25.0 Hz). As a result, the kappa estimates reflect values that would be expected to be measured based on empirical analyses of wavefields propagating throughout the profiles at low loading levels and reflect the effective damping or "effective" Q_s within the profile [12]. Changes in kappa can exert a strong impact on derived control motion spectra (Figure B-11) and as a result are an important part of the input model for development of control motions. Hence, similar to the treatment of uncertainties in shear-wave velocity profiles, multiple base-cases (mean and upper and lower ranges) may be developed for kappa.

B-5.1.3.1 Development of Base Case Kappa Models

Mean base-case kappa values were developed differently for soil and firm rock sites.

Rock Sites: For rock sites with at least 3,000ft (1000m) of firm sedimentary rock (V_{S30} > 500 m/s) overlying hard rock, the kappa-V_{S100} (average shear-wave velocity over the upper 100ft of the profile) relationship of

$$\log(\kappa) = 2.2189 - 1.0930*\log(V_{S100}),$$

Where V_{S100} is in ft/s, is proposed to assign a mean base-case estimate for kappa [36, 38]. The requirement of a 3000ft (1,000m) thickness of firm materials reflects the assumption that the majority of damping contributing to kappa occurs over the upper km of the crust with a minor contribution from deeper materials (e.g., 0.006s for hard rock basement material). As an example, for a firm sedimentary rock with a shear-wave velocity of 5,000ft/s (1525m/s), this relationship produces a kappa estimate of about 0.02s. The assumption that is typically used implies a kappa of 0.014s is contributed by the sedimentary rock column and 0.006s from the underlying reference rock (Table B-4), and reflects an average Q_s of about 40 over the 3,000ft depth interval. The Q_s value of 40 for sedimentary rocks is consistent with the average value of 37 observed (measured) over the depth range of 0m to 298m in Tertiary claystones, siltstones, sandstones, and conglomerates at a deep borehole in Parkfield, California [22].

For soft/firm rock cases with low estimated velocity values, an upper bound kappa value of 0.04s should be imposed. The maximum kappa value of about 0.04s reflects a conservative average for soft rock conditions [31, 32].

For cases where the thickness of firm rock is less than about 3,000ft (1000m) and the relationship cited above is not applicable, the kappa contributed by the firm rock profile can be computed assuming a Q_s of 40 plus the contribution of the reference rock profile of 0.006s (Table B-4). For the three base-case firm rock template profiles shown in Figure B-6, the total kappa values assuming a Q_s of 40 are 0.019s, 0.025s, and 0.015s for the mean, lower range, and upper range base-cases respectively.

Soil Sites:

For soil sites (either in the WUS or CEUS) with depths exceeding 1000m (3,000ft) to hard rock, a mean base-case kappa of 0.04s should be assumed based upon observed average values for deep soil sites and low loading levels. The mean base-case kappa of 0.04s adopted for deep firm soils is lower than the value of approximately 0.06s based on recordings at alluvium sites located in Southern California [4, 32]. For soil sites, due to nonlinear effects, low strain kappa may be overestimated depending upon loading level and the nonlinear dynamic material properties. To avoid potential bias in the deep, firm soil, low strain kappa, the value of 0.04s is based on inversions of the Abrahamson and Silva [3] soil site GMPE [32]. In that inversion, a range of rock site loading levels was used with the soil value of 0.04s based upon a rock site peak acceleration of 5% g or less, clearly a low strain estimate. The deep soil mean base-case kappa of 0.04s is adopted for both the upper and lower range profiles with the assumption that the suite of profiles reflect deep firm soils. The assumed kappa of 0.04s for deep (≥ 1000m (3,000ft)) firm soils in the CEUS is somewhat less than the 0.054s inferred by Campbell [12] based on Cramer et al. [15] analyses for effective Q_s within the 960m deep sedimentary column in the Mississippi embayment near Memphis, Tennessee. The deep firm soil kappa of 0.04s is in fair agreement with 0.052s found by Chapman, et al. [13] for the 775m thick sedimentary column near Summerville, South Carolina.

In summary, for deep firm soil sites (≥ 1000m (3,000ft) to basement rock) in the CEUS, a nominal kappa value of 0.04s based on an average of many empirical estimates predominately in the WNA tectonic regime is proposed. Sparse analyses for deep soil sites in the CEUS suggest 0.04s reflects some conservatism. However it should be noted the small strain total kappa is rapidly exceeded (i.e., becomes less important) as loading level increases due to nonlinear response. The initial low strain kappa serves primarily as a means of adjusting (lowering) kappa to accommodate the scattering component due to the profile randomization. Hence, no significant bias in the final amplification functions at loading levels of engineering interest is anticipated.

For cases of shallower soils, less than 1000m (3,000ft) to hard rock basement material, the empirical relation of Campbell [12] should be used for the contribution to kappa from the thickness of the sediment column (H):

$$\kappa \text{ (ms)} = 0.0605 * H \text{ (thickness in meters).}$$

The assumed basement kappa value of 0.006s (Table B-4) is used in lieu of Campbell's [7] estimate of 0.005s to estimate the soil contribution to total kappa. For 1000m (3,000ft) of soil, Campbell's [12] relation predicts a total kappa of 0.0665s (0.0605 contribution from soil and 0.006 contribution from basement rock) , considerably larger than the mean base-case value of 0.04s, suggesting a degree of conservatism at low loading levels for CENA firm soils. For continuity, in the implementation of Campbell's equation, a maximum kappa of 0.04s should be implemented for sites with less than 1000m (3,000ft) of firm soils.

B-5.1.3.2 Representation of Epistemic Uncertainty in Kappa

The parameter kappa is difficult to measure directly. Since no measurements of the type required exist at the sites of interest, a large uncertainty is applied in the site response analyses. Epistemic uncertainty in kappa is taken as 50% ($\sigma_{\mu Ln}$ = 0.40, Table B-1; [18]) about the mean base-case estimate for this assessment. The uncertainty is based on the variability in kappa determined for rock sites which recorded the 1989 **M** 6.9 Loma Prieta Earthquake [18], and adopted here as a reasonable expression of epistemic uncertainty at a given site. As with the shear-wave velocity profiles (Section 3.2.1), the +/-0.51 natural log units (1.28 σ_{μ}) variation is considered to reflect 10% and 90% fractiles with weights of 0.30 and a weight of 0.40 for the mean base-case estimate. The models for epistemic uncertainty are summarized in Table B-1.

B-5.1.4 Source Model

Alternative conceptual models to represent the earthquake source spectral shape exist in the literature. A single corner frequency model of the earthquake source spectrum has been widely used in the simple seismological model described above [10 and 18]. However, based on the limited ground motion data in CENA as well as inferences from intensity observations, an alternative empirical two-corner source model for CENA earthquakes has been developed [5]. The two-corner source model addresses the potential for CENA source processes to reflect a significant spectral sag at large magnitude (**M** ≥ 6) and intermediate frequency [6], compared to source processes of tectonically active regions. Such a trend was suggested by the 1988 **M** 5.9 Saguenay, Canada and 1985 **M** 6.8 Nahanni, Canada earthquakes. The two-corner source model for CENA [6] incorporates the spectral sag between two empirical corner frequencies which are dependent on magnitude. The two-corner model merges to the single-corner model for **M** less than about **M** 5. Interestingly, the two-corner model has been implemented for tectonically active regions and shown to be more representative of WNA source processes than the single-corner model [7], albeit with a much less pronounced spectral sag than the CENA model.

The debate regarding the applicability of these two source models continues. The lack of relevant observations for **M** >6 in CENA precludes identifying either model as a unique, preferred model. As a result, in the interest of representing the epistemic uncertainty in this element of the control motions, both single- and two-corner [6] source models were used with **M** 6.5 to develop control motions. The two models were considered to reflect a reasonable range in spectral composition for large magnitude CENA sources. As a result, equal weights were selected as shown in Table B-1 to develop amplification factors using each source model.

Additionally, for moderately stiff soils, typical for NPP siting, the difference in amplification between single- and double-corner source models becomes significant only at the higher loading levels as Figure B-13 illustrates. Figure B-13 compares the amplification computed for both the single- and double-corner source models using the EPRI modulus reduction and hysteretic damping curves (Figure B-8), the most nonlinear set of curves for soils. These results suggest the difference in amplification between single- vs. double-corner source models are significant enough to consider the implied epistemic uncertainty in central and eastern North America (CENA) source processes at large magnitude (**M** >6).

B-5.1.4.1 Development of Input Motions

It is necessary to define the site response over a broad range of input amplitudes to develop amplification functions. For sites in the CEUS, the Mid-continent crustal model [18] (Table B-5) with a shear-wave velocity of 2830 m/s, a defined shallow crustal damping parameter (kappa; [4]) of 0.006 s, and a frequency dependent deep crustal damping Q model of 670 $f^{0.33}$ [18] is used to compute reference motions (5% damped pseudo absolute acceleration spectra). The selected Q(f), kappa, and reference site shear-wave velocities are consistent with the EPRI GMPEs (Ground Motion Prediction Equations) [19]. The site-specific profiles are simply placed on top of this defined crustal model which has a reference shear-wave velocity of 2830 m/s (≈ 9,300 ft/s) and a reference kappa value of 0.006 s. Distances are then determined to generate a suite of reference site motions with expected peak acceleration values which cover the range of spectral accelerations (at frequencies of 0.5, 1.0, 2.5, 5.0, 10.0, 25.0, 100.0 Hz) anticipated at the sites analyzed. To cover the range in loading levels, eleven expected (median) peak acceleration values at reference rock are needed to span from 0.01g to 1.50g. Table B-4 lists the suite of distances for the single-corner source model and Table B-6 lists the corresponding distances for the double-corner model.

Amplification factors (5% damping response spectra) are then developed by placing the site profile on the Mid-continent crustal model at each distance with the input motion being equal to the reference rock motion convolved with a diminution function which implements the site specific kappa (e.g. kappa from the equations in Section 5.1.3 and a 0.006s contribution from basement rock) , generating soil motions, and taking the ratios of site-specific response spectra (5% damped) to hard rock reference site response spectra. For the higher levels of rock motions, above about 1 to 1.5g for the softer profiles, the high frequency amplification factors may be significantly less than 1, which may be exaggerated. To adjust the factors for these cases an empirical lower bound of 0.5 is to be implemented [18, 3].

The general framework for the site response calculations are summarized in Figure B-1 and Tables B-1 and B-3.

B-6.0 Development of Probabilistic Hazard Curves

The procedure to develop probabilistic site-specific soil hazard curves was described by McGuire et al. (2001) and by Bazzurro & Cornell (2004) [24, 8]. That procedure (referred to as Approach 3) computes a site-specific soil hazard curve for the spectral acceleration at a selected spectral frequency (or period) given the site-specific hazard curve for the bedrock spectral acceleration at the same oscillator period and site-specific estimates of soil response. The soil response is quantified through the period/frequency-dependent amplification factor, AF(*f*). The function AF(*f*) is given by:

$$AF(f) = Sa_{SOIL}(f)/Sa_{ROCK}(f),$$

where *f* is frequency, and $Sa_{SOIL}(f)$ and $Sa_{ROCK}(f)$, are the 5% damped spectral accelerations at the soil surface and bedrock, respectively. Since the near-surface materials frequently exhibit nonlinear behavior, the variation of AF(*f*) with input intensity needs to be captured. Most commonly the input intensity is quantified by Sa_{ROCK} at the frequency of interest.

In the fully probabilistic approach, the annual probability of exceedance of soil ground motion level *z* ($G_Z(z)$) at spectral frequency *f* is computed as:

$$G_z(z) = \int_0^{\infty} P\left\langle AF \geq \frac{z}{x} \middle| x \right\rangle f_x\,(x)dx$$

Where $P\left\langle AF \geq \frac{z}{x} \middle| x \right\rangle$ is the probability that AF is greater than the quantity $\frac{z}{x}$, given a bedrock amplitude of *x*, and $f_X(x)$ is the probability density function of Sa_{ROCK}.

In discretized form, the above equation can be expressed as:

$$G_Z(z) = \sum_{all\ x_j} P\left[\left(AF \geq \frac{z}{x}\middle|x_j\right)\right] p_x(x_j)$$

Where $p_x(x_j)$ is the annual probability of occurrence for Sa_{ROCK} equal to x_j. This probability is obtained by differentiating the appropriate rock hazard curve. Then, $P\left[\left(AF \geq \frac{z}{x}\middle|x_j\right)\right]$ can be computed by assuming *AF* is lognormally distributed and a function of *x*, since

$$P\left[\left(AF \geq \frac{z}{x}\middle|x\right)\right] = \hat{\varphi}\left(\frac{\ln\frac{z}{x} - \mu_{lnAF|x}}{\sigma_{lnAF|x}}\right)$$

Where $\mu_{lnAF|x}$ is the mean value of *ln AF* given $Sa_{ROCK} = x$, and $\sigma_{lnAF|x}$ is the standard deviation of *ln AF* given $Sa_{ROCK} = x$. The term for $\hat{\varphi}(*)$ is simply the standard Gaussian cumulative distribution function. The parameters $\mu_{lnAF|x}$ and $\sigma_{lnAF|x}$ are obtained from the distribution for AF derived from the site response analyses described above, and are a function of bedrock amplitude *x*.

The site amplification functions are to be developed as described in Sections B-1 through B-5. As discussed in those sections, multiple models of site amplification functions are derived. To compute site-specific hazard results using the equations above, these multiple models are to be combined, with associated weights (See Figure B-1 and Table B-1), to derive overall log-mean and log-standard deviation values for each spectral frequency. For each spectral frequency and input rock amplitude, the total log-mean, μ_T ($\mu_{lnAF|x}$ in the equation above), and log-standard deviation, σ_T ($\sigma_{lnAF|x}$ in the equation above), are calculated as:

$$\mu_T = \sum_i w_i \mu_i$$

$$\sigma_T = \sqrt{\sum_i w_i((\mu_i - \mu_T)^2 + \sigma_i{}^2)}$$

Where *i* indicates individual site amplification models, w_i is the weight on each model, and μ_i and σ_i are the log-mean and log-standard deviation of each site amplification model, *i.*

B-7.0 Hazard-Consistent, Strain-Compatible Material Properties

Section to be added by Walt

B-8.0 References

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TABLES

Table B-1 Site Independent Relative Weights and Epistemic Uncertainties

Table B-1. Site Independent Relative Weights and Epistemic Uncertainty

Parameter	Relative Weight	σ_μ
Mean Base-Case Profile	0.40	0.35
Lower-Range	0.30	
Upper-Range	0.30	
Mean Base-Case Kappa	0.40	0.40
Lower-Range	0.30	
Upper-Range	0.30	
G/Gmax and Hysteretic Damping Curves		0.15*, 0.30**
Soil		
EPRI Cohesionless Soil	0.5	
Peninsular Range	0.5	
Firm Rock		
EPRI Rock	0.5	
Linear	0.5	

* Aleatory variability in modulus at cyclic shear strain 3 x 10^{-2}%

** Aleatory variability in shear-wave damping at cyclic shear strain 3 x 10^{-2}%

Table B-2. Model to Estimate Density from Shear-Wave Velocity

Shear-Wave Velocity (m/s)	Density (g/cm^3)
<500	1.84
500 to 700	1.92
700 to 1,500	2.10
1,500 to 2,500	2.20
>2,500	2.52

Table B-3. Maximum Number of Models to Characterize Epistemic Uncertainty

Parameter	Maximum	Soil	Firm Rock	Soil/Firm Rock
	N	N	N	N
Profile	3	3	3	3
Curves	2	2	2	2
Kappa	3	1	3	3
Magnitude	2	1	1	1
1,2-Corner	2	2	1	2
Total Models	**72**	**12**	**18**	**36**

Table B-4
Suite of Hard Rock Peak Accelerations, Source Epicentral Distances, and Depths (M 6.5; 1-corner source model)

Expected Peak Acceleration (%g)	Distance (km)	Depth (km)
1	230.00	8.0
5	74.00	8.0
10	45.00	8.0
20	26.65	8.0
30	18.61	8.0
40	13.83	8.0
50	10.45	8.0
75	4.59	8.0
100	0.0	7.0
125	0.0	5.6
150	0.0	4.7

Additional parameters used in the point-source model are:

$\Delta\sigma$ (1-corner) = 110 bars
ρ = 2.71 cgs
β = 3.52 km/s
R_C = 60 km, crossover hypocentral distance to $R^{-0.5}$ geometrical attenuation
T = $1/f_c$ + 0.05 R, RVT duration, R = hypocentral distance (km)
Q_o = 670
η = 0.33
kappa(s) = 0.006

Table B-5
Generic CEUS Hard Rock Crustal Model

Thickness (km)	V_s (km/sec)	ρ (cgs)
1	2.83	2.52
11	3.52	2.71
28	3.75	2.78
--	4.62	3.35

Table B-6
Suite of Hard Rock Peak Accelerations, Source Epicentral Distances, and Depths (M 6.5; 2-corner source model)

Expected Peak Acceleration (%g)	**Distance (km)**	**Depth (km)**
1	230.00	8.0
5	81.00	8.0
10	48.00	8.0
20	28.67	8.0
30	20.50	8.0
40	15.60	8.0
50	12.10	8.0
75	6.30	8.0
100	0.0	7.9
125	0.0	6.4
150	0.0	5.4

Table B-7
Geometrical spreading and attenuation (Q(f))models for the CEUS and WUS Used in Figure B-11

Region	**Geometric Spreading**	**Anelastic Attenuation**
CEUS	1/R for R ≤ 60km 1/60 for 60km < R ≤ 130km $(1/60)(60/R)^{0.5}$ for R > 130 km	$Q(f) = 670f^{0.33}$
WUS	1/R	$Q(f) = 180f^{0.45}$

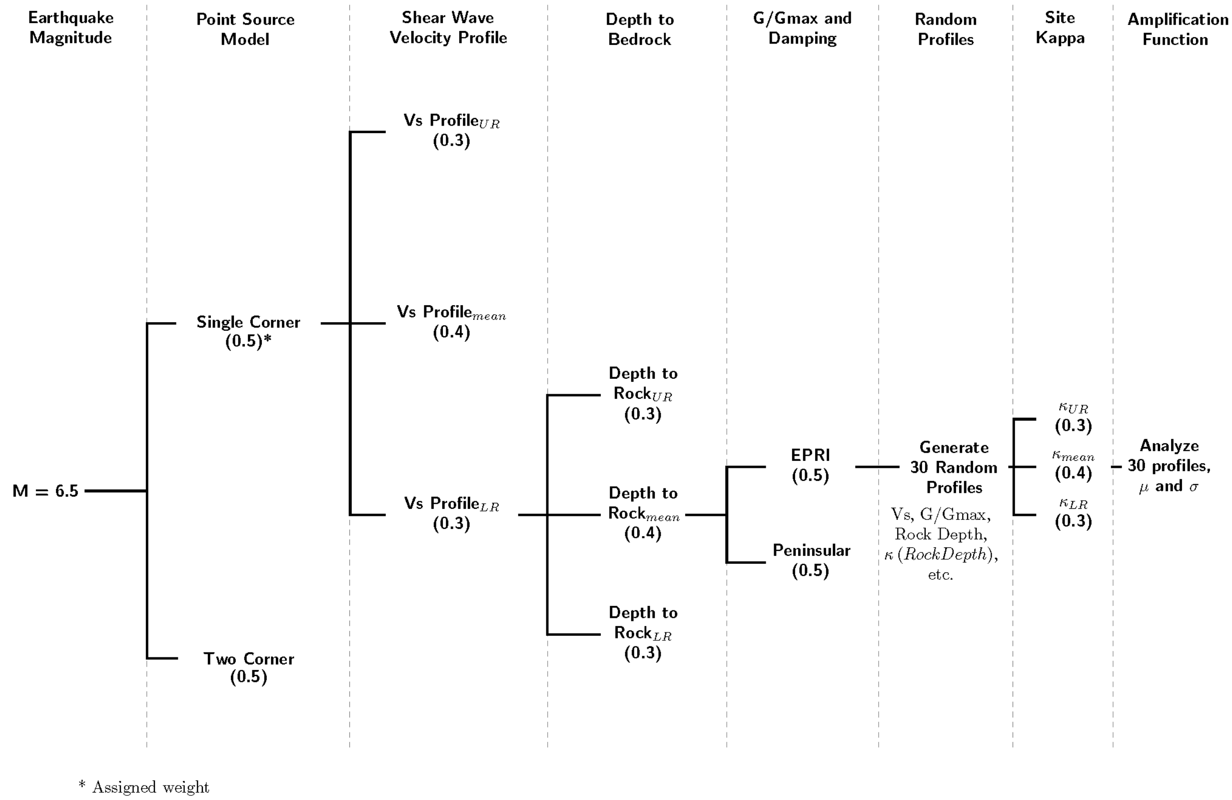


Figure B-1. Logic tree illustrating the process for capturing uncertainty in the development of site-specific amplification functions. This illustration is for a site with limited at-site geophysical and geotechnical data available. UR and LR indicate Upper-Range and Lower-Range about the mean Base-Case model.

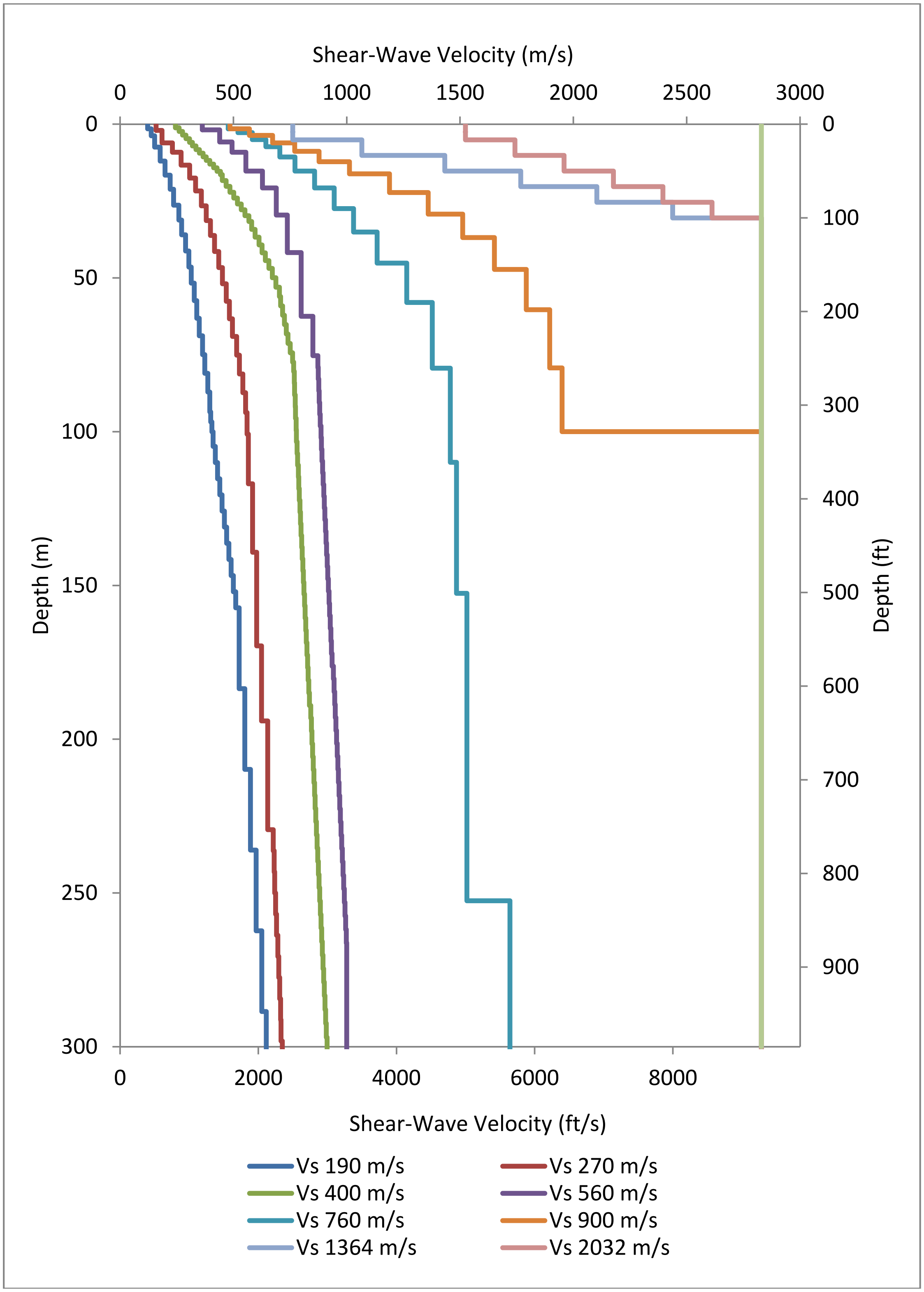


Figure B-2. Template Shear Wave Velocity Profiles for Soils, Soft Rock, and Firm Rock. Rock Profiles Include Shallow Weathered Zone. Indicated velocities are for V_{S30}.

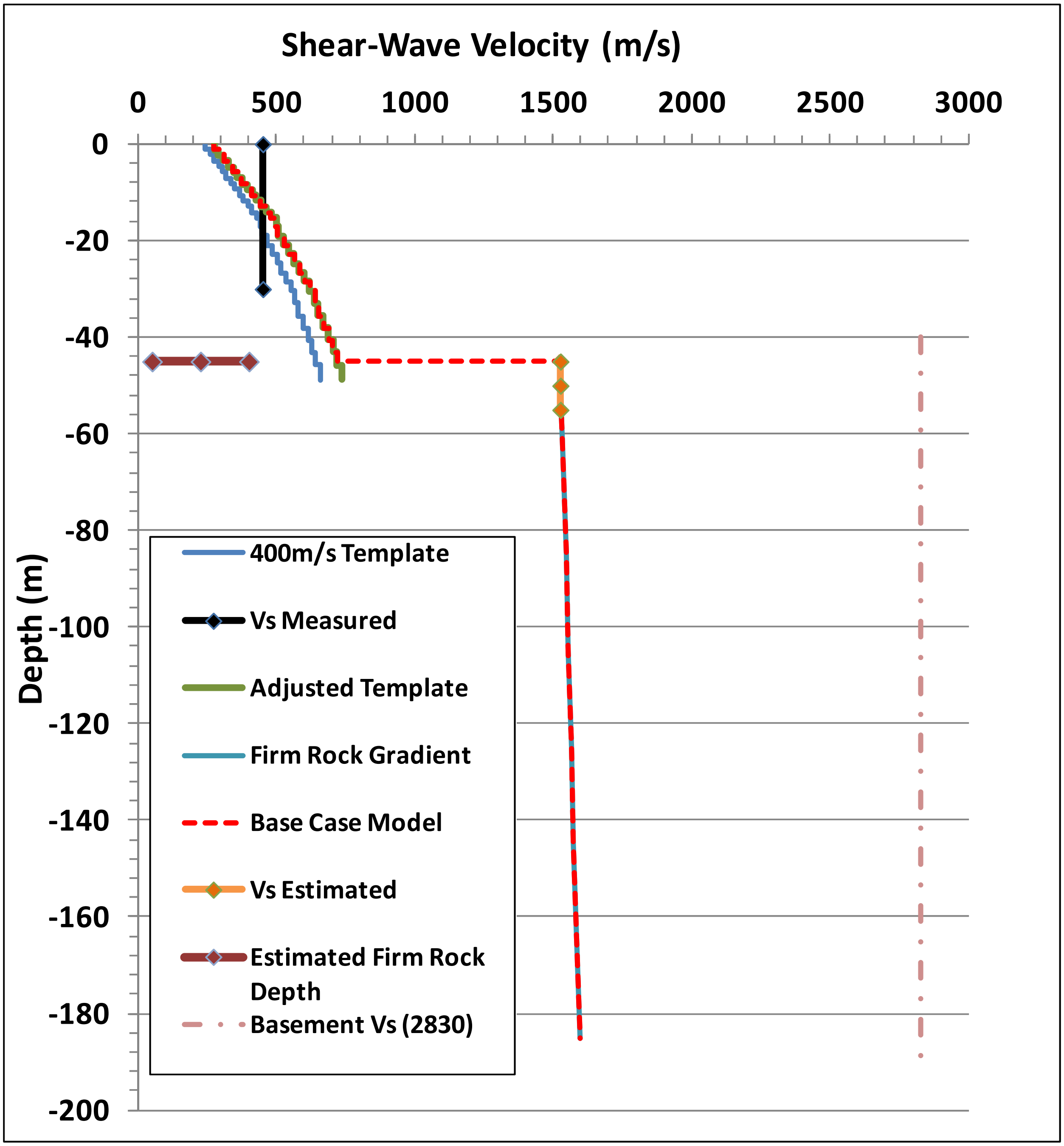


Figure B-3. Illustration of how available information is used to develop a mean base-case profile. The information available is represented by the measured near-surface soil V_{S30} (solid black line), estimated depth to firm rock (solid brown line) and estimated firm rock V_S (solid orange line). Proposed mean base-case V_S profile is indicated by dashed red line which overlies the firm rock gradient below ~45m.

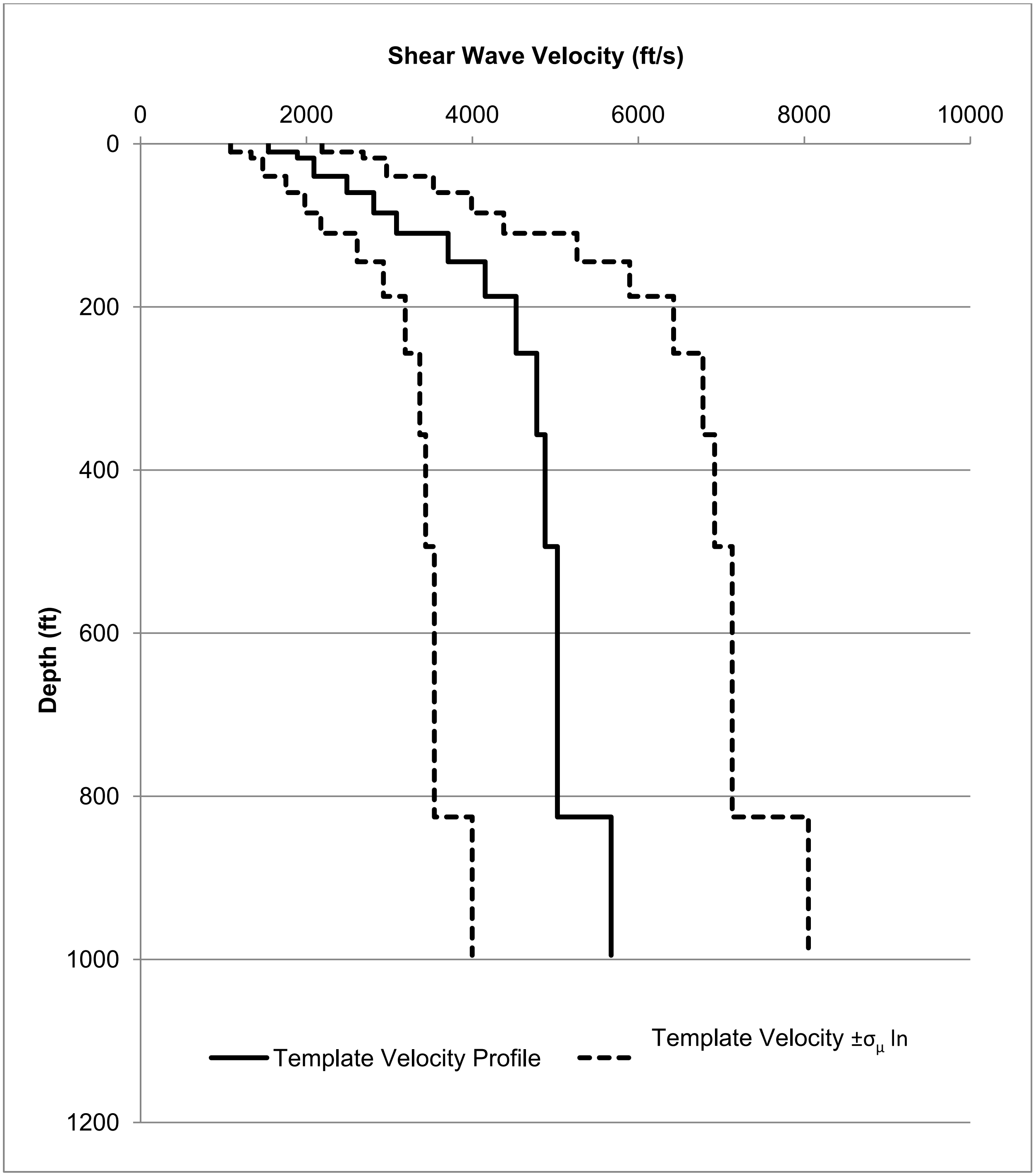


Figure B-4. This figure illustrates the range of velocity implied by the method used to account for epistemic uncertainty in site specific shear wave velocity profiling where sparse or limited information is available. Displayed is the 760 m/s WNA reference rock template velocity (solid curve) with dashed curves representing $\pm\ 1\sigma_\mu$ ln = 0.35.

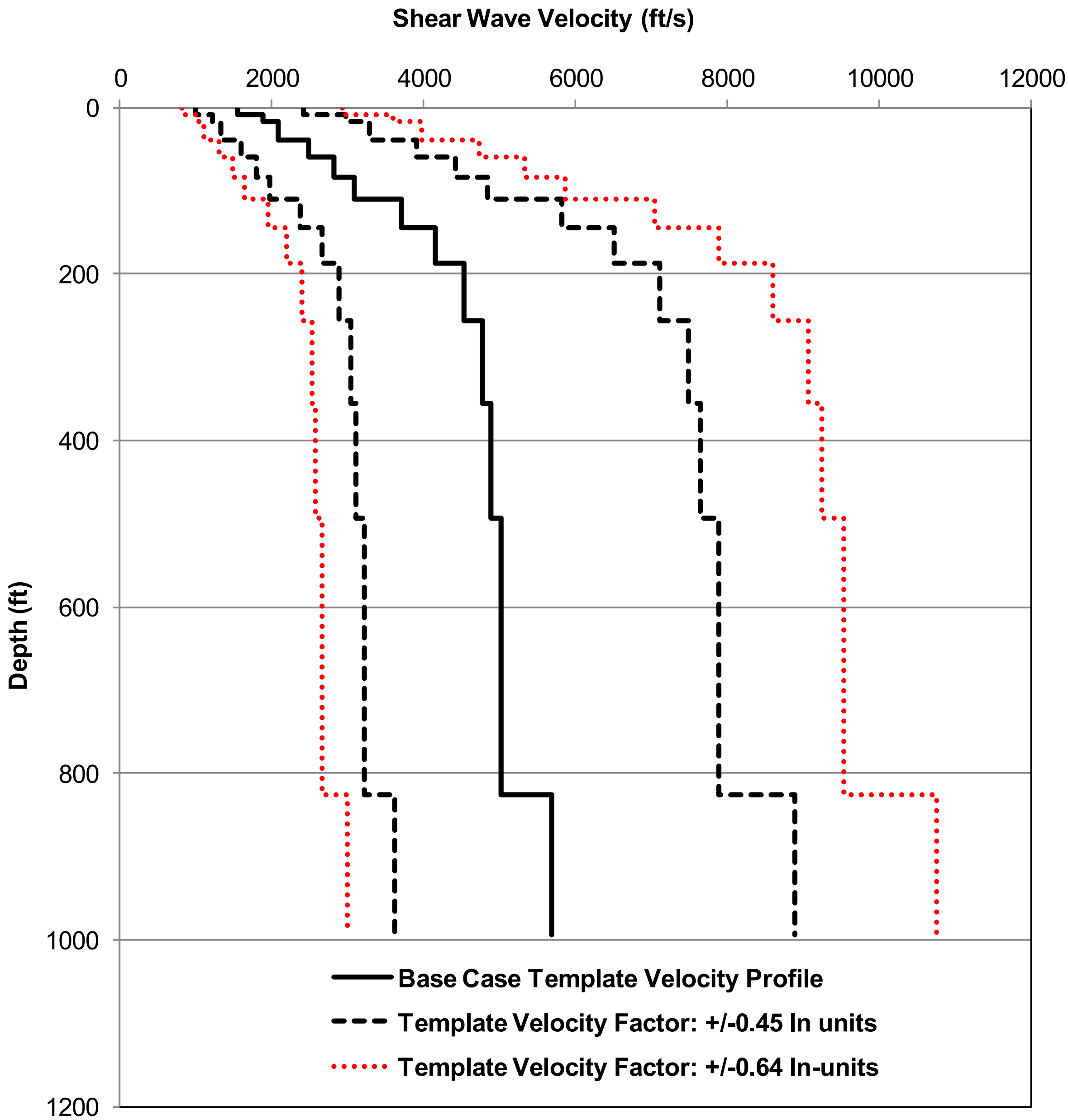


Figure B-5. This figure displays the method used to account for epistemic uncertainty in site specific shear wave velocity profiling where very limited or no information is available. Displayed is the 760m/s reference template velocity (solid black curve) with black dashed curves representing 10^{th} and 90^{th}-percentile values (±0.45 natural log units which corresponds to a σ_{μ} ln = 0.35). Dotted red curves are for ±0.64 natural log units which corresponds to a σ_{μ} ln = 0.5.

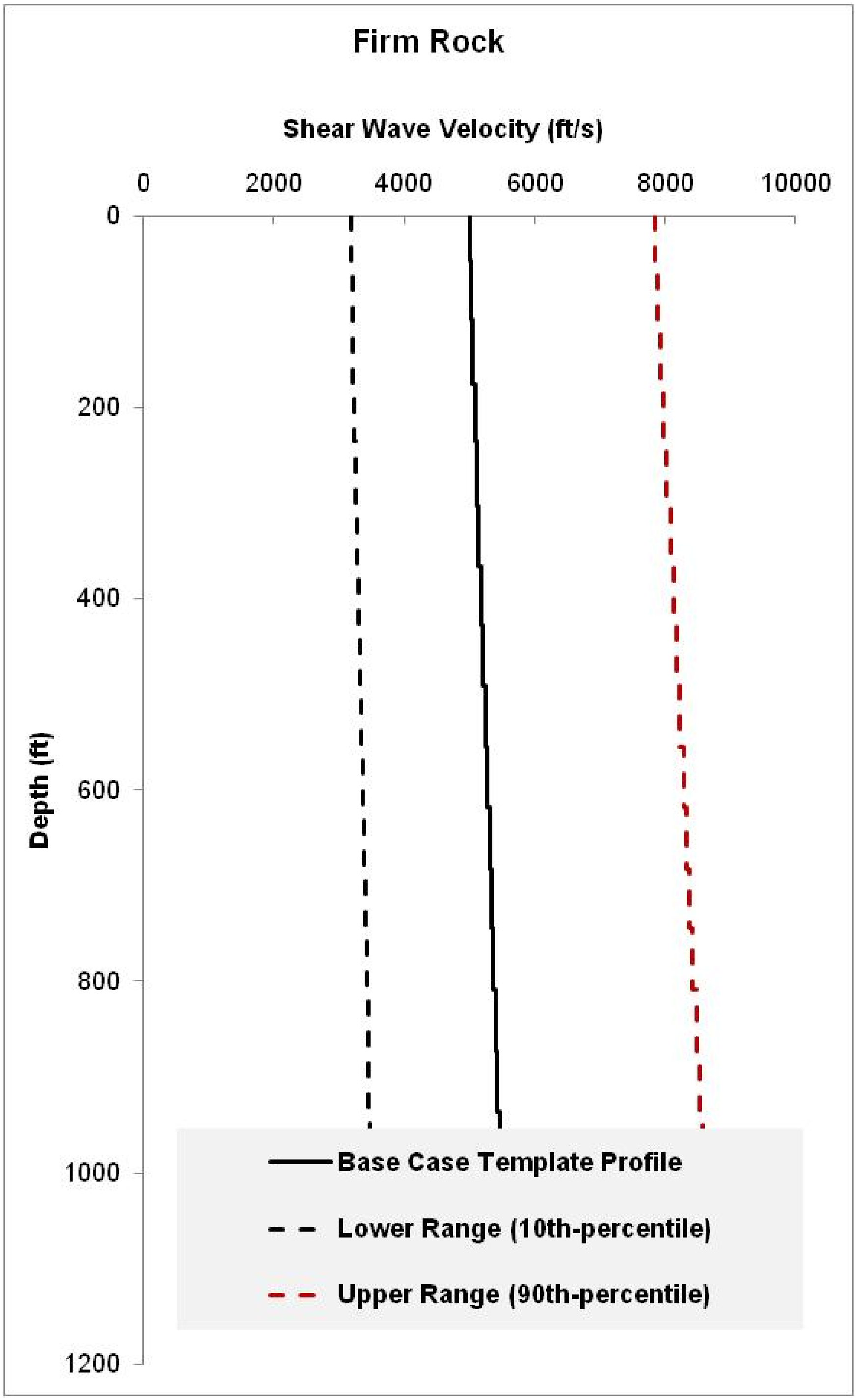


Figure B-6. Illustration of Upper Range and Lower Range Base-Case profiles (10^{th} and 90^{th} percentiles) developed to represent the epistemic uncertainty in the Mean Base-Case for firm rock conditions. A mean surface velocity of 5000ft/s (1525m/s) was assumed for the Base Case and the empirical gradient of Fukishima et al. (1995) [20] was applied. A σ_{μ} ln = 0.35 was used.

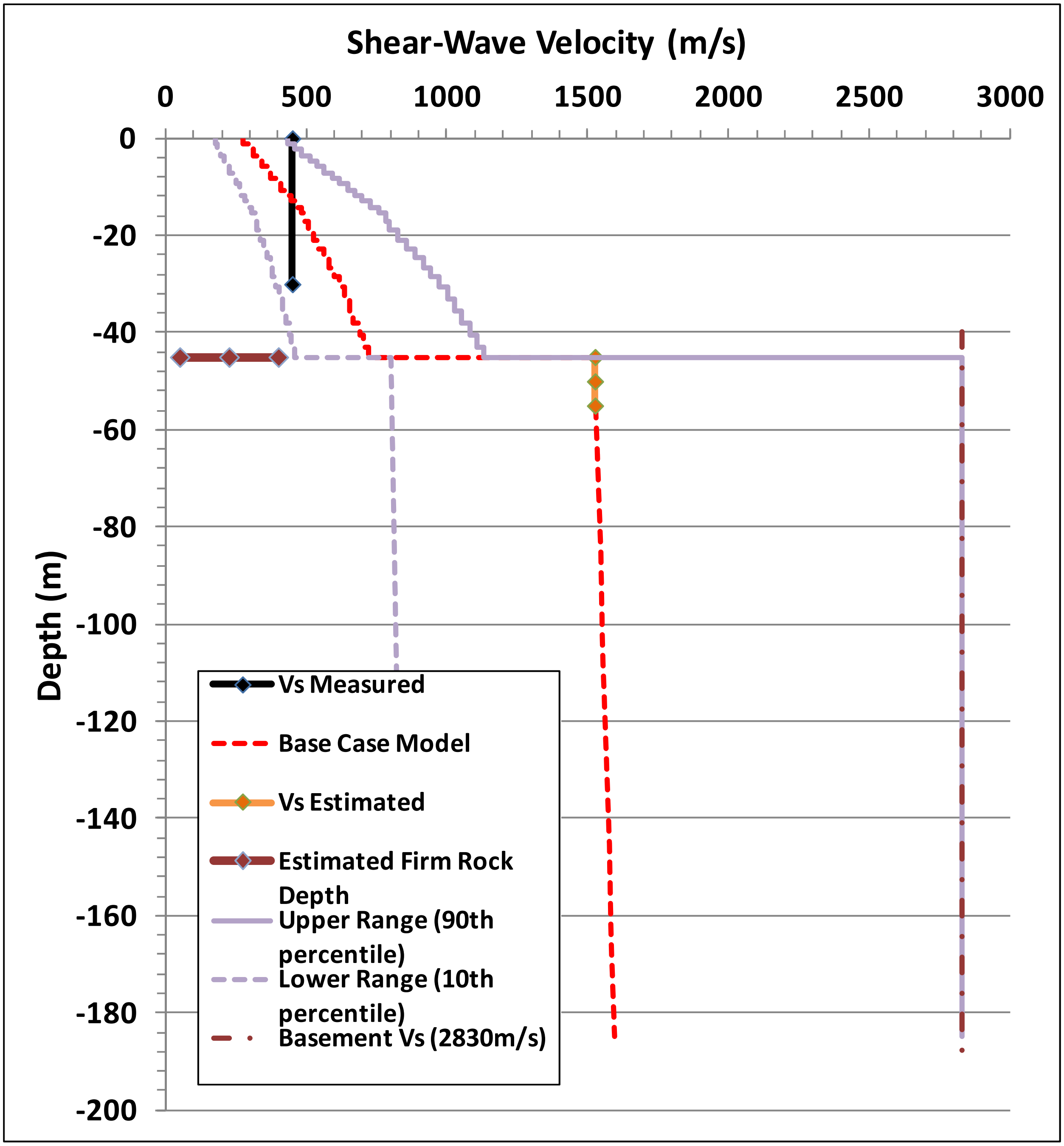
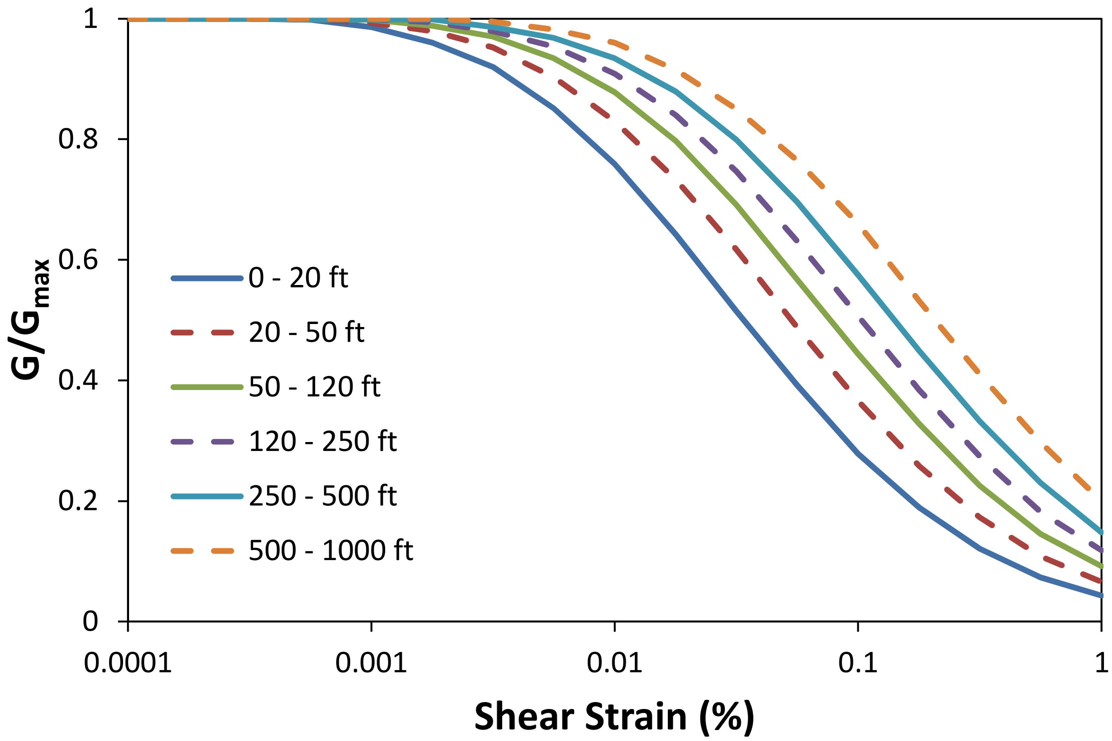


Figure B-7. This figure illustrates the development of Upper Range and Lower Range profiles to accommodate epistemic uncertainties for the hypothetical example shown in Figure B-3. A σ_{μ} ln = 0.35 has been used to develop the 10^{th} and 90^{th}-percentile curves in the upper portion of the profile where sparse Vs measurements were available. A σ_{μ} ln = 0.50 was applied to the lower portion of the profile where the Vs of the Base Case was inferred from geological information. The 90^{th}-percentile curve was capped at a value equal to the 2830m/s Vs value assumed for the hard rock basement.



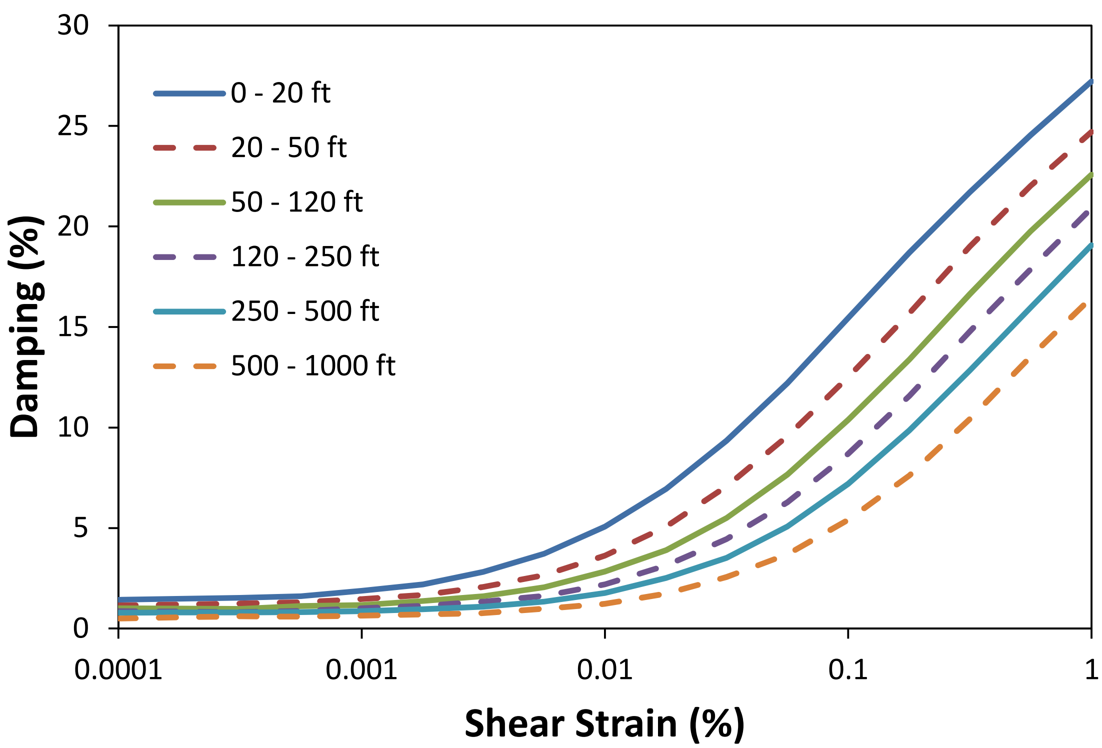


Figure B-8. Generic G/G_{MAX} and hysteretic damping curves for cohesionless soil [18]. Note that damping will be limited to 15% for this application.

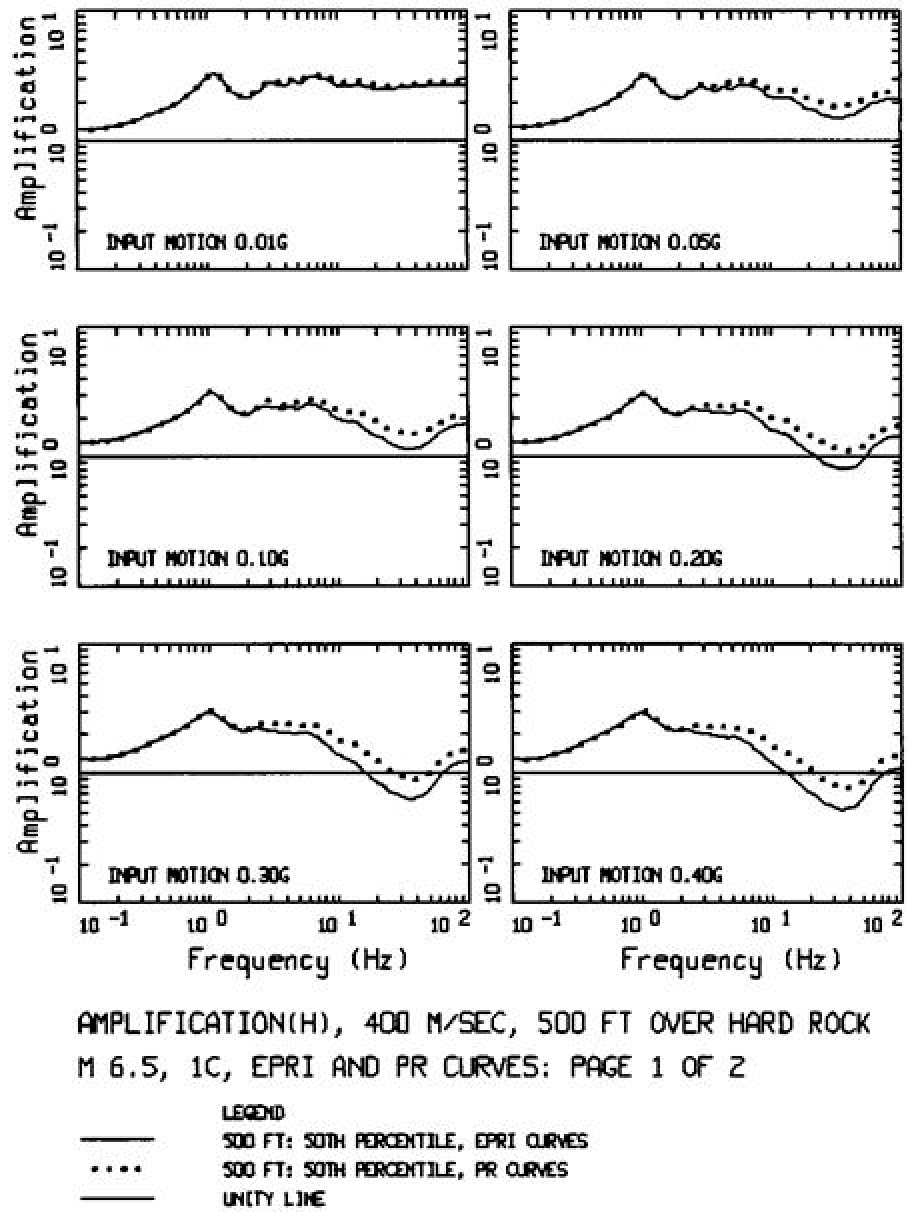


Figure B-9a. Comparison of median amplification functions (5%-damped PSa) derived using the EPRI (1993) [18] (see Figure B-8) and Peninsular Range [32]) G/G_{MAX} and hysteretic damping curves. The results are for the 400 m/sec V_{S30} template profile and a single-corner source model for reference rock loading levels of 0.01 to 1.50g.

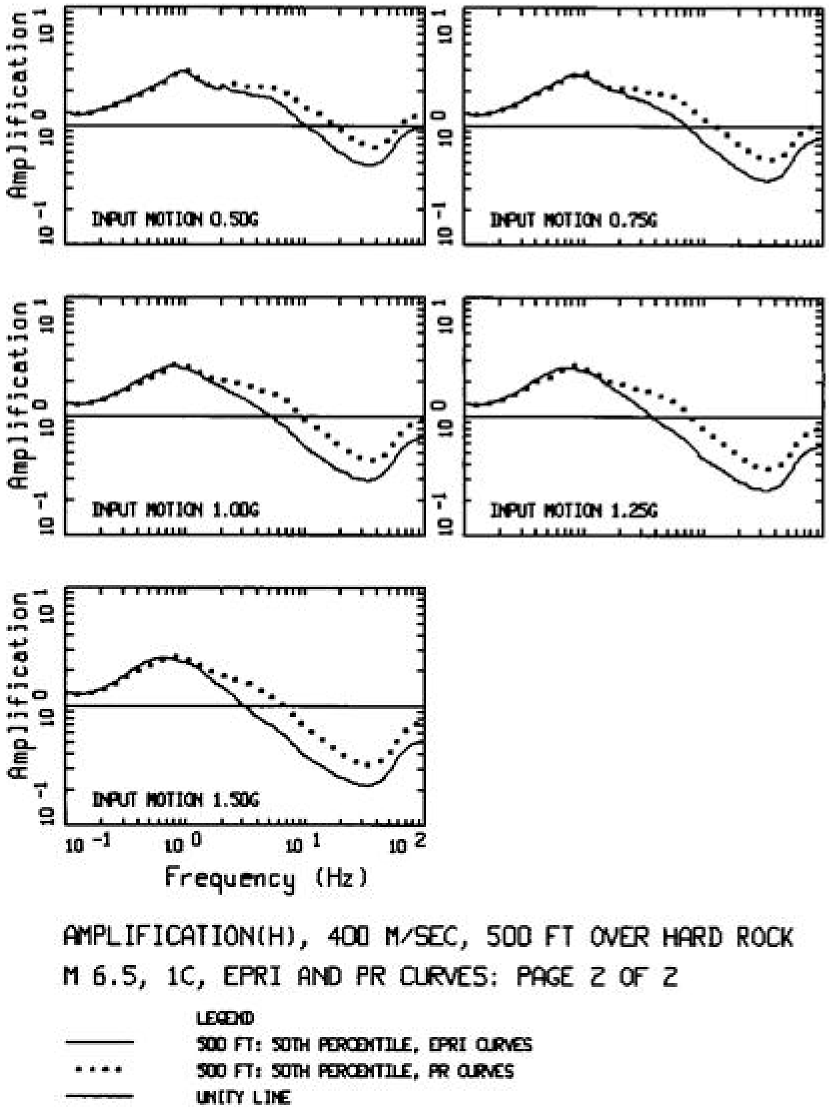


Figure B-9b. Comparison of median amplification functions (5%-damped PSa) derived using the EPRI (1993) [18] (see Figure B-8) and Peninsular Range [32] G/G_{MAX} and hysteretic damping curves. The results are for the 400 m/sec V_{S30} template profile and a single-corner source model for reference rock loading levels of 0.01 to 1.50g.

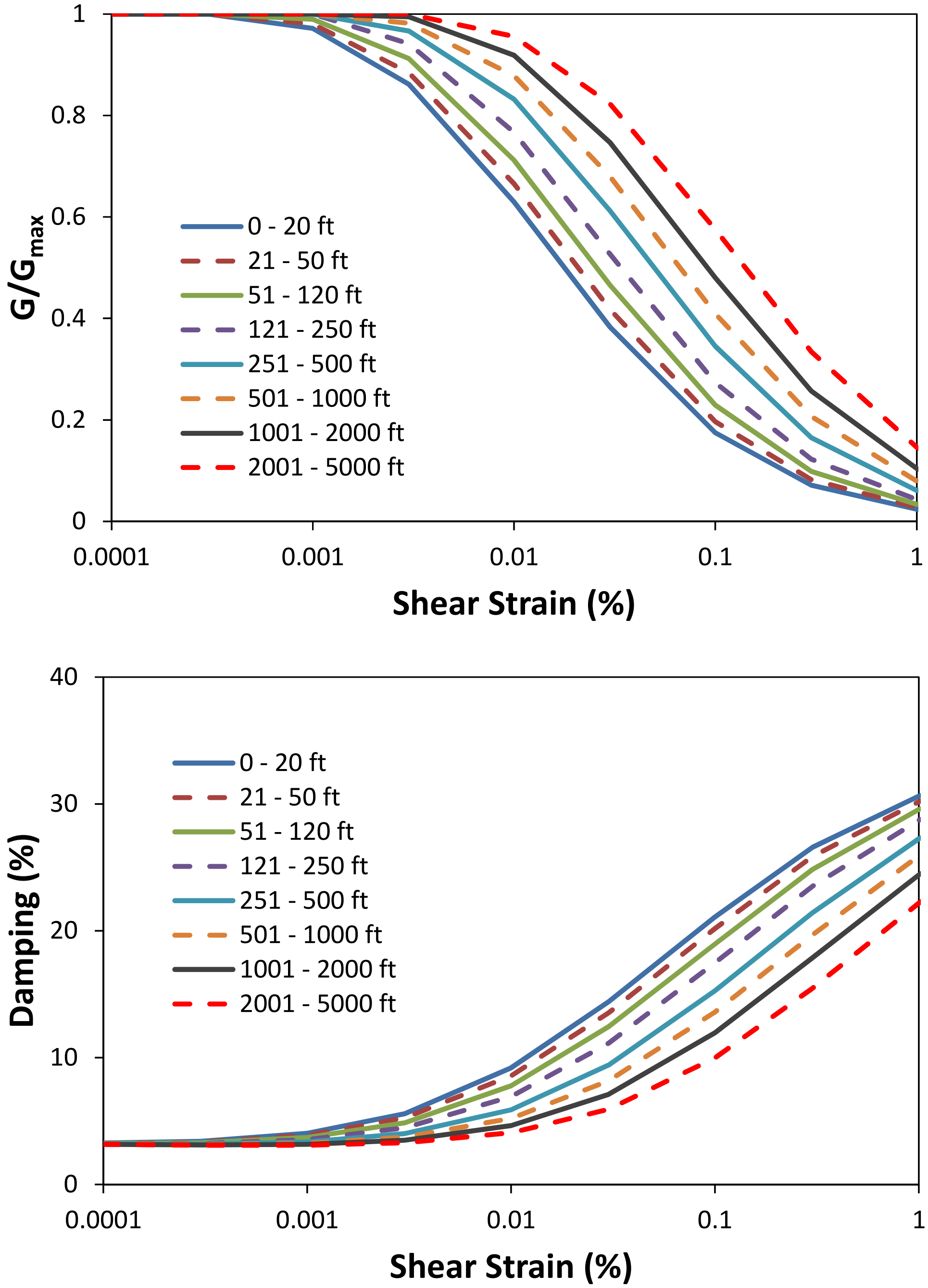


Figure B-10. Generic G/G_{MAX} and hysteretic damping curves developed for firm rock in the EPRI (1993) study [18] (from Dr. Robert Pyke). Note that damping is limited to 15% in this application.

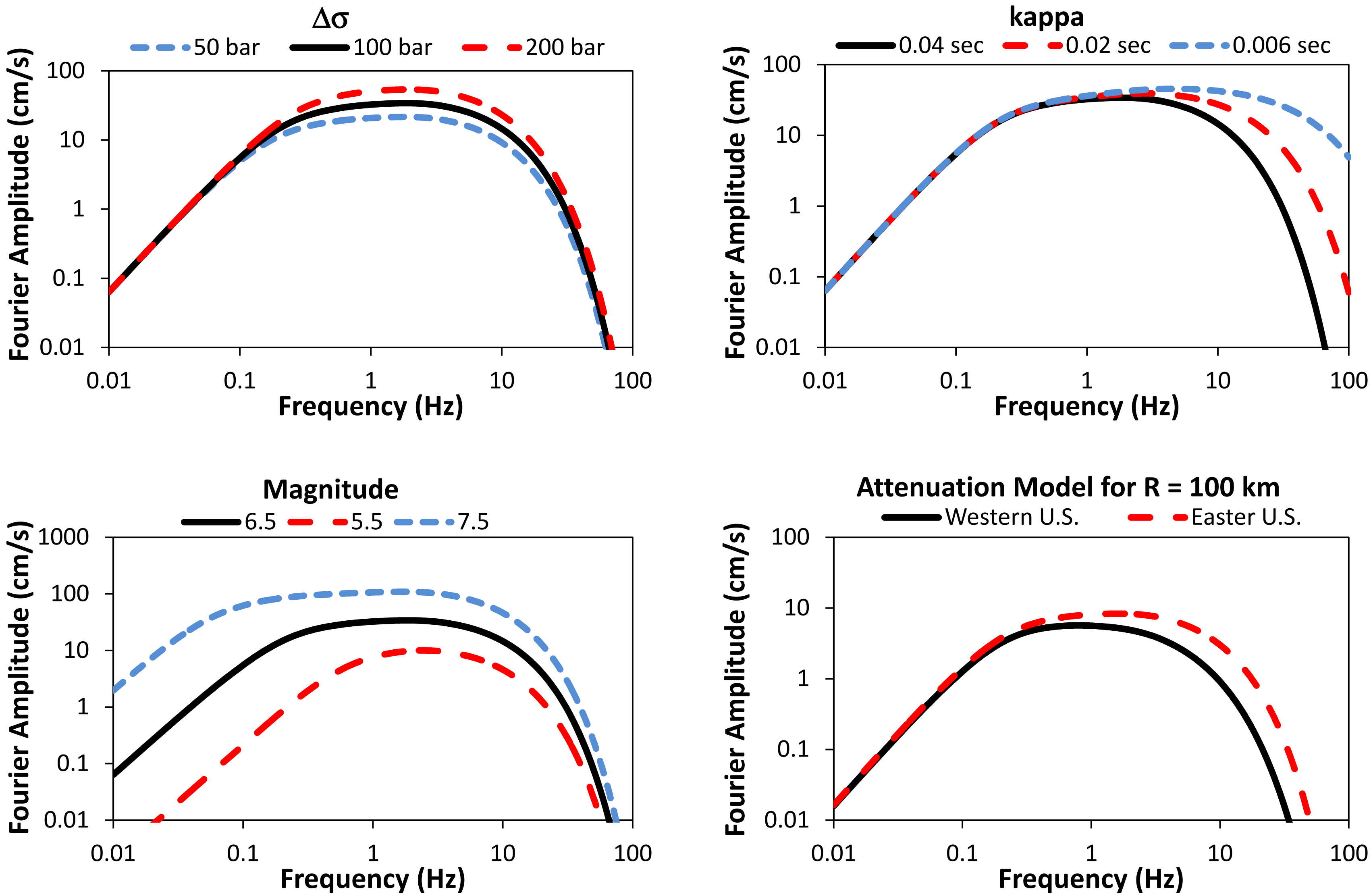


Figure B-11a. Illustration of effect of various factors in the simple seismological model on Fourier spectral shape.

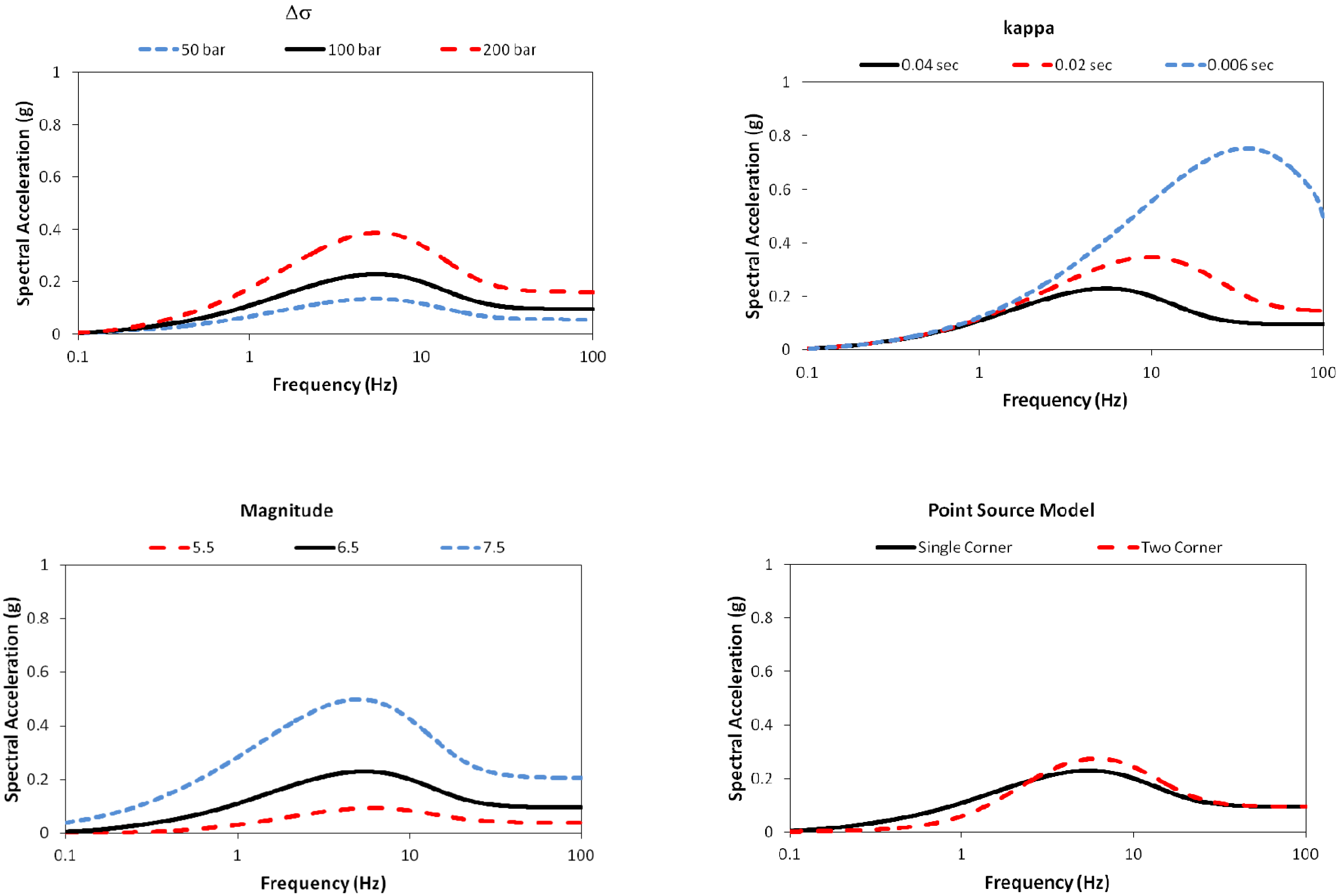


Figure B-11b. Illustration of effect of various factors in the simple seismological model on response spectral shape.

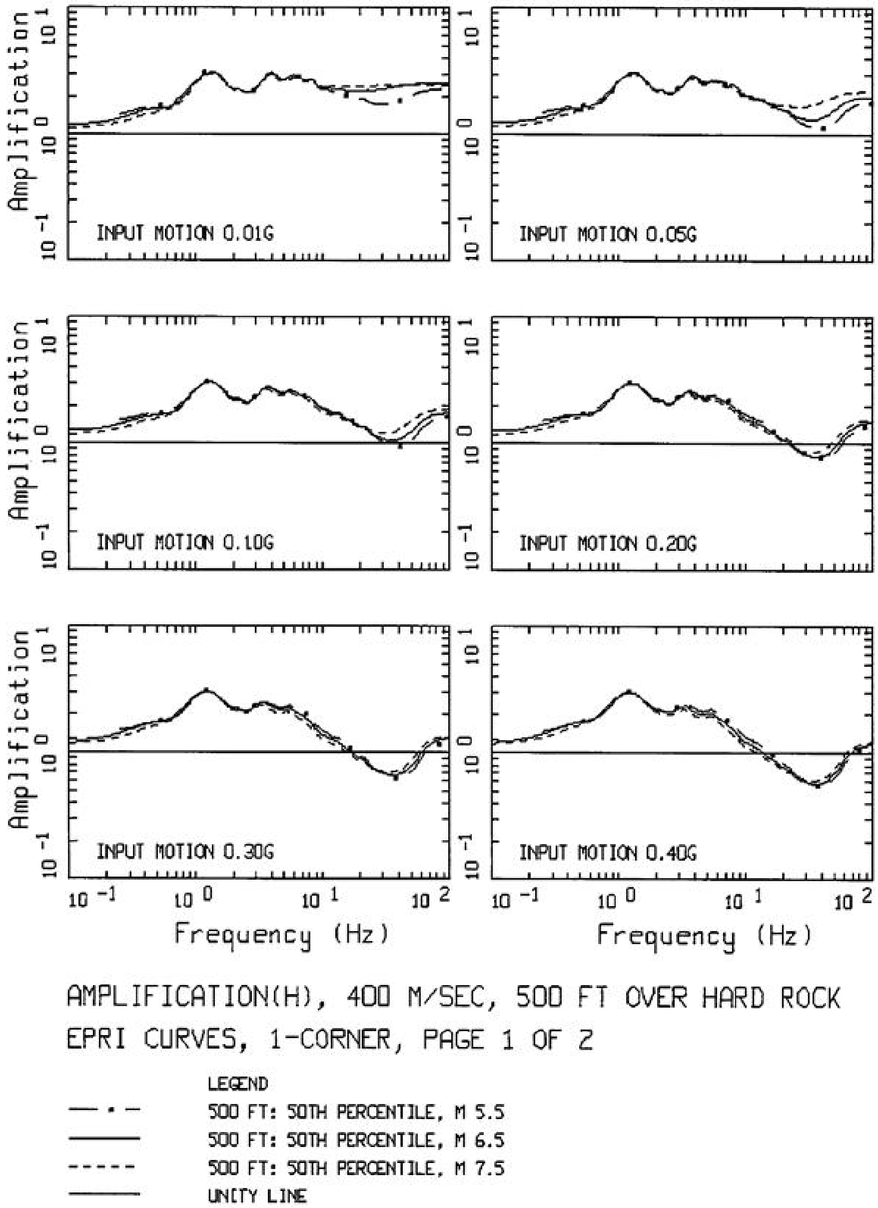


Figure B-12a. Comparison of amplification functions (5%-damped PSa) computed for magnitudes of **M** 5.5, 6.5, and 7.5, using the single-corner source model and the 400 m/sec V_{S30} stiff-soil template profile (Figure B-2) with the EPRI (1993) [18] G/G_{MAX} and hysteretic damping curves (Figure B-8). The input reference rock loading levels varied from 0.01 to 1.50 g (Table B-4).

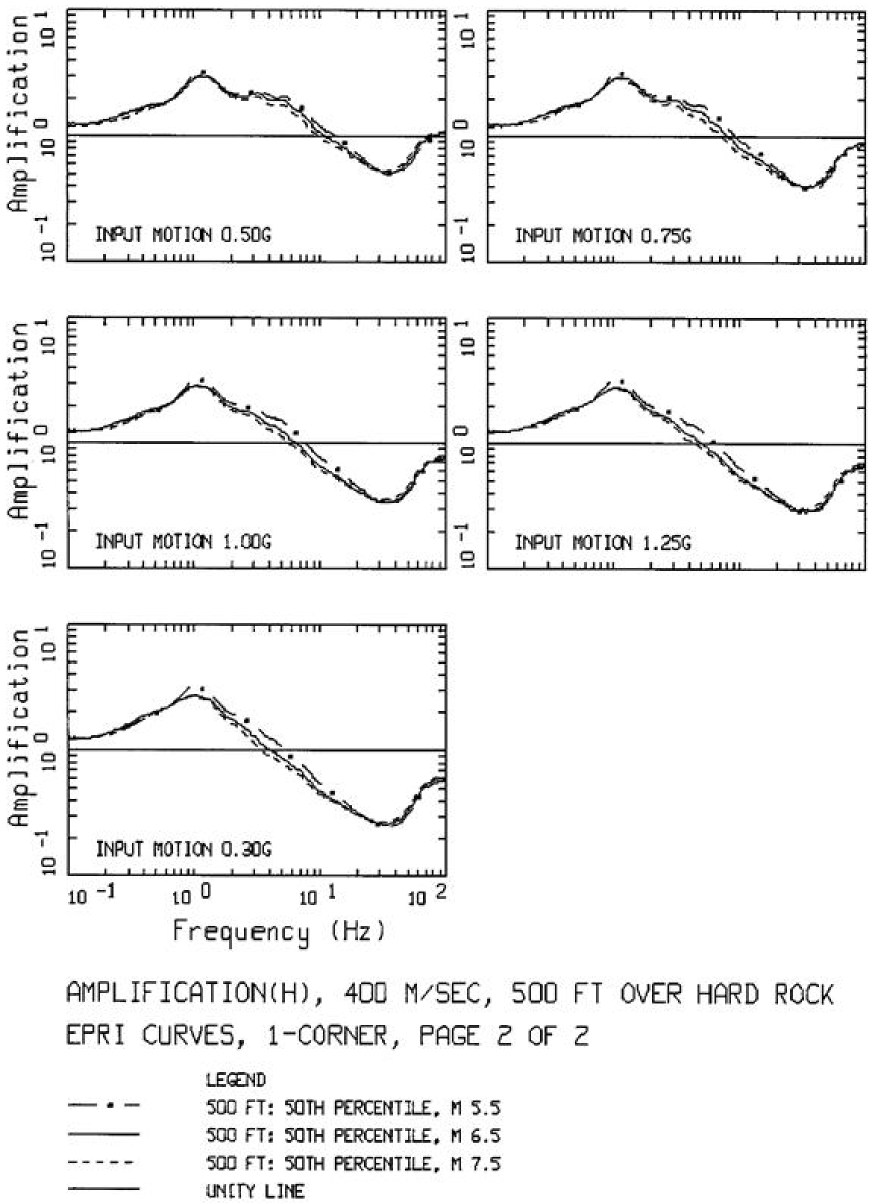


Figure B-12b. Comparison of amplification functions (5%-damped PSa) computed for magnitudes of **M** 5.5, 6.5, and 7.5, using the single-corner source model and the 400 m/sec V_{S30} stiff-soil template profile (Figure B-2) with the EPRI (1993) [18] G/G_{MAX} and hysteretic damping curves (Figure B-8). The input reference rock loading levels varied from 0.01 to 1.50 g (Table B-4).

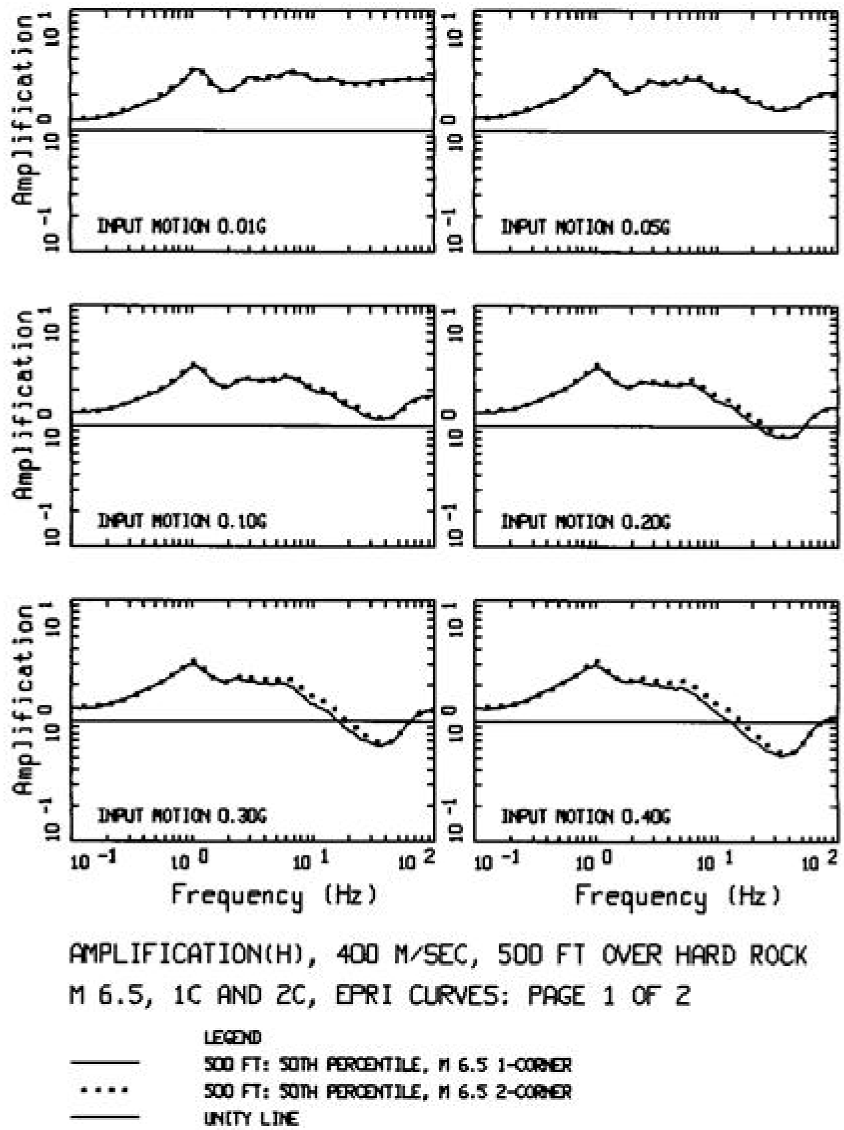


Figure B-13a. Comparison of amplification functions (5% damped PSa) computed using the Single- and Double-Corner source models (Tables B-4 and B-6) for the 400 m/sec V_{S30} stiff-soil template profile (Figure B-2) with the EPRI (1993) [18] G/G_{MAX} and hysteretic damping curves (Figure B-8). The input reference rock loading levels varied from 0.01 to 1.50 g (Tables B-4 and B-6).

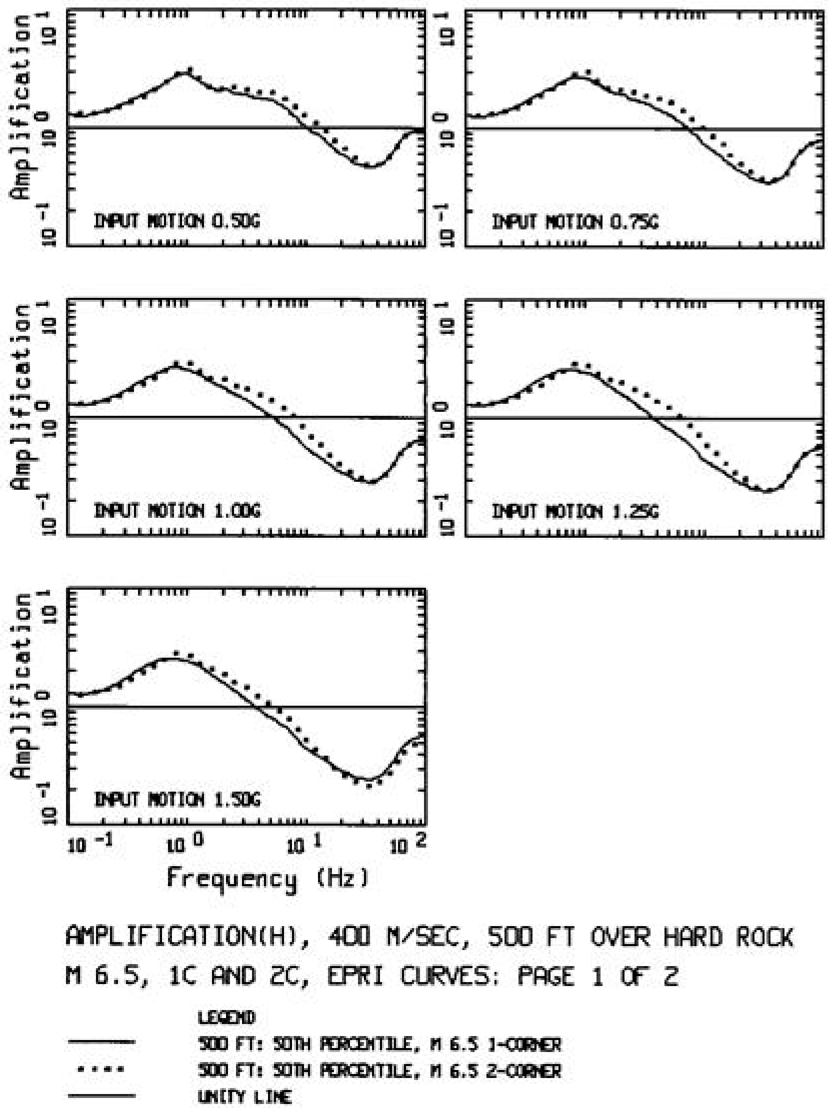


Figure B-13b. Comparison of amplification functions (5% damped PSa) computed using the Single- and Double-Corner source models (Tables B-4 and B-6) for the 400 m/sec V_{S30} stiff-soil template profile (Figure B-2) with the EPRI (1993) [18] G/G_{MAX} and hysteretic damping curves (Figure B-8). The input reference rock loading levels varied from 0.01 to 1.50 g (Tables B-4 and B-6).