ABSTRACT

BARKER, TYLER MADISON. Consistent Site Response Spectra for use in SSI Analysis. (Under the direction of Dr. Abhinav Gupta.)

The objective of this study is to determine if the current-state-of-practice for developing in-layer motions needed in an SSI analysis generates consistent responses and, if not, to propose a solution to do so.

Current trends in nuclear power plant design and licensing require that free-field surface and foundation surface site-specific seismic demands are encompassed by a certified design basis. If the certified design is less than the site-specific seismic demands, then the plant must undergo soil structure interaction analysis, typically using the impedance method. Common SSI programs require a foundation elevation in-layer input response, which must be developed from the foundation surface design spectra.

The current-state-of-practice deconvolves the surface design motion to the bedrock elevation. The bedrock motion is then convolved up to the foundation elevation where an in-layer motion is generated. However, if the in-layer response from the current-state-of-practice is compared to an in-layer response found directly from the same input bedrock motion, then the two responses are not close to each other.

Using a closed-form solution for wave propagation in a two layer soil column over a uniform bedrock halfspace, analysis for harmonic input motion illustrates the concept of "profile-motion consistency," i.e., if a single soil profile is used to propagate the bedrock motions upwards and downwards, then the consistency is maintained. If either the motion or the soil profiles are changed, then the responses of the system become inconsistent. In the current-state-of-practice, the design spectra are determined as the average of multiple spectra, which results in an inconsistent response. Additionally, the current practice uses a

reduced number of soil profiles to transfer ground motions. If the surface design motion is deconvolved to the bedrock, averaging the surface responses can double the response as compared to the original bedrock input.

The closed-form solution is used to define a frequency dependent correction factor, which is specific to the soil profile and soil column height. It can be applied at either end of the soil column, traditionally the surface or bedrock. If the current-state-of-practice is modified such that the bedrock response generated during the motion transfer is multiplied by the correction factor, then the resulting responses are more consistent than those calculated using the current practice.

Consistent Site Response Spectra for use in SSI Analysis

by
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Biography

Tyler Madison Barker was born and raised in Raleigh, North Carolina. He attended North Carolina State University and earned a Bachelor of Science in Civil Engineering and a Bachelor of Science in History, graduating as a class valedictorian and summa cum laude in December 2007. He continued at NC State University studying under Dr. Abhinav Gupta, graduating with a Master of Science in Civil Engineering in December 2009.

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Lastly, I would like to thank my family who has always been a supportive and loving presence in my life, helping me in all facets of my life.

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List of Abbreviations

BE	Best Estimate
BLU	Best estimate, Lower bound, and Upper bound soil profiles
CSDRS	Certified Seismic Design Response Spectrum
COL	Combined Construction and Operating License
DRS	Design Response Spectrum
ESP	Early Site Permit
FIRS	Foundation Input Response Spectrum
GMRS	Ground Motion Response Spectrum
LB	Lower Bound
PGA	Peak Ground Acceleration
SSE	Safe Shutdown Earthquake Ground Motion
SDB	Seismic Design Basis
SDC	Seismic Design Criteria
SSI	Soil-Structure Interaction
SSCs	Structures, Systems, and Components
PF	Target Acceptable Mean Annual Probability of Seismic Induced Unacceptable Performance
UHRS	Uniform Hazard Response Spectrum
USNRC	United States Nuclear Regulatory Commission
UB	Upper Bound

CHAPTER 1: INTRODUCTION

1.1 Regulatory Process

As utilities seek to build the first nuclear power plants since the 1970's, a significant

change is underway in the regulatory acceptance and licensing process. Historically,

utilities applied for regulatory acceptance of a particular plant design unique to a specific

site. Currently, the vendors of new power plants have applied for and are seeking regulatory

certification for a generic seismic design that could be constructed at multiple locations

without a complete relicensing on part of the utility responsible for construction and future

operation. Rather than requiring certification and approval for individual plant designs, the

United States Nuclear Regulatory Commission (USNRC) allows vendors to submit a

standard design for certification, including the Westinghouse AP1000, the Areva EPR, and

the GE ESBWR. Utilities can purchase the certified design from vendors with the body of

the plant already approved, allowing the utility to focus on a Combined Construction and

Operating License (COL) and an Early Site Permit (ESP) that addresses the site-specific

design rather than overall nuclear power plant design and certification.

Nuclear site-specific seismic analysis in the United States is currently regulated by 10

CFR 100, "Reactor Site Criteria," Section 100.23, "Geologic and Seismic Siting Factors,"

which requires the development of a Safe Shutdown Earthquake (SSE) ground motion that

defines the largest probable earthquake that the plant must be able to resist and still allow safe operational shutdown. In order to determine the SSE for a nuclear power plant, it is necessary to determine the design capacity of the plant, consider the site geological conditions, and evaluate seismic demands (USNRC, 1997). Rather than design the plant based on site-specific seismic demands or earthquake motions, vendors design the entire plant for one generic response spectrum. USNRC defines this design basis as the Certified Seismic Design Response Spectrum (CSDRS), which is a response spectrum that covers potential seismic demands for multiple site conditions.

1.1.1 *Certified Seismic Design Response Spectrum*

CSDRS is typically defined as an envelope of multiple design spectra, each of which is evaluated using different approaches and/or different sets of recorded ground motions. Each individual spectrum is normalized to the same value of design Peak Ground Acceleration (PGA). The design spectra can be calculated in one of the following ways:

- Use the design spectra specified in USNRC Regulatory Guide (RG) 1.60 "Design Response Spectra for Seismic Design of Nuclear Power Plants," corresponding to a design PGA such as 0.3 g.

- Use a Probabilistic Seismic Hazard Analysis (PSHA) to generate a design spectra for given site conditions. Multiple spectra can be calculated corresponding to different site conditions that represent an ensemble of typical geological site conditions across the United States.

- Use a set of recorded earthquakes recorded over many decades to arrive at a new design spectrum using a procedure similar to that used for defining spectra in RG 1.60.

The CSDRS is often defined at the base of the nuclear power plant, where it is used as a fixed based input motion to determine in-structure demands. The CSDRS is a site independent spectrum which is used as the design basis for the seismic design of the plant, and is generally a smooth, broad banded spectrum defined at the plant foundation elevation.

1.2 Site-Specific Response Spectra

Once the utility purchases the certified design, the site-specific seismic demands must be compared to the CSDRS. In order to determine the site-specific seismic demands, the current-state-of-practice involves three steps:

- A geological and seismic source analysis is conducted to determine both local and regional geological and seismological characteristics
- A PSHA is used to develop a bedrock motion that incorporates uncertainties in site-specific seismic source analysis, and
- A determination is made of the free-field ground motion response.

The ground motion responses are found using one dimensional wave propagation programs, such as SHAKE (Schnabel, et al. 1972). Soil wave mechanics requires that the response be based on the combination of a vertically upward incident wave and a downward reflected wave. The seismic response at any elevation in the soil column is the summation

of the two waves, which must be equal at a free surface. Any site can be characterized by a predetermined number of soil profiles, say *n*, where USNRC requires a minimum of 60 soil profiles (USNRC, 2007a). The characteristics for each layer of the *n* soil profiles are developed using a Monte Carlo procedure to reflect site soil conditions and capture the variability and uncertainty in the soil properties (BNL, 2007). The results of geological and seismic site analysis and a PSHA include:

- Uniform Hazard Response Spectrum (UHRS) defined at the bedrock elevation,

- Design Response Spectrum (DRS), typically the mean free-field ground surface response, and

- A sufficient number of soil profiles to describe the site conditions .

1.2.1 *Ground Motion Response Spectrum*

While the DRS is a generic term for surface response spectra, regulatory guidelines refer to the DRS as the Ground Motion Response Spectrum (GMRS). The GMRS is found by propagating the bedrock UHRS to the free-field surface elevation using one of several approved methods. USNRC allows seismic demands to be calculated from one of three ways:

- RG 1.60 can be used to develop the free-field surface motion as previously described

- RG 1.165 "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," which incorporated PSHA procedures for site-specific seismic analysis, and

- RG 1.208 "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion" which incorporates PSHA procedures and performance-based design factors.

While all three methods are acceptable for regulatory certification, the 'current' state of practice is based on RG 1.208, and it is described in Chapter 2. The evolution of seismic analysis and previous regulations are described in Appendix A.

1.2.2 *BLU Soil Profiles*

Once the design spectra are determined, subsequent soil-structure analysis uses strain iterated soil profiles developed from the n soil profiles to account for soil weight and overburden pressures. Additional studies do not use the n soil profiles, but instead only three soil profiles are used to minimize computational effort. These three soil profiles are the Best Estimate (BE) which is the mean soil column, the Lower Bound (LB) which is found as the mean minus one standard deviation, and the Upper Bound (UB) which is the mean plus one standard deviation. Collectively, and for the purposes of this thesis, these three soil profiles will be referred to as the BLU soil profiles. The three BLU soil profiles reduce the computational cost in future soil-structure analysis and yet maintain the variability contained in the original n soil profiles.

1.2.3 *Foundation Input Response Spectra*

In its simplest case, the nuclear power plant is at the free-field elevation, and demand and capacity are compared when the CSDRS is checked to envelope the GMRS. However, for some plant designs the structure is embedded to some foundation elevation, such that the CSDRS is no longer comparable with the free-field based GMRS. In order to compare the CSDRS to the seismic response at the foundation, it is necessary to transfer either the GMRS or the UHRS to the foundation level. USNRC refers to this transferred response as the Foundation Input Response Spectrum (FIRS). Typically, this response can be found by propagating the UHRS response up a soil column with the excavated material removed from the analysis. The mean surface response of this truncated column is then comparable to a foundation level CSDRS as an acceptance check of the CSDRS relative to the FIRS response. For surface structures, the FIRS by definition would be equal to the GMRS.

If the CSDRS envelops the corresponding demand curve (FIRS or GMRS), then the nuclear power plant could be certified for the particular site-specific seismic requirements. However, if the demand exceeds the CSDRS, it is necessary to run a Soil-Structure Interaction (SSI) analysis to check the capacity of the plant. Additionally, utilities might consider running an SSI analysis as it generally lowers the seismic demand on the plant, potentially resulting in cost savings for the utility.

1.2.4 *Soil Structure Interaction*

An SSI analysis can be done one of two ways; the direct method or the impedance method, both of which will be discussed in Chapter 2. The direct method uses a full finite

element analysis of the structure and surrounding soil down to the bedrock elevation. For

nuclear power plants, this analysis has high computational costs and instead, a typical SSI

analysis for nuclear power plants uses the impedance method. The soil is replaced by

equivalent springs and dashpots that give the effective stiffness of the soil below the

foundation (Stewart & Fenves, 1999). This requires in-layer soil motions at various control

point elevations as input motions, which are generated from a full soil column. As the FIRS

is taken as the basis for any input motions, the design spectra must be transferred to in-layer

motions at various embedment locations. Figure 1.1 illustrates the relative elevations of

various motions, where the CSDRS is typically defined at the foundation elevation of the

nuclear power plant.
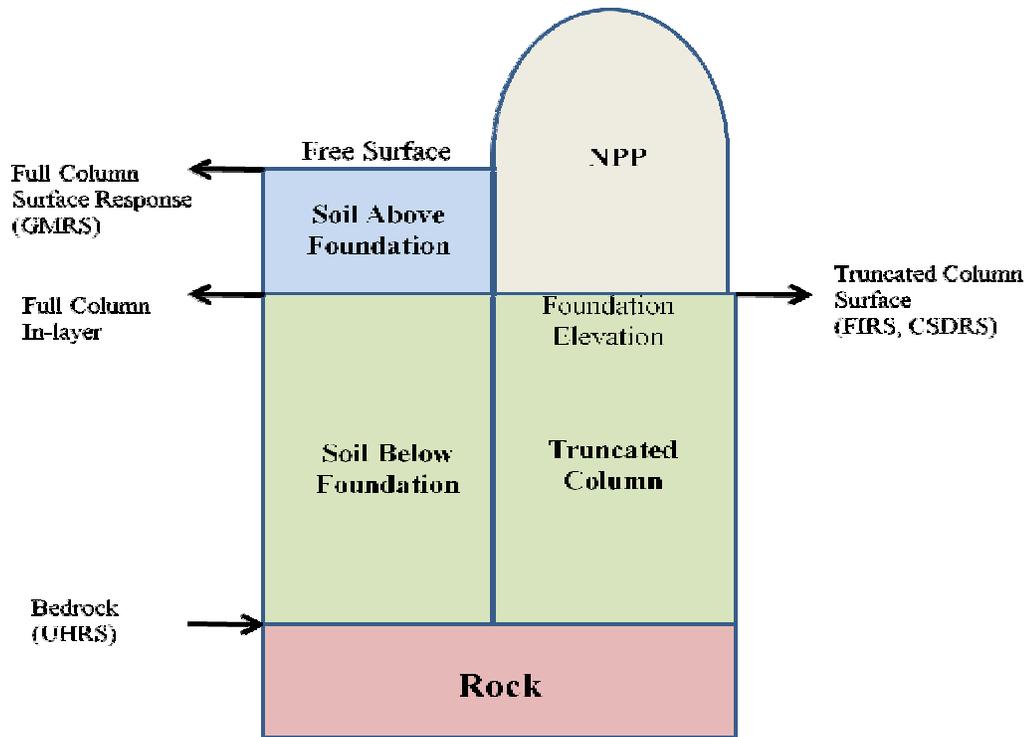
Figure 1.1: Relative Ground Motion Elevations

1.3 Inconsistencies in Current Practice

Generically, the CSDRS, GMRS, FIRS, or any design spectrum can be transferred to an in-layer motion. Transferring this motion begins with deconvolving either the foundation spectrum or the free-field spectrum to the bedrock layer. Theoretically, if the response at the foundation level is then used as the input motion and transferred back to the bedrock level for the same soil profile, the resulting bedrock motion would be identical to the original input bedrock motion. However, when transferring a design motion, the consistent soil profiles and motions are likely altered, such that they would not return the original input. Typically, the SSI analysis and the site-specific seismic analysis are performed separately by two different organizations, where the original inputs used to develop the design spectra are not known (BNL, 2009). In order to run an SSI analysis, the FIRS will be the basis for defining the input motion, and will likely require motion transfer without the original analysis arguments (USNRC, 2008b).

Concerns have been raised regarding the current-state-of-practice. For example, although SSI programs can transfer surface motions to foundation elevations, USNRC has recommended that all motion transfers should be calculated outside of SSI programs, and that foundation level input should be used. Also, USNRC has identified the practice of using the three BLU soil profiles instead of the original n profiles as a potential source of inconsistency. Rather than using the n probabilistic profiles, typical SSI programs use the three BLU profiles in a deterministic approach. As a result, the soil profiles used in the SSI

analysis are not in "strict compatibility" with the input motion (FIRS) generated with the full soil suite, i.e. the set of 60 soil profiles (USNRC, 2009)

Some practitioners have noticed that for some soil profiles the bedrock motion derived from the current practice can contain higher response than the original bedrock input. When this alternative bedrock motion is used as the input to develop the in-layer motions, there is the potential for inconsistency in the in-layer response. While this problem can be avoided if the in-layer responses are calculated at the same time as the GMRS or FIRS, the calculation of surface motions and SSI inputs are traditionally two separate analysis. Thus, any areas for inconsistency must be identified in order to modify the current-state-of-practice to obtain more accurate SSI input motions.

1.4 Problem Definition

Based on the discussion above, we can see that the current-state-of-practice is based on using a bedrock response spectra to define surface design spectra, such as the GMRS or FIRS. In order to generate in-layer motions for SSI inputs, the design spectra is deconvolved to the bedrock using the reduced soil suite. The corresponding bedrock motions are then convolved to the surface, generating in-layer motions. This process can start from one of two locations: the free field surface response (GMRS) or the foundation elevation surface response (FIRS).

In either case, inconsistency may be introduced. The reason is that the original design response spectra were generated using a large suite of n soil profiles whereas, the in-layer

motions are developed using only the three BLU soil profiles. In order to address the question of consistency, one can start with a common bedrock design spectrum.

Using the given bedrock design spectrum, *n* soil profiles can be used to generate the surface design response spectra, the GMRS and FIRS, as well as the in-layer response at the foundation elevation, which will be referred to as the benchmark in-layer response. Using the surface design response spectra generated directly, the response can be transferred to the in-layer motion following the current-state-of-practice procedure. As both the benchmark in-layer and the current practice in-layer are found using the same original bedrock motion and soil properties, the benchmark and current practice are expected to be equal if the current process is consistent. The comparison of the benchmark to the current practice in-layer response can be used to identify any inconsistency in the current practice.

1.5 Objectives

The objective of this thesis is to investigate the current-state-of-practice for characterizing the site-specific seismic design basis and to identify potential areas of inconsistency. Specific tasks needed to address the objective are:

Task 1: Develop an understanding of site-specific ground motion characterization procedures in the current-state-of-practice, including site analysis, PSHA, development of demand spectra, transferring motion between elevations, and required inputs for SSI analysis.

Task 2: Use actual earthquake and soil data to characterize site-specific ground motion. For consistency, begin with the bedrock motion, and then calculate the GMRS, FIRS, and appropriate SSI inputs.

> **Task 2.1:** Identify any potential trends in the case study, as well as potential areas of inconsistency.

> **Task 2.2:** Evaluate any inconsistency that may be introduced in the current-state-of-practice due to the limited number (three) of soil profiles as compared to a larger number (typically 60) of soil profiles at any site.

Task 3: Using a closed-form solution for theoretical wave propagation for harmonic excitation of a two layer soil column, calculate the displacement transfer functions to transfer motion between soil column elevations. Isolate specific reasons for inconsistencies in the current-state-of-practice using the displacement transfer functions as identification of specific issues is often difficult when using actual soil and earthquake data due to the large variability in frequency content and soil characteristics.

Task 4: Propose the necessary modifications to the current-state-of-practice for improved consistency.

1.6 Organization

The thesis is organized as follows:

 CHAPTER 2: CURRENT-STATE-OF-PRACTICE: This chapter introduces the current-state-of-practice regarding site seismic analysis. A brief discussion of geological investigations and a section covering PSHA is included. Additionally, it briefly

examines performance based design and development of GMRS and FIRS, concluding with a discussion of the current method to transfer design spectra to other elevations.

CHAPTER 3: EXPLORATORY STUDY USING RECORDED DATA: This chapter discusses the process and results of a parametric study using actual soil profiles, UHRS, and earthquake motions to show the inconsistency in developing in-layer earthquake motions.

CHAPTER 4: ANALYSIS USING A THEORETICAL SYSTEM: This chapter describes the soil wave mechanics found in programs such as SHAKE, with application to a two layer system. Using displacement transfer functions developed for a harmonic solution, the limitations of current practices are demonstrated and discussed.

 CHAPTER 5: PROPOSED CORRECTION AND LIMITATIONS: This chapter introduces and discusses potential alternatives for transferring foundation responses.

CHAPTER 6: SUMMARY AND CONCLUSIONS: This chapter summarizes the findings of this thesis and presents conclusions, as well as recommendations for future study.

CHAPTER 2: CURRENT-STATE-OF-PRACTICE

The current-state-of-practice described in this thesis is based on several recent regulatory documents. While this section focuses on the most recent site-specific seismic analysis methods, two earlier methods described in Chapter 1 are also acceptable, and are described in detail in Appendix A. The current-state-of-practice site-specific seismic analysis is based on:

- American Society of Civil Engineers ASCE 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities" (2005), which defines procedures for performance-based design

- USNRC RG 1.208 (2007a), which combines probabilistic seismic analysis with performance based design

- USNRC *Standard Review Plan* NUREG-0800 section 3.7.1 *"Seismic Design Parameters"* (2007b), which outlines the evaluation procedure for regulatory certification of site-specific seismic analysis

- USNRC *"*Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses" or ISG-17 (2009), which focuses on the definition of various spectra and earthquake motions, as well as techniques for transferring ground motion based on the relative elevations of the plant foundation versus the demand curves.

2.1 Site Characterization

As a first step in site-specific site characterization, any potential nuclear power plant sites need to undergo a comprehensive investigation that includes geological, seismological, geophysical, and geotechnical engineering analyses of both the site and region. For the site *region* (200 mile radius), it is necessary to conduct geological and seismological investigations to determine any seismic sources (USNRC, 2007a). Seismic sources can include a point source such as a volcano or relatively short fault, a line source from a defined fault line, or an area source where there are several faults. The behavior cannot be simplified to a linear source when several similar faults exist in the same area. For the site *vicinity* (25 mile radius), a more detailed study is necessary to identify any tectonic or seismogenic sources, or to provide evidence that such sources do not exist, and to identify any seismic sources that may need more detailed investigation. At the site *area* (5 mile radius), it is necessary to determine the potential for tectonic deformation near the ground surface as well as the transmission characteristics of the local soil. Finally, at the site *location* (0.6 mile radius), a detailed geological and geophysical investigation is necessary to develop detailed soil and rock characteristics. Additionally, the soil hydrologic conditions should be analyzed to check for potential liquefaction (USNRC, 2007a).

2.2 PSHA

Following a site characterization study, the next step for defining the site-specific seismic demands is a PSHA. The PSHA can be broken into three main steps:

1. Develop a seismic-hazard source model, which defines any earthquake sources, such as those found in the geological investigation, and also determines the probability of a seismic event from each source.

2. Select a ground motion model, which uses attenuation relationships to relate PGA to distance for some magnitude of earthquake.

3. Perform the probabilistic calculation which is the probability of exceeding some design earthquake in a given return period (Field, 2005).

Additionally, some discussion will cover procedures to handle the high level of uncertainties in the PSHA as well as a process known as deaggregation.

2.2.1 *Characterizing Seismic Potential*

The seismic potential is the combination of the location, magnitude, and rate of occurrence for significant earthquakes. The location is discussed in Section 2.1, where the seismic sources are characterized based on their shape and location relative to the site. Additionally, a probability distribution is applied based on the likelihood of an event at a specific location. Usually, sources are assumed to have a uniform probability for an event across the length or area of the source. The probable site to source distance can be approximated as a probability density function dependent on site to source distance, r, and fault length, L_f (Kramer, 1996).

$$f_R(r) = \frac{r}{L_f \sqrt{r^2 - r_{min}^2}}$$

2.1

Additionally, time uncertainty must be taken into account to determine the relative frequency of a specific earthquake. Typically, a Poisson Model is employed, which is based on the principle that events occur randomly without effects from time, size, location, or past events. Other models argue that through elastic rebound theory, earthquakes are not independent of previous events, where a large earthquake will decrease the probability of another event of the same magnitude for the same fault. While these models are particularly useful when a single source dominates the seismic hazard, there is often insufficient data to use the more complex temporal models. As a result, the Poisson Model is widely used in PSHAs for both simplicity and not needing historical earthquake data. For a mean annual rate of exceedance[1], λ_m, and some duration of interest, t, the probability of one earthquake exceeding the frequency (Kramer, 1996) can be defined as:

$$P[N \geq 1] = 1 - e^{-\lambda_m t}$$

2.2

2.2.2 *Earthquake Magnitude Determination*

The earthquake magnitude uncertainty is based on the ability of the fault to produce an earthquake of a particular magnitude during some period of time. All sources have a maximum potential earthquake magnitude, with an increasing probability of lower magnitude earthquakes. In order to determine the site-specific response for different earthquakes, an attenuation relationship is selected for some response term relative to distance. Typically peak ground acceleration (PGA) is plotted versus distance, with

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[1] The inverse of the mean annual rate of exceedance is the return period for earthquakes exceeding that magnitude.

different attenuation relationships for various magnitudes and soil or tectonic regimes (Field, 2005).

To account for the wide scatter in recorded earthquake data, a recurrence relationship is needed. A widely accepted relationship, the Gutenberg-Richter Recurrence Law, is based on the theory that the return period of smaller earthquakes would be lower than that of a larger earthquake, where the relative return values can be defined linearly on a semi-log scale (Kramer, 1996). Additionally, the spread in earthquake data is accounted for by using a predictive relationship, commonly represented by a log-normal distribution, such that 95% of observed earthquakes will fall within a factor of 2.7 from the predicted PGA. This step in the PSHA has a high degree of uncertainty, with limited data in some geographic areas or limited data points from higher magnitude earthquakes (Field, 2005).

2.2.3 *Probability Computations*

In order to combine the various uncertainties in seismic design, the PSHA can express a response in one of several ways including seismic hazard curves or ground motion estimates. For a given earthquake scenario, the probability of exceeding some particular earthquake parameter (such as PGA or CAV), λ_y, is a function of the log-normal predictive relationship, and probability density functions for magnitude and distance (Kramer, 1996).

$$\lambda_y = \sum_{i=1}^{N} \iint P[Y > y \mid m, r] f_M(m) f_R(r) dm dr \qquad \textbf{2.3}$$

Summing this probability over all possible scenarios and including the Poissonian

temporal probability, the seismic hazard curve can be determined, relating the rate of

exceedance to PGA (Field, 2005). Additionally, the UHRS can be determined at a bedrock

elevation for varying recurrence values. The PSHA includes the UHRS for annual

frequencies of $1x10^{-4}$, $1x10^{-5}$, and $1x10^{-6}$ which are later used to calculate the site-specific

demands such as the GMRS or FIRS.

2.2.4 *Uncertainty and Deaggregation*

Uncertainty in the PSHA comes from two primary sources, epistemic and aleatory.

Epistemic uncertainty is due to a lack of knowledge, such as the value of the true mean and

standard deviation for an attenuation relationship. This type of uncertainty can be handled

by performing a seismic analysis for multiple scenarios and then enveloping all of the

individual hazard curves. An alternative method to account for epistemic uncertainty is to

use logic trees to map out multiple scenarios and using the varied responses to account for

individual uncertainties and develop a final model (Field, 2005). Aleatory uncertainty is due

to variability in nature in the sense that we have not measured every possible earthquake.

Aleatory uncertainty can be accounted for in the probability distributions by the selection of

predictive relationships, such as the log-normal distribution. The mean and the standard

deviation of the log-normal distribution are selected in order to account for the scatter in

earthquake data, and our uncertainty in defining the next earthquake. It is extremely

important to document the individual decisions and assumptions made throughout the PSHA

to account for all uncertainties. PSHAs are subject to review by multiple experts, as small

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differences in analysis and assumptions for the same location can result in substantial

differences (over 10%) in response (Field, 2005).

Deaggregation is a process where the probable earthquake for a given return period can

be defined in terms of an additional parameter in the probability calculation. Deaggregation

is a useful tool for checking the results from the PSHA, where it is often favorable to

calculate the most probable earthquake as a factor of magnitude or distance to the site. This

is achieved by moving the specific probability density function out of the probability

calculation in Equation 2.3. The removed probability density function is instead included as

a term in the probability of exceedance. For example, the mean annual rate of exceedance

as a function of the magnitude would be

$$\lambda_y(m_j) = P[M = m_j] \sum_{i=1}^{N} \iint P[Y > y \mid m_j, r] f_R(r) dr \qquad \textbf{2.4}$$

RG 1.208 requires deaggregation to check the PSHA, and also presents a procedure that

can determine the controlling earthquake through deaggregation (USNRC, 2007a).

2.3 1-D Soil Columns: Equivalent Linearization

While the theoretical discussion of one dimensional wave propagation is discussed in

detail in Chapter 4, it is necessary to obtain a general idea of the inputs and processes of

programs such as SHAKE and its application in PSHA and in transferring ground motions.

While earthquake motion is generally in the horizontal direction in the earth's crust, the

waves refract upwards at soil boundaries, so that in surface soil layers the earthquake waves are nearly vertical. Thus, horizontal bedrock motion will generate vertical waves that create horizontal displacement in the soil layers. A soil deposit can be defined as a single column, made up of *n* discrete soil layers overlaying a bedrock uniform halfspace. The properties of each soil layer can be uniquely defined with properties such as unit weight, layer thickness, shear wave velocity, and damping. As the vertical waves propagate through the soil column, equilibrium of strain and shearing stress are maintained between each soil layer.

2.3.1 *Soil Properties for Generation of Design Spectra*

For the PSHA and later transferring motion, the site is typically defined by at least 60 random soil profiles developed using a Monte Carlo procedure to account for variability in soil properties. When the UHRS is transferred from the bedrock to calculate the GMRS, these *n* soil profiles need to be strain iterated (USNRC, 2008a). All soil analyses have highly non-linear material behavior, due to a variety of effects including damping and non-linear stress-strain behavior. In order to maintain consistency in future analyses, these non-linear effects can be approximated by using soils with strain dependent shear wave velocities and damping values. When calculating the GMRS, the soil column goes through several iterations to maintain compatibility of shear stress and strain between each soil layer. After each iteration, the soil shear wave velocity and damping are changed based on the layer strain and repeated until equilibrium is achieved. After convergence, the resulting soil profiles are the strain iterated soil profiles used in later analyses. Subsequent analyses are all linear because using non-linear analysis with the truncated soil columns would remove

the effects of the soil above the foundation elevation, which is assumed to be similar in weight as the future nuclear plant. All analyses past the GMRS calculation are linear with the assumption that the iterated soil profiles account for the non-linear behavior (BNL, 2009).

2.3.2 *Soil Profiles for SSI Analyses*

In order to reduce the computational demand, generation of SSI input motions and the required motion transfer utilize a reduced soil suite. Starting with the *n* strain iterated soil profiles, three more soil profiles are generated. The first profile, or Best Estimate, is the mean set of properties from the soil suite. The Lower Bound soil profile is the mean soil profile minus one standard deviation. The Upper Bound soil profile is calculated from the mean soil profile plus one standard deviations. Collectively, these three soils are reffered to as the BLU soil profiles, which are used to account for the variability in the response of the original soil suite and reduce computational effort in future analysis.

2.3.3 *Wave Mechanics and Corresponding Responses*

For one dimensional wave mechanics, response is based on the combination of an upward incident wave and a downward reflected wave. These two waves can be used to determine four separate responses. At any free surface, the two waves must be equal, and generate a geologic outcrop response. Between soil layers, the two waves can be added algebraically, generating an in-layer response that incorporates effects from soil above and below. A third output is what many typical soil seismic analysis programs call an 'outcrop'

response at some foundation elevation in the soil column. This response is generated as twice the incident wave, but is not the same as a geologic outcrop because the incident wave is influenced by the soil above the outcrop elevation. As a result, it is not advisable to use the 'outcrop' response at any locations other than a free surface (BNL, 2009). The more appropriate method for finding an outcrop response at some foundation elevation is to remove the soil layers above the foundation level and create a truncated column that is then used to develop a geologic outcrop response. It is important to use the full column during the strain iterations to account for nonlinearities, whereas the strain iterated soil profiles can be truncated and still incorporate the effects of the soils removed above the foundation elevation.

2.4 Performance Based Motion

ASCE 43-05 defines an approach to apply probabilistic, performance based goals for various nuclear facilities. In order to establish a target performance goal, the facilities must be evaluated on a graded approach, including performance, qualitative, and quantitative goals. For a graded approach, structures, systems, and components (SSCs) have varying failure consequences, and appropriate seismic design levels. Additionally, these SSCs have performance goals – both quantitative and qualitative – that define acceptable performance.

Quantitatively, the facility has a Target Acceptable Mean Annual Probability of Seismic Induced Unacceptable Performance, PF, based on Seismic Design Criteria (SDC). Qualitatively, the SSCs should have appropriate Limit States ranging from large permanent damage to primarily elastic distortions. The SDCs are listed in Table 2.1 and the Limit

States are defined in Table 2.2. The combination of SDC's and Limit States results in a

Seismic Design Basis (SDB) category (BNL, 2007).

Both the SDC and Limit States are determined based on the criteria in ANSI/ANS 2.26,

which then defines the SDB for the SSCs in the facility. With the SDB, ASCE 43-05

references ANSI/ANS 2.27 and 2.29 to characterize the site and appropriate ground motion

found in a PSHA. Additionally, ASCE 43-05 defines a Probability Ratio (Rp) and Hazard

Exceedance Probability (HD), as shown in Table 2.3, which are then used to develop the

design basis earthquake (DBE) for each SDC. RG 1.208 adopted the procedure issued in

ASCE 43-05 with the understanding that all nuclear power plants are designated as SDB-5D

facilities, or the most conservative designation (USNRC, 2007a).

Table 2.1: Seismic Design Criteria

SDC	Target Performance Goal (P_F)	Approximate Return Period
3	1×10^{-4}	10,000 years
4	4×10^{-5}	25,000 years
5	1×10^{-5}	100,000 years

Table 2.2: Limit States

Limit State	Qualitative Performance Goal
A	Large Permanent Distortion
B	Moderate Permanent Distortion
C	Limited Permanent Distortion
D	Essentially Elastic

Table 2.3: SDC Design Parameters

	SDC		
	3	4	5
Target Perfomance Goal (P_F)	1 x 10^{-4}	4 x 10^{-5}	1 x 10^{-5}
Probability Ratio (R_P)	4	10	10
Hazard Exceedance Probability (H_D) $H_D = R_P \times P_F$	4 x 10^{-4}	4 x 10^{-4}	1 x 10^{-4}

In the PSHA, the UHRS is found at the bedrock level for varying annual frequencies. In order to determine the GMRS, it is necessary to generate the surface-based UHRS. Using the *n* soil profiles (typically not strain iterated), an earthquake seed is fitted to the UHRS and the corresponding motion is used as an input at the bedrock level. Both the 1x10^{-4} and 1x10^{-5} annual frequency surface-based UHRS are developed as the mean response of the *n* individual responses. Using design factors for SDB-5D design and the PSHA generated amplification factors, the surface-based UHRS are scaled to create the GMRS by using a design factor, DF, or

$$DF = \max\{1.0, 0.6(A_R)^{0.8}\}$$

2.5

where A_R is a frequency dependent ground motion slope ratio of the spectral accelerations from the P_F and H_D,

$$A_R = \frac{mean \ 1x10^{-5} \ surface \ UHRS}{mean \ 1x10^{-4} \ surface \ UHRS}$$

2.6

and GMRS defined on the basis of the DF and the 1 x 10^{-4} surface UHRS as

$$GMRS = surface\ UHRS * DF \qquad\qquad \textbf{2.7}$$

The GMRS is thus determined as the free-field motion at the ground surface. There is some ambiguity, as the free-field is sometimes defined as the surface of the top competent soil layer rather than the true free-field surface. This requires a "hypothetical outcrop" surface motion with the material that would be excavated on site removed from the soil column, only after the soil profiles are strain iterated for the entire soil column. The FIRS can be found in the same way as the GMRS by calculating the truncated surface UHRS transferred and applying the appropriate design factor (NEI, 2009).

2.5 SSI Methods

When SSI analysis is required, there are two potential methods for analysis, the direct method and the impedance method, shown graphically in Figure 2.1 (ASCE, 1998). The direct method models the entire soil-foundation-structure system from the bedrock to the top of the structure. The lateral boundaries are selected to be far enough from the structure to not impact the structural response, where the bottom boundary is either at the bedrock level but does not need to exceed three times the largest foundation measurement below the foundation. The input motion would be the design earthquake, or the UHRS, at the bedrock. While the direct method is a better representation of the soil column, the computational cost results in either a less detailed structural model or a coarser soil mesh. Since a large size model is required for the nuclear power plant, the direct method is not practical for determining structure forces for comparison in design.

The impedance method involves simplifying the soil into equivalent springs and dashpots, reducing the computational cost (Chuhan & Wolf, 1998). The foundation stiffness, or impedance functions, are determined for a rigid foundation. This simplifies the stiffness of the soil layers under the foundation, allowing for a more detailed structural model. For embedded structures, the impedance functions are calculated using a truncated column, where the soil above the foundation has been removed. The input motion must be determined at key control points, including the foundation base, defining the motion at several points around the structure (Bhatt, et al. 2001). Many SSI programs require these motions to be in-layer motions to incorporate the effects of the soil above the foundation elevation. The impedance method is the preferred method for SSI analysis in the nuclear power plant industry, and thus requires generating in-layer motions consistent with the design spectra.
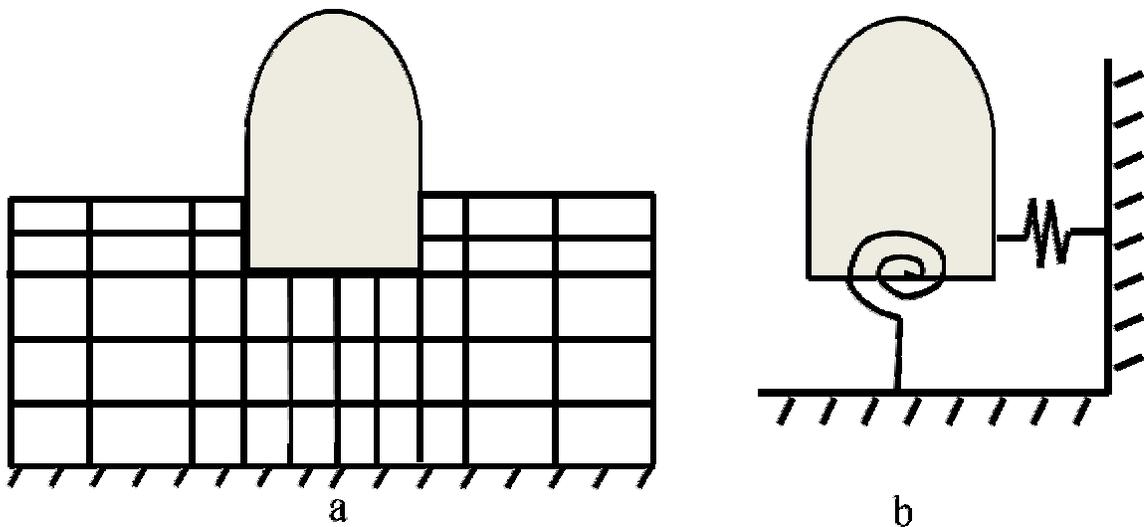
Figure 2.1: (a) Direct Method and (b) Impedance Method for SSI Analysis

2.6 Generation of In-layer Ground Motion

Transferring the PSHA generated UHRS to the surface and generating the GMRS and FIRS only requires convolving the bedrock motion up the soil column. When SSI analysis is required, it may be necessary to instead deconvolve a design spectrum, typically the FIRS, from the free truncated surface first to the bedrock level. This bedrock motion is then used as an input to convolve up to either a free surface or some in-layer response at any elevation. The current process is to first fit an earthquake time history to the design spectrum, developing an input ground motion. This motion is defined at the surface of the truncated column and deconvolved down to the bedrock level, typically using the three BLU soil profiles. The resulting motions at the bedrock are then used as input motions for full column soil profiles. The motion is then convolved to any in-layer elevations or the free surfaces, and can be used either for regulatory acceptance checks or SSI analysis. Typically, SSI programs use the three BLU in-layer responses at multiple foundation elevations, and should not be given surface motions as inputs (USNRC, 2007a).

CHAPTER 3: EXPLORATORY STUDY USING RECORDED DATA

In order to evaluate the consistency in the current-state-of-practice, the entire procedure can be evaluated by starting from a bedrock design spectrum, developing benchmark response spectra, and finally propagating the design motions to develop SSI inputs for comparison to the benchmark. As discussed in Section 2.3, beginning with a bedrock design spectrum, it is possible to calculate consistent motions at the free-field surface, foundation surface, and foundation in-layer elevations. However, there are concerns that the current process to transfer a surface motion to in-layer motions can introduce inconsistency. With the capability to develop in-layer motions directly and use motion transfer to calculate a comparable in-layer motion, a parametric study can be used to study variability that may be introduced in this process. As previously mentioned, the calculation of the design response spectra (GMRS and FIRS) and the transfer of motion for SSI inputs are typically done in separate analyses. However, to check the consistency in the current practice, this study assumes that the surface and in-layer motions can be calculated directly by propagating the bedrock motion, generating benchmark values consistent with the bedrock design spectra and should be consistent with motions generated for SSI inputs.

3.1 Framework

The study is divided into two main steps, the first using the results of actual PSHAs to develop benchmark motions, and the second transferring surface spectra to in-layer motions.

3.1.1 *Generation of Surface Design Spectra and Benchmark Inlayer Response*

Starting with a site-specific PSHAs, extract the 60 strain iterated soil profiles and a bedrock design spectrum, such as the UHRS. The 60 soil profiles are made up of n layers over a uniform bedrock halfspace, and need to be strain iterated to account for overburden stress and nonlinear soil behavior. These 60 soil profiles can be used to develop the Best Estimate, Lower Bound, and Upper Bound, or BLU soil profiles. To find foundation elevation surface motions, truncated soil columns are generated by removing the excavated soil from the top of the full soil columns. It is important to note that the foundation level is the same for both the in-layer elevation and the truncated column surface. This study will examine multiple foundation depths, so for clarity the individual truncated columns will be referenced by their foundation depth. Generic soil columns are illustrated in Figure 3.1, with relative elevations termed as: bedrock (A), full column free-field surface (B), foundation elevation in-layer (C), and truncated column surface (D).
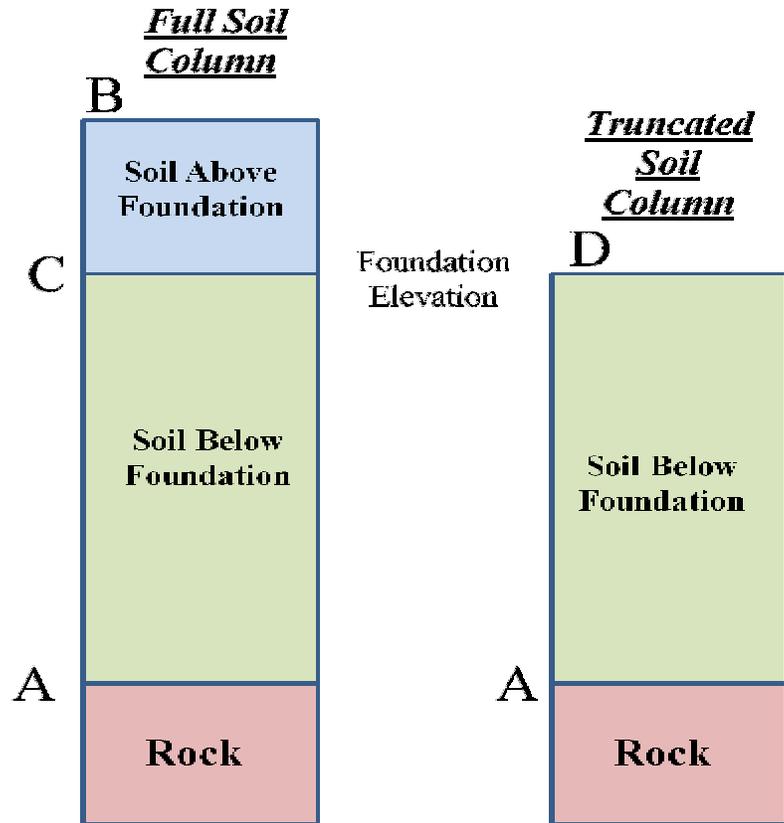
Figure 3.1: Generic Soil Columns and Elevations

The PSHA specifies a bedrock design spectrum, but does not explicitly give an acceleration time history to use as the input motion. In order to develop a time history consistent with the design bedrock response, the first step is to select an earthquake seed (a recorded ground motion) that has similar spectral characteristics to the design spectrum. To create an artificial time history, it is necessary to fit the earthquake seed time history to the bedrock design spectrum and develop an artificial time history. There are several commonly used programs that use Fourier transforms to splice together a consistent time history. This

study uses RASCAL13, which fits an earthquake seed to the bedrock design spectra at point A, generating a bedrock design motion.

Using a soil wave propagation program, in this case SHAKE91, input the 60 soil profiles and BLU profiles for the full column as well as the truncated columns. Soil properties used in SHAKE include thickness, unit weight, iterated shear wave velocity, and iterated damping ratio. The artificial bedrock design motion is input as an outcrop motion at the uniform halfspace, and convolved up to elevations B through D using a linear analysis. The resulting time histories at each elevation are then transformed into response spectra. The spectra corresponding to the individual soil profiles are then averaged at each elevation.

The average response spectrum from the complete soil profile suite at elevation B, the full column free-field surface, is used as the design ground motion of the system and comparable to a GMRS. Similarly, the average response at D, the truncated column surface, is taken as the design surface motion, comparable to the FIRS. The mean in-layer response at C is found directly, and will be used as the benchmark in-layer response for the system. Figure 3.2 illustrates the elevation and location of these benchmark and design values.
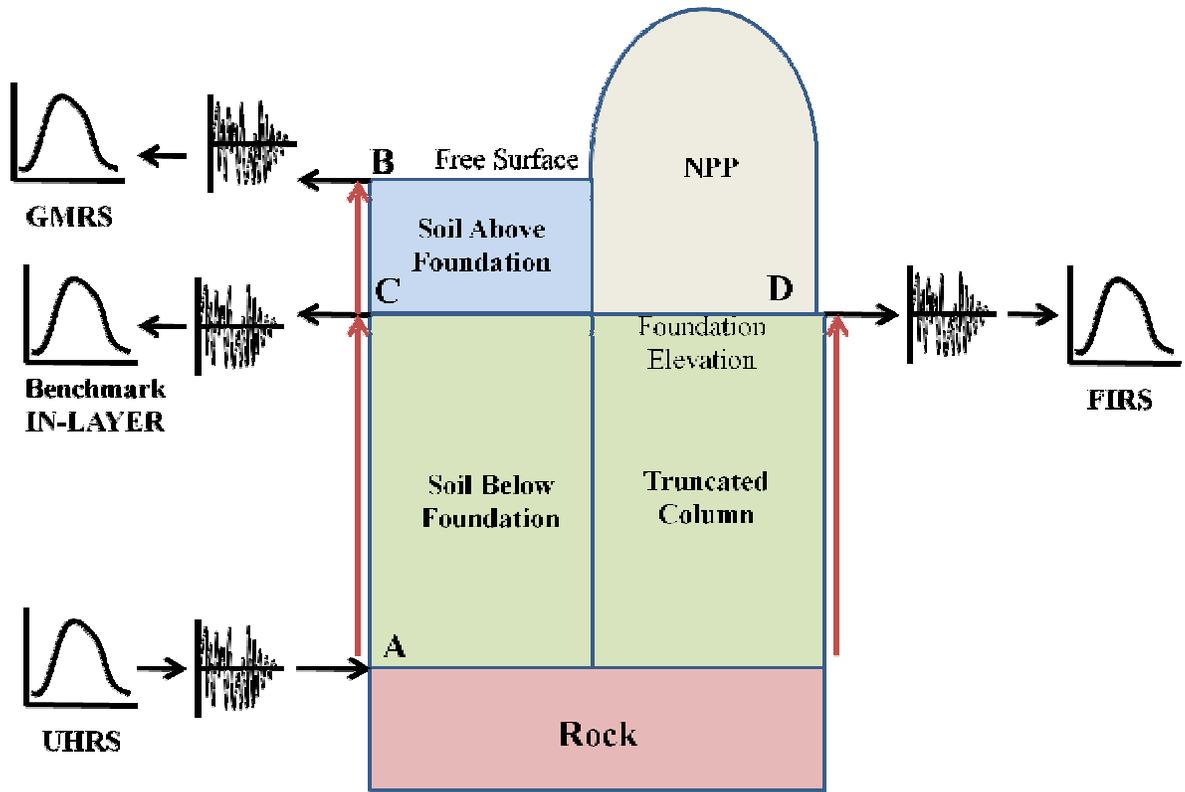
Figure 3.2: Generation of Surface Design Spectra and Benchmark In-layer Response

3.1.2 *Generation of SSI In-layer Inputs*

Next, the study uses the processes defined by the current-state-of-practice to develop the

SSI in-layer motion at C. First, RASCAL13 is used to fit an earthquake seed to the average

response of the 60 soil profiles from the truncated column response at D (the FIRS),

generating the truncated column surface design motion. This surface motion is then used as

the input motion at elevation D, or the top of the truncated column.

Starting with this artificial acceleration time history, the motion is deconvolved down to

the bedrock elevation A, generating an acceleration time history. This bedrock elevation

acceleration time history is used directly as the input time history at the base of the full

column. The time history at A is convolved up to the foundation elevation in-layer motion at C, generating an acceleration time history. This time history is then converted into a corresponding response spectrum. The resulting response spectrum is the current-state-of-practice SSI in-layer response. This process is illustrated by Figure 3.3.
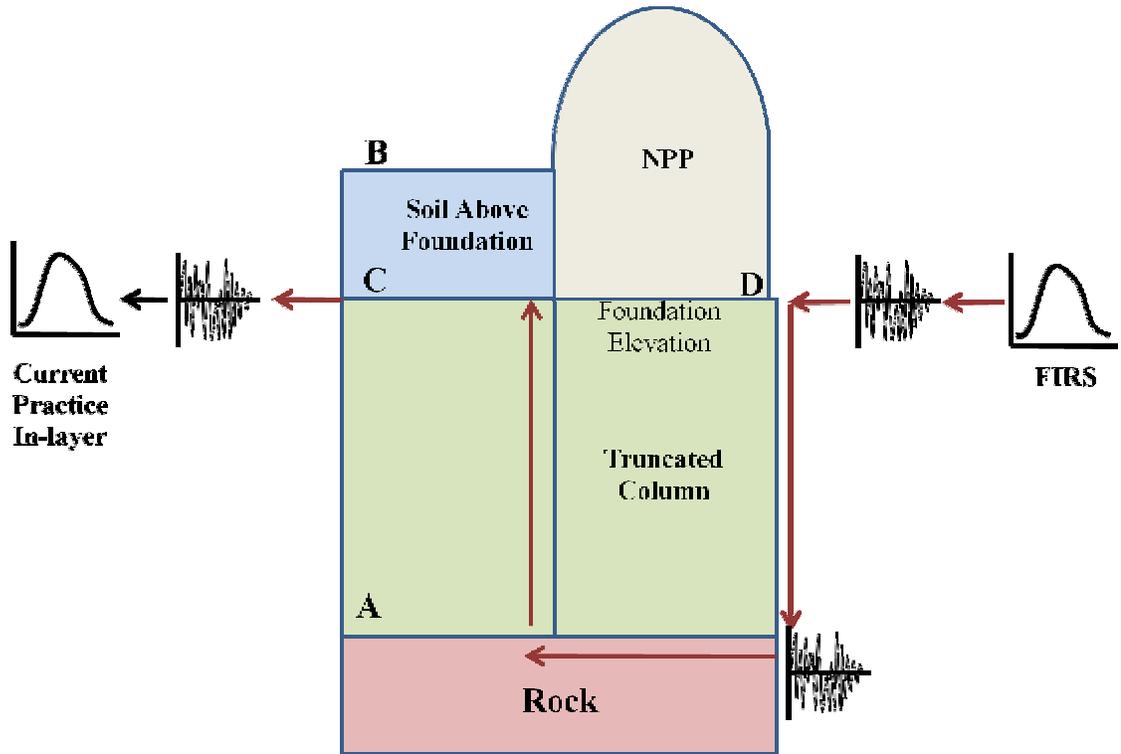
Figure 3.3: Generation of Current-State-of-Practice SSI In-layer Motion

Traditionally, the motion transfer in the current-state-of-practice is conducted using only the three BLU soil profiles, which are assumed to account for soil variability. To check the consistency of this assumption, the current-state-of-practice will be run using the three BLU soil profiles and the full suite of the original 60 soil profiles. Finally, the generated response spectra can be compared at the foundation elevation. Specifically, the current-state-of-

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practice SSI in-layer response should be consistent with the Benchmark response directly generated in Section 3.1.1.

3.2 Study Logistics and Parameters

In order to run the parametric study effectively, the analysis process is implemented in an automated suite of programs. The analysis depends upon three established programs: SHAKE91 for soil column response, RASCAL for creating artificial time histories from earthquake seeds and target spectra, and Nigam and Jennings Response Spectra piecewise exact method to transform ground acceleration time histories into response spectra. These three programs are interfaced together by one primary MATLAB file, FIRS.m. This MATLAB based program suite calls other MATLAB .m files, batch files, Excel files, and the RASCAL and SHAKE91 executables such that there is only pre- and post-processing required for an entire run. The required inputs are the soil profiles, earthquake seed acceleration time history, and the rock target response spectra, and some minor changes to text headers and two input files.

The program generates the surface design response spectra, benchmark in-layer response, and the current-state-of-practice SSI in-layer response. Additionally, the program can be used to determine the response for three different foundation elevations. In order to identify inconsistencies in the current-state-of-practice, the entire process (including the generation of design motions and the current practice responses), is run using three real-life soil suites taken from PSHAs from actual sites in the United States. The 60 soil profiles are the actual strain iterated soil data from a PSHA to define one specific site, and are not

generic soil profiles. The first soil suite is taken from a Western US site with typical deep soil profiles with several drastic velocity changes. The second soil suite is taken from a Eastern US deep soil with soil stiffness increasing with depth in a generally linear fashion. The third soil is altered from the Eastern US Soil to resemble that of a shallow, predominately linear, Midwestern US soil profile. The Best Estimate shear wave velocity profile for each soil is shown in Figure 3.4. The complete BLU soil profile shear wave velocities and damping values for each soil type are in Appendix B. Additionally, three foundation depths are considered, generally around 25, 50, and 75 ft. Table 3.1 shows the various study parameters examined in this thesis. The corresponding in-layer is the specific soil layers for each foundation depth, and the original earthquake seed is the 1994 Northridge Burbank-Howard Road Component 330. Other seed earthquakes were tested, with insignificant variability due to the fitting of the seed to the design spectra.

Table 3.1: Study Parameters

Soil	Depths (ft)			Corresponding Soil In-layer			Earthquake
Western US	25	45	75	4	6	9	Northridge
Eastern US	19	45	78	4	6	9	Northridge
Midwestern US	28	50	72	5	8	11	Northridge
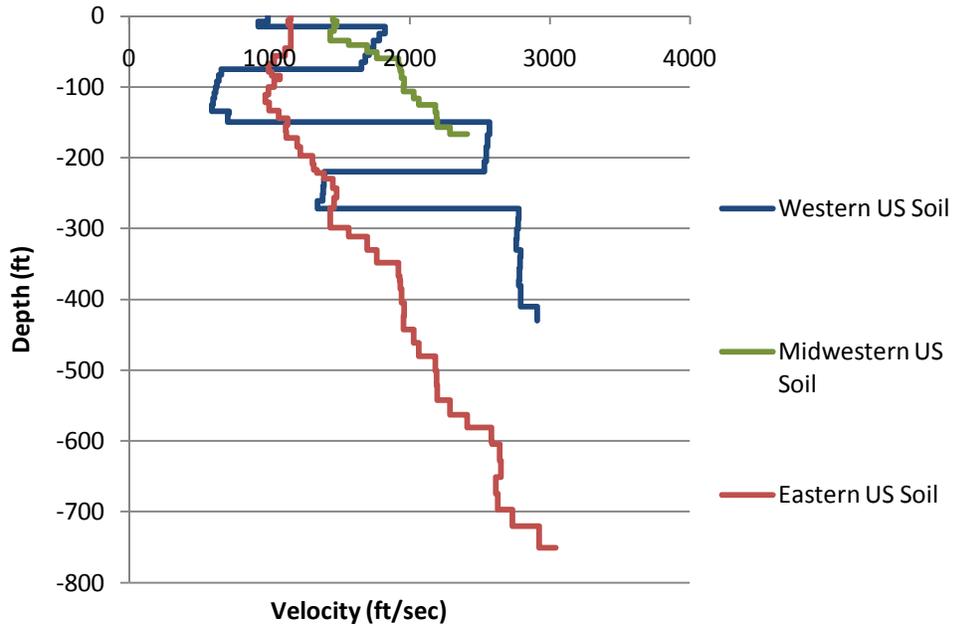
Figure 3.4: Best Estimate Soil Profile Shear Wave Velocities

3.3 Results

The study is designed to provide multiple scenarios for comparing the in-layer

Benchmark response to the response generated by the current-state-of-practice, with the goal

of identifying inconsistencies in the current-state-of-practice. Primary areas of focus

include the effect of reducing the number of soil profiles from 60 to three BLU soil profiles,

consistency at the foundation elevation in-layer response, and the identification of

inconsistency in the current-state-of-practice for generating SSI in-layer inputs.

3.3.1 *Average Responses from 60 Soil Profiles Versus BLU Soil Profiles*

Current regulatory documents express a concern with reducing the number of soil profiles from a full suite of soil profiles to the three BLU soil profiles. USNRC ISG 17 (2009) suggests that the reduction of soil profiles could be a source of inconsistency, but there is no recommendation to modify the current-state-of-practice that uses the three BLU soil profiles. To determine if the reduction of soil profiles is a potential source of inconsistency, the average response of the three BLU soil profiles is compared to the response generated when using the mean of all 60 soil profiles. Both the average response of the BLU soil profiles and the average response of the 60 soil profiles are calculated and can be compared at four elevations:

- The full column free-field surface (Elevation B)

- Benchmark foundation elevation in-layer (Elevation C)

- The truncated column surface (Elevation D)

- Bedrock response generated from deconvolving the design surface motions during the current-state-of-practice motion transfer procedure (Elevation A)

The average responses of the BLU soil profiles and the 60 soil profiles are plotted at each of these four locations for the Western US soil in Figure 3.5 through Figure 3.8.
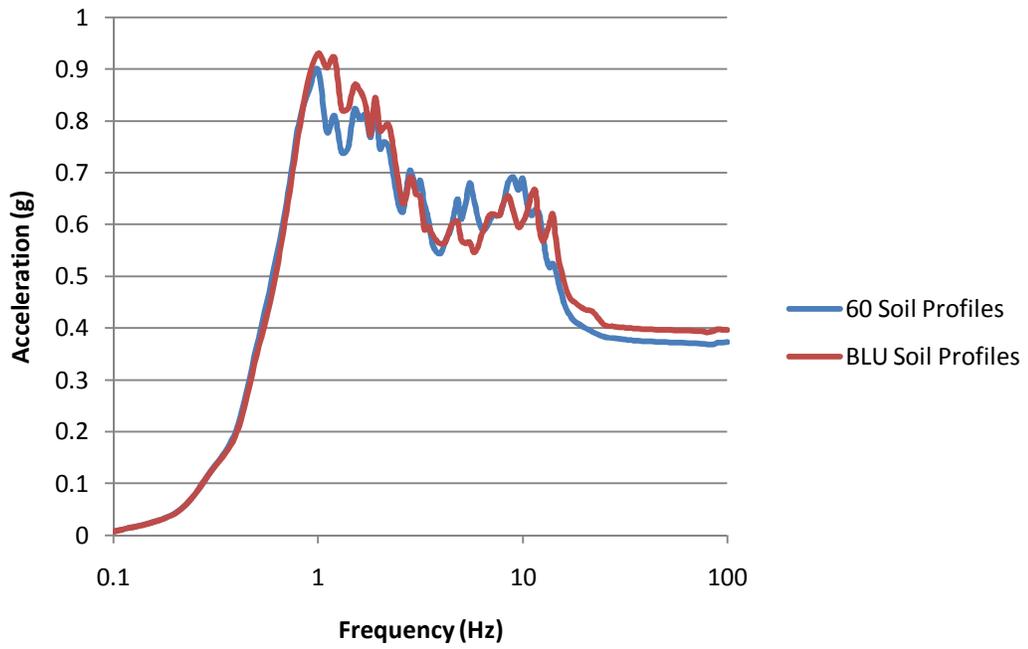
**Figure 3.5: Average Free-field Surface Response Spectra for Western Soil**
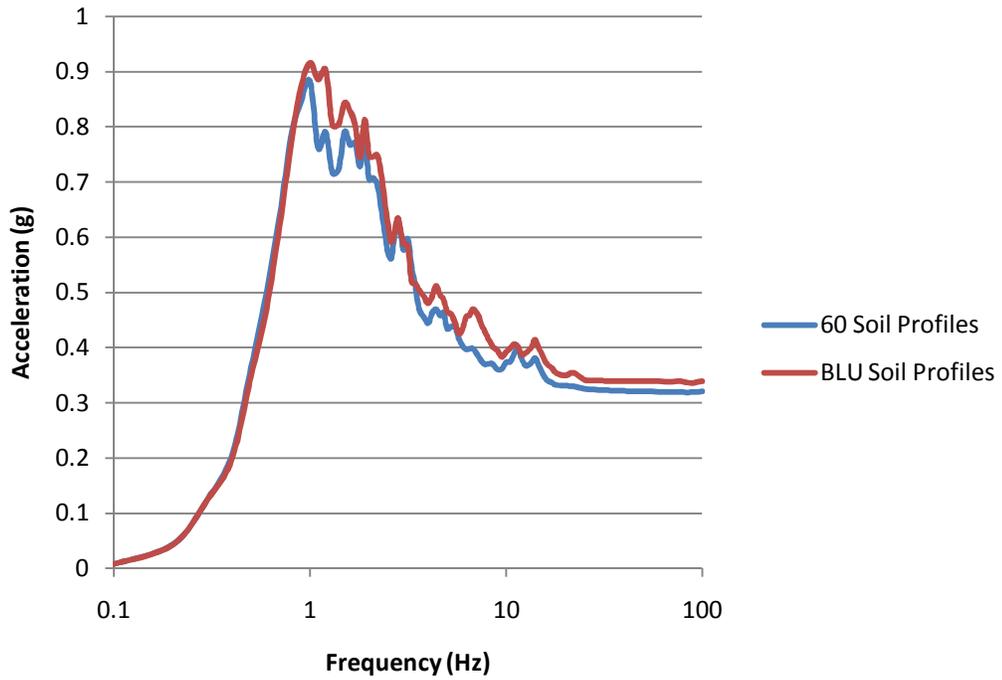
Figure 3.6: Average In-layer Response Spectra for Western Soil at the 50ft Foundation Elevation
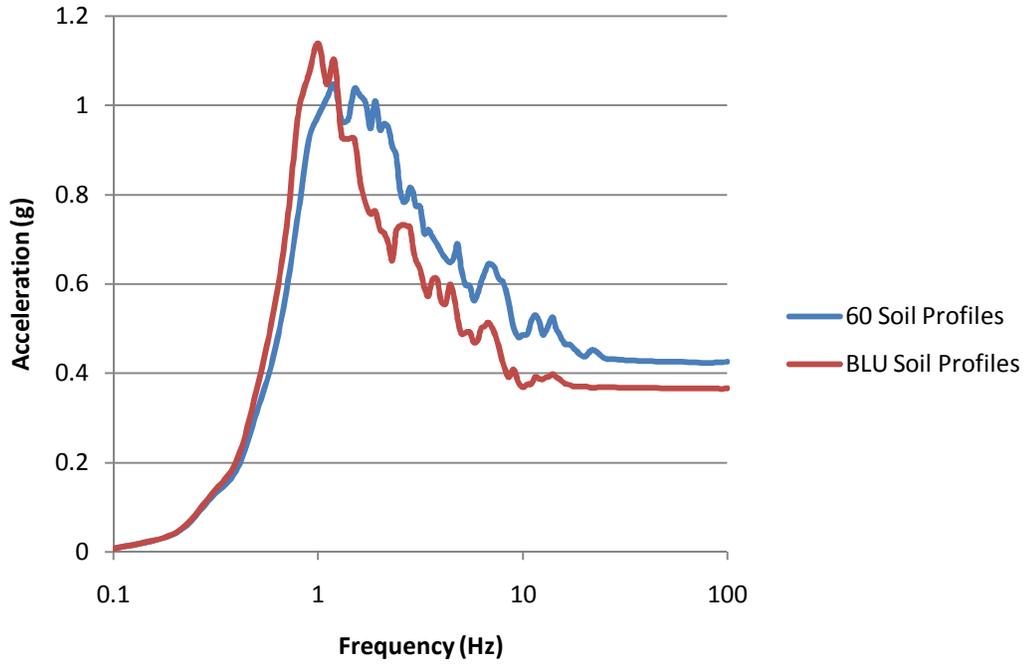
Figure 3.7: Average Truncated Surface Response Spectra for Western Soil at the 50ft Truncated Surface
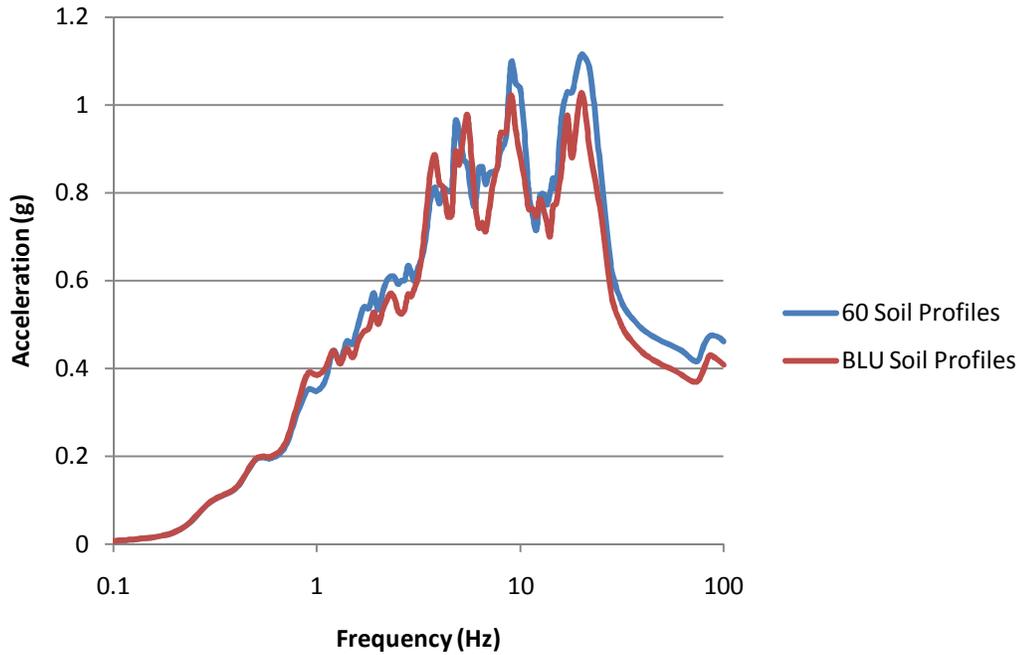
Figure 3.8: Average Bedrock Response Spectra for Western Soil deconvolved from 75ft Foundation Elevation

For the majority of the soil profiles and foundation elevations, there are negligible differences between the response from the BLU soil profiles and the 60 soil profiles. The two response spectra only differ significantly at the bedrock elevation developed by deconvolving the truncated column surface response for the Eastern and Western US soil profiles, shown in Figure 3.9 and Figure 3.10.
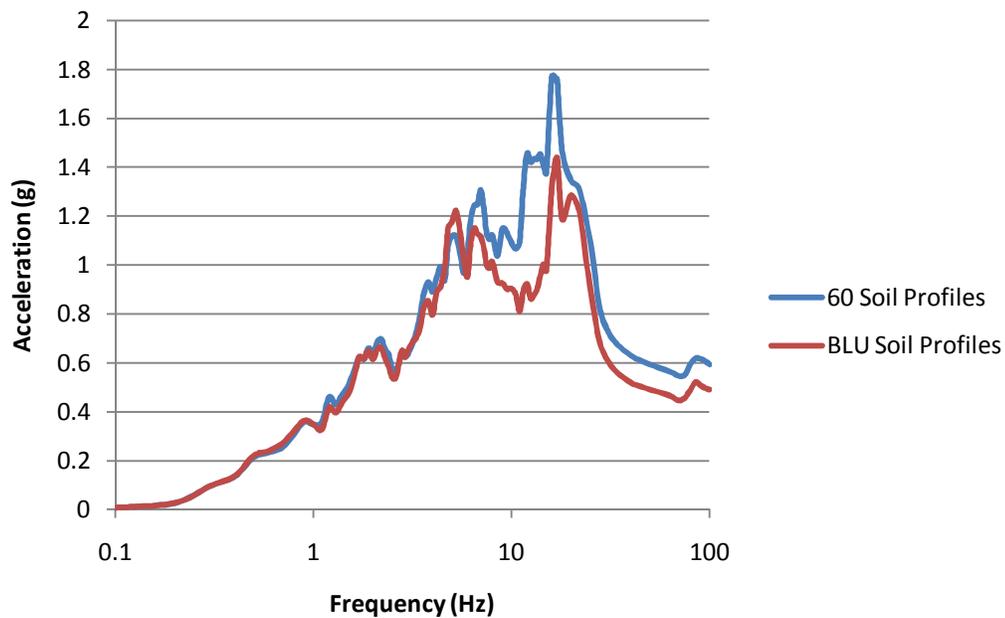
Figure 3.9: Bedrock Response Spectra for the Western Soil Deconvolved from the 25ft Truncated Foundation Elevation
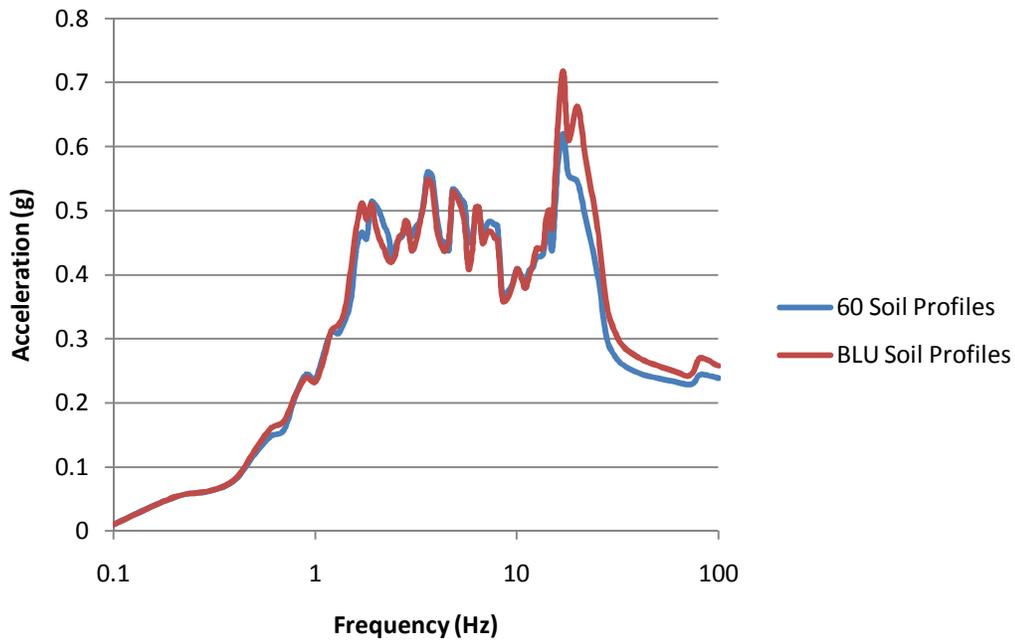
Figure 3.10: Bedrock Response Spectra for the Eastern Soil Deconvolved from the 25ft Truncated Foundation Elevation

3.3.2 *Consistency for Various Soils and Foundation Elevations*

The Benchmark in-layer foundation responses and the current-state-of-practice SSI input in-layer responses are compared for all three soil types and their respective foundation elevations. The response spectra are compared for both the magnitude of response and the frequency content of the response, as well as any trends based on soil depth.

Western US Soil Profile

The Western US soil has the greatest differences between the in-layer Benchmark response and the corresponding response from the current-state-of-practice. While the frequency content of the two responses are fairly consistent, the current practice response is

consistently higher at peak frequencies, as shown by Figure 3.11 and Figure 3.12. Additionally, the added response increases with depth, such that the 25 foot foundation elevation has the least error, and the 75 foot foundation elevation has the greatest error at the peak frequencies.

Eastern US Soil Profiles

The Eastern US soil also has substantial differences in the in-layer Benchmark response and the response from the current practice. Again, the frequency content is similar between the two responses, but the current practice consistently exceeds the Benchmark peak response, as shown by Figure 3.13 and Figure 3.14.

Midwestern US Soil Profile

The Midwestern US soil has the least variation between the in-layer Benchmark and the current practice responses. Additionally, it is the only case where the Benchmark exceeded the current practice response. While the current practice response is higher than the Benchmark in-layer response at the 25 foot foundation elevation, it falls below the Benchmark response by the 75 foot foundation elevation, as shown in Figure 3.15 and Figure 3.16.

All three soil types have noticeable differences between the in-layer Benchmark response and the current practice response. These differences can be upwards of 23% of peak response (Western US at 75 foot Elevation) or be under peak response by 6% (Midwestern at 75 foot Elevation).
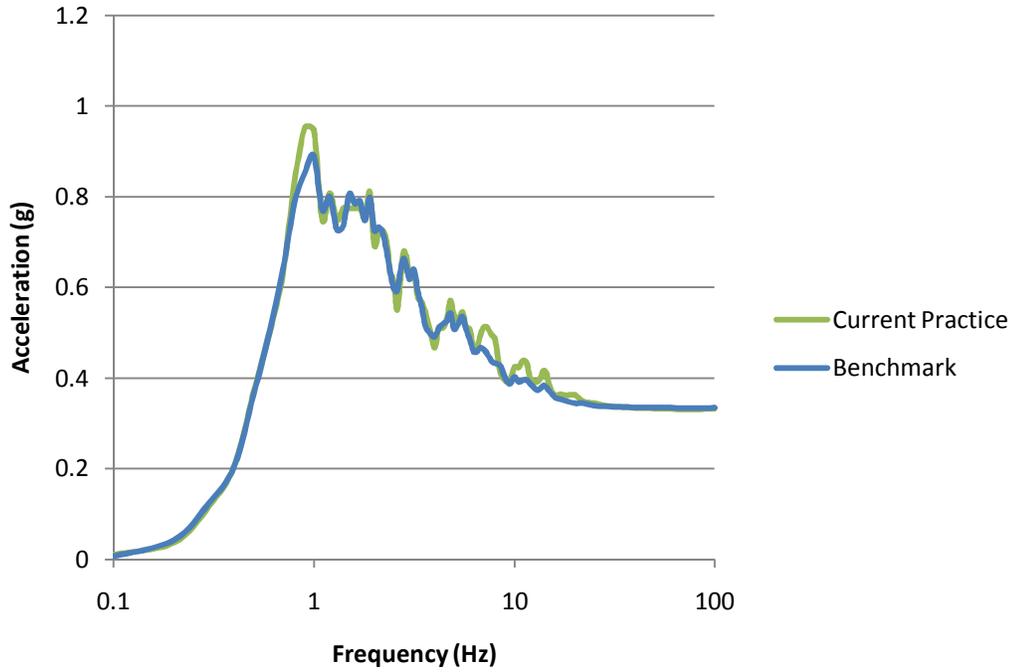
Figure 3.11: In-layer Response Spectra for the Western Soil at the 25ft Foundation Elevation
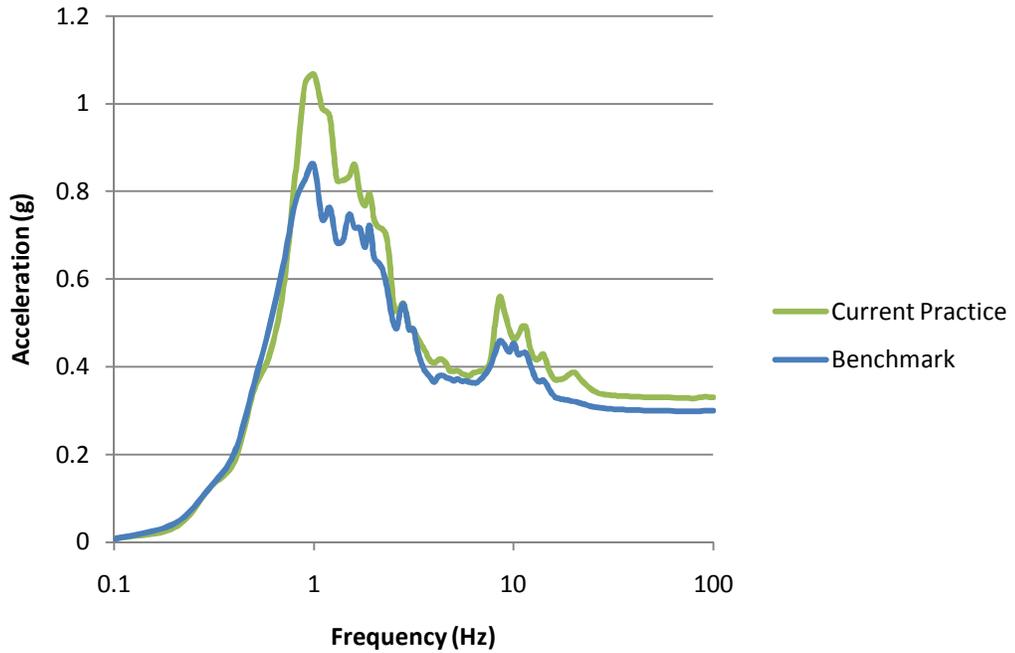
Figure 3.12: In-layer Response Spectra for the Western Soil at the 75ft Foundation Elevation
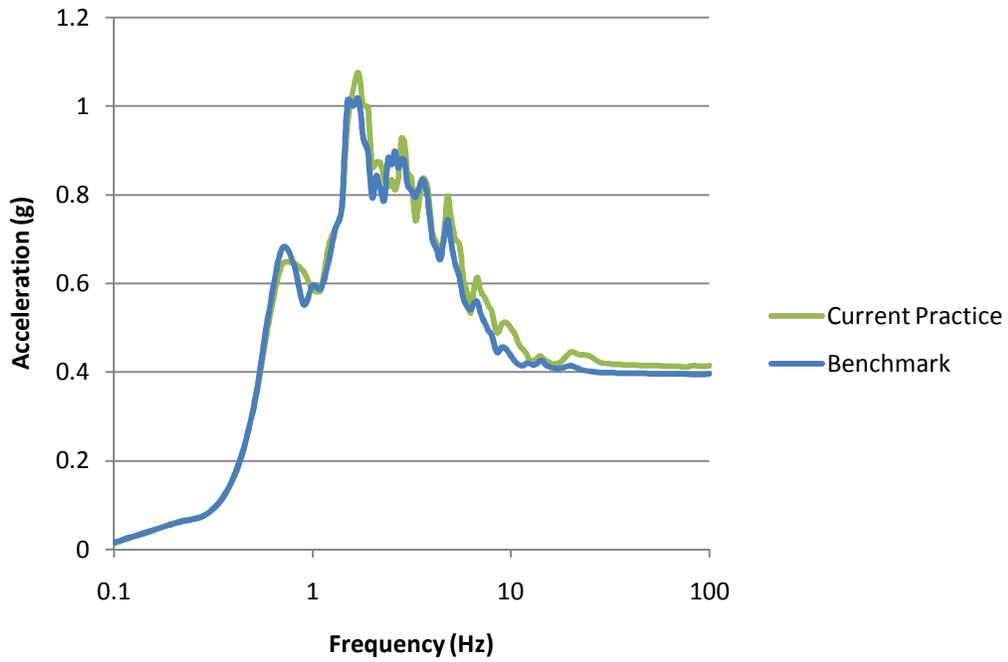
Figure 3.13: In-layer Response Spectra for the Eastern Soil at the 25ft Foundation Elevation
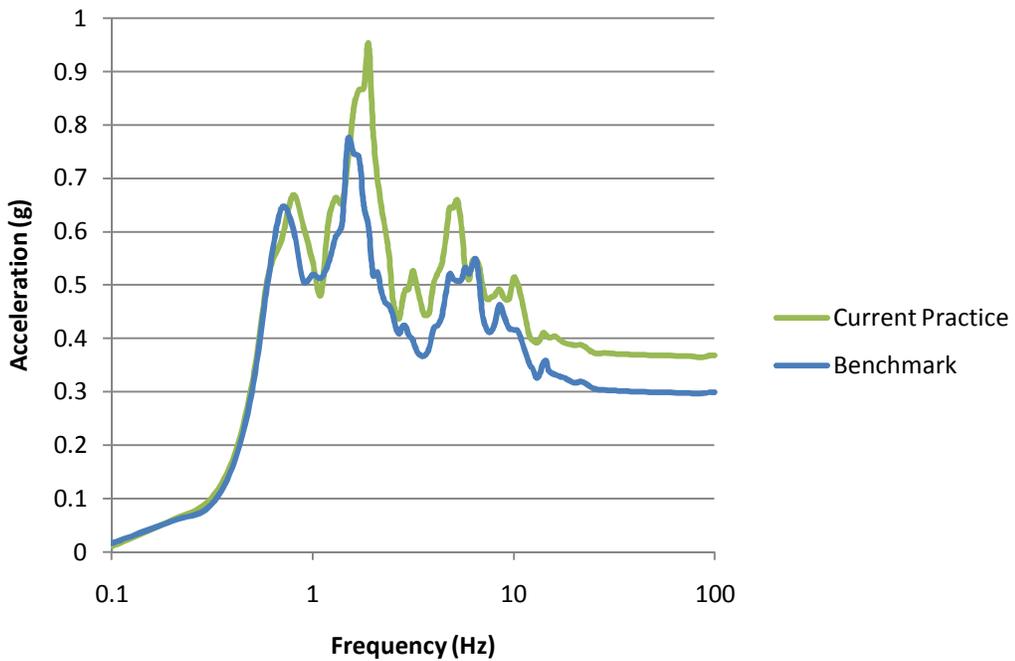
Figure 3.14: In-layer Response Spectra for the Eastern Soil at the 75ft Foundation Elevation
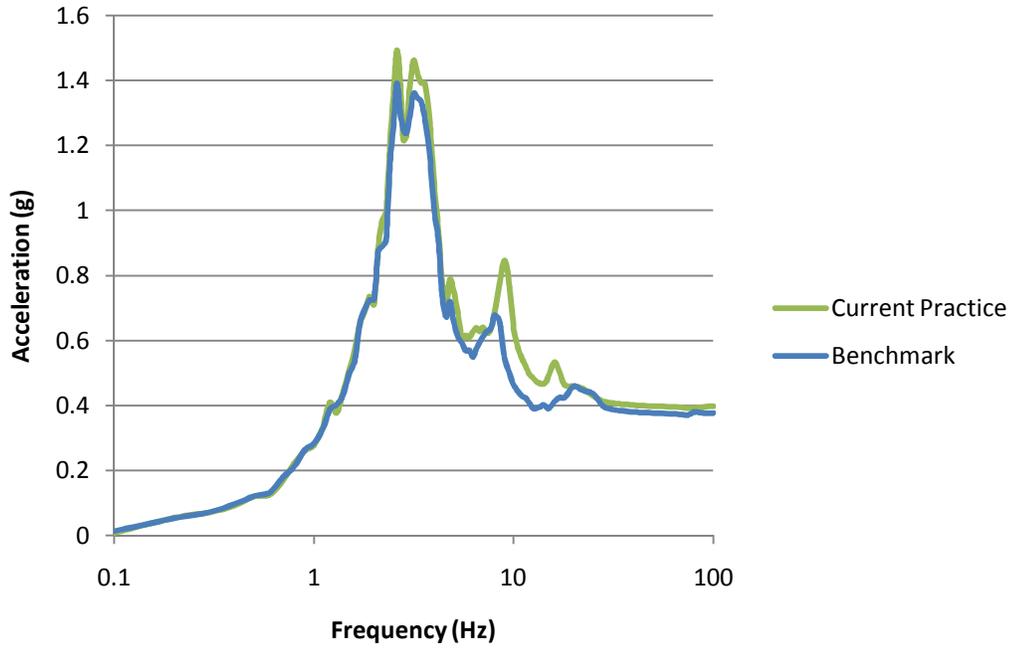
**Figure 3.15: In-layer Response Spectra for the Midwestern Soil at the 25ft Foundation Elevation**
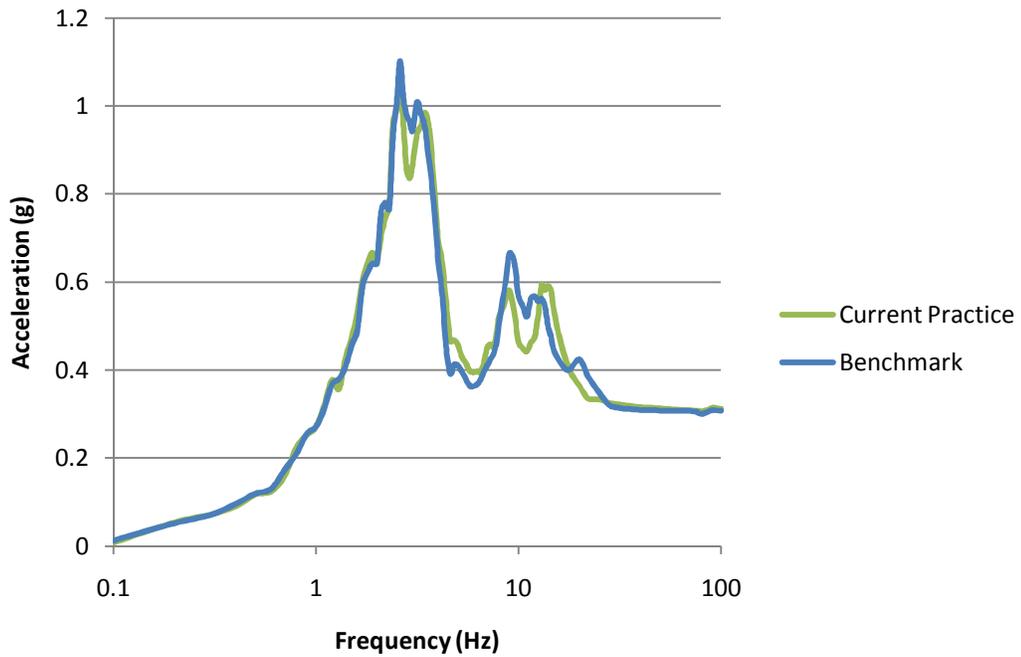
Figure 3.16: In-layer Response Spectra for the Midwestern Soil at the 75ft Foundation Elevation

3.3.3 *Bedrock Motions*

Having identified that the current-state-of-practice is not consistent in transferring motions, the inconsistency in response is first seen at the bedrock level. In order to develop SSI input motions, motions at the surface of a soil column are transferred down to the bedrock and can be compared to the original bedrock target spectrum. For the Western US soil, shown in Figure 3.17, there is substantial acceleration gain at the bedrock elevation across a wide frequency band. This increased bedrock response when used to propagate up the full soil column leads to an overestimation for the foundation elevation in-layer motion. The Eastern US soil also has response gain at the bedrock, although substantially less than the Western US soil, as shown in Figure 3.18. The Midwestern US soil response, shown in Figure 3.19, has the best match between the original input and the deconvolved response, which helps explain the decreased error at the in-layer elevation.

There are a few additional observations regarding the bedrock motion.

- When the free-field surface motion is deconvolved to the bedrock elevation, there is an increased response compared to the original input. This shows that the added response is not only found in truncated soil columns.

- For the relatively accurate case of the Midwestern US soil, there are small frequency spikes that are likely a result of the key frequencies of the soil column. These small spikes are present in both the response from the BLU soil profiles

and the average response of the 60 profiles, again showing consistency when using the BLU soil profiles.
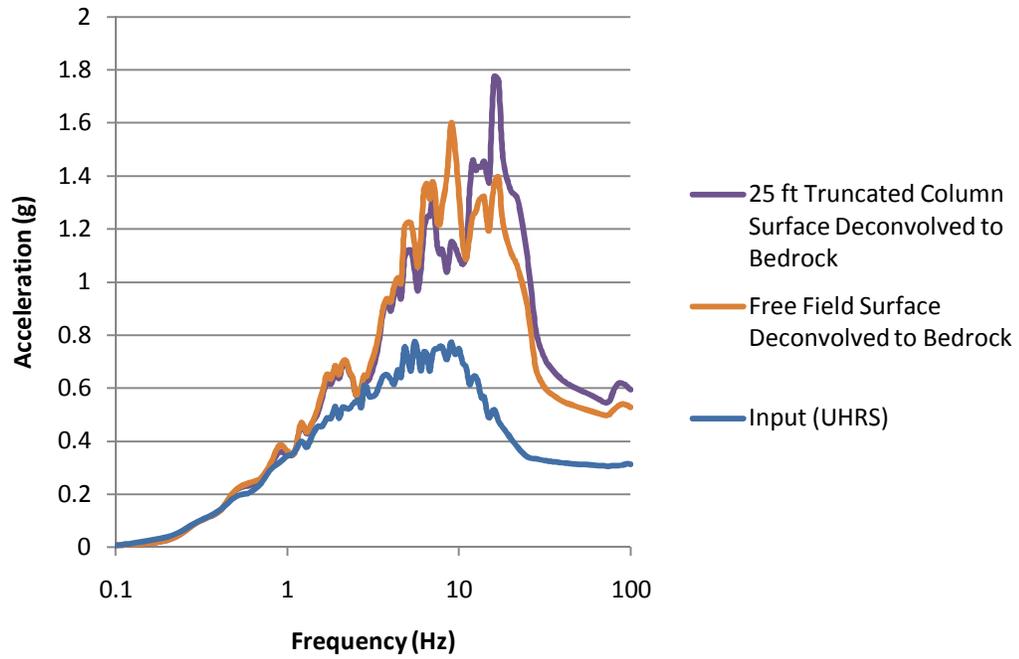
Figure 3.17: Bedrock Response Spectra for the Western Soil
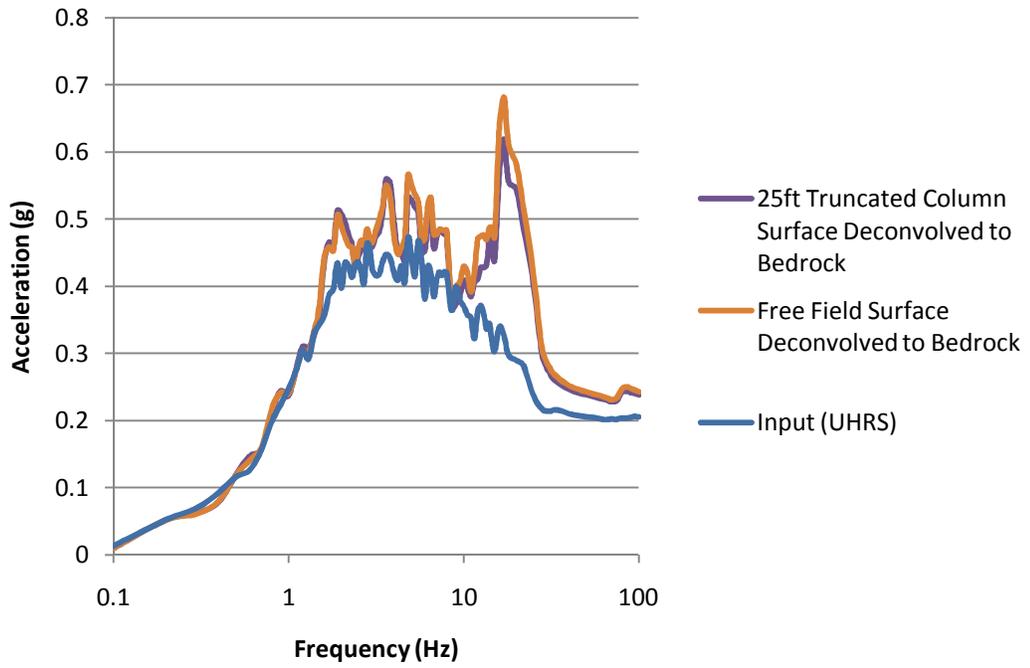
Figure 3.18: Bedrock Response Spectra for the Eastern Soil
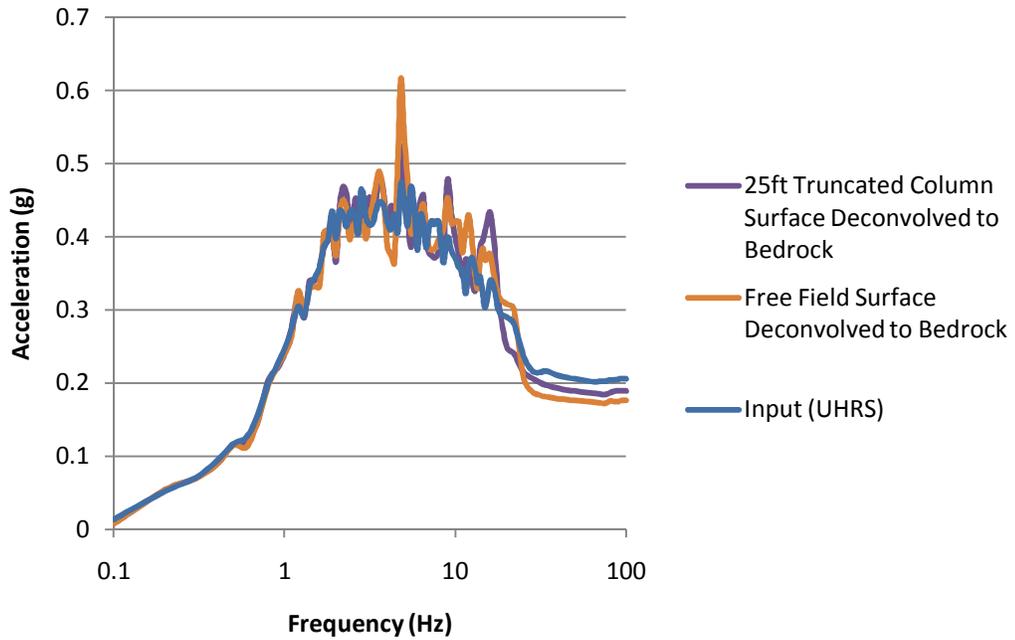
Figure 3.19: Bedrock Response Spectra for the Midwestern Soil

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3.4 Summary of Key Observations

In summation, after completing this study there are several observations and conclusions.

- The average response of the three BLU soil profiles usually matches the average response of 60 soil profiles. While there are concerns with compatibility using the BLU soil profiles, this study did not identify any significant issues with using the BLU soil profiles.

- The response for all three soils types had instances where the current-state-of-practice overestimated or underestimated the Benchmark response across the spectral frequencies or at peak acceleration. This error ranged from 23% to -6% at peak frequencies across the three soil suites.

- The inconsistency in the current-state-of-practice first occurs at the bedrock level, which in turn affects the consistency of the in-layer responses.

CHAPTER 4: ANALYSIS USING A THEORETICAL SYSTEM

The study in Chapter 3 used a typical earthquake soil response program, SHAKE91, which is based on vertical wave propagation of shear waves through a linear system of soil profiles. Each of the n soil layers are defined by a thickness h, mass density ρ, shear modulus, G, and damping factor, β. They extend infinitely in the horizontal direction, and overlay a uniform halfspace bedrock layer. Although the system allows for infinite layers, we will consider a two layer system over a halfspace to isolate and identify specific reasons that contribute to inconsistency in the current-state-of-practice. An n soil layer system overlaying a bedrock halfspace is shown Figure 4.1, with local axis systems defined at the surface of each layer.
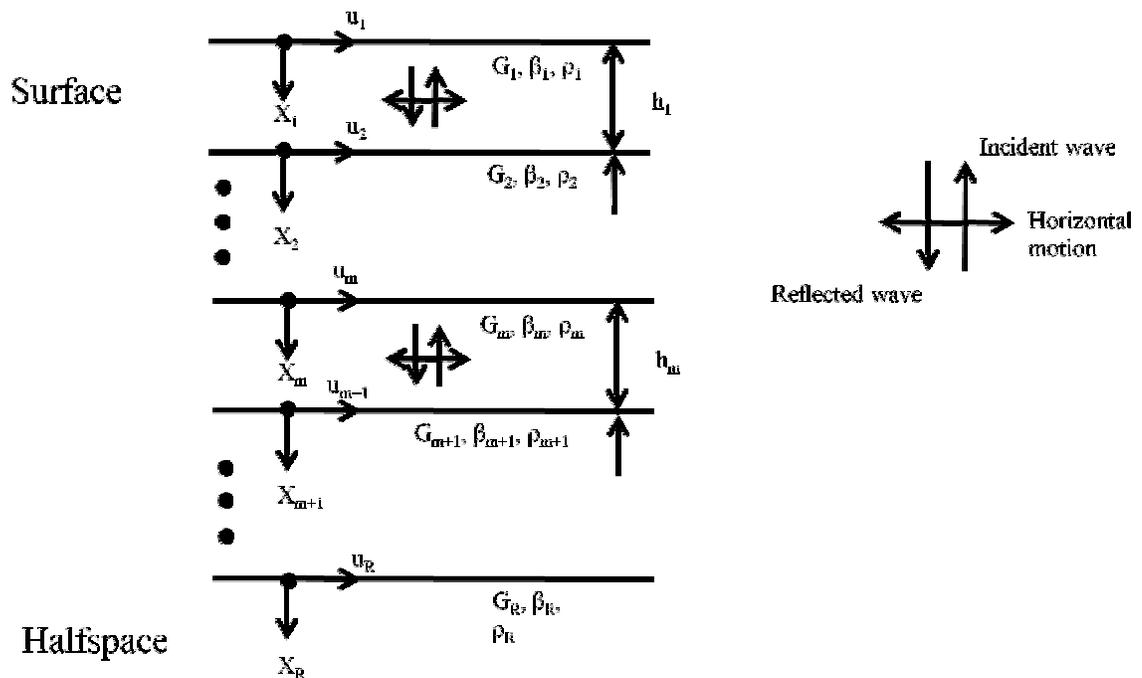
Figure 4.1: n Layer 1-D Soil System

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4.1 Wave Propagation

For the horizontal displacement u at any location x and time t,

$$u = u(x, t) \qquad \textbf{4.1}$$

Vertical shear waves produce primarily horizontal displacements that must satisfy the wave equation in terms of density, shear modulus, and viscosity η:

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial x^2} + \eta \frac{\partial^3 u}{\partial x^2 \partial t} \qquad \textbf{4.2}$$

Assuming displacements to be harmonic with frequency ω, Equation 4.1 can be written as:

$$u(x, t) = U(x)e^{i\omega t} \qquad \textbf{4.3}$$

Substituting Equation 4.3 into Equation 4.2 we get an ordinary differential equation

$$(G + i\omega\eta) \frac{\partial^2 U}{\partial x^2} = \rho \omega^2 U \qquad \textbf{4.4}$$

The solution of Equation 4.4 can be written as

$$U(x) = E e^{ikx} + F e^{-ikx} \qquad \textbf{4.5}$$

Where k is the complex wave number and can be defined by in terms of G^*, the complex shear modulus.

$$k^2 = \frac{\rho \omega^2}{G + i\omega\eta} = \frac{\rho \omega^2}{G^*} \qquad \textbf{4.6}$$

For most soils, G and β, the shear modulus and the critical damping ratio, are nearly constant across the predominant frequency range. The viscosity η is related to G^* and β

according to the following relationship, which allows the definition of G^* independent of frequency.

$$\omega\eta = 2G\beta$$

$$G^* = G(1 + 2i\beta) \qquad\qquad \textbf{4.7}$$

Substituting Equation 4.5 into Equation 4.3 yields:

$$u(x,t) = Ee^{i(kx+\omega t)} + Fe^{-i(kx-\omega t)} \qquad\qquad \textbf{4.8}$$

The first term in the above equation with the positive exponent, $Ee^{ikx+\omega t}$, represents the incident wave travelling upward and the second term with the negative exponent, $Fe^{-i(kx-\omega t)}$, represents the reflected wave traveling downward. The coefficients E and F are the amplitudes of the displacement equation.

If a local coordinate system is introduced with the downward direction as positive, then the displacement at the top and bottom of any layer m can be written as:

$$u_m(X = 0) = (E_m + F_m)e^{i\omega t} \qquad\qquad \textbf{4.9}$$

$$u_m(X = h_m) = (E_m e^{ik_m h_m} + F_m e^{-ik_m h_m})e^{i\omega t} \qquad\qquad \textbf{4.10}$$

The shear stress on a horizontal plane can be written in terms of Equation 4.8, where

$$\tau(x,t) = G\frac{du}{dx} + \eta\frac{du}{dxdt} = G^*\frac{du}{dx} \qquad\qquad \textbf{4.11}$$

$$\tau(x,t) = ikG^*(Ee^{ikx} - Fe^{-ikx})\, e^{i\omega t} \qquad\qquad \textbf{4.12}$$

and can be solved at the top and bottom of layer m in the same form as Equations 4.9 and 4.10:

$$\tau_m(X = 0) = ik_m G_m^*(E_m - F_m)e^{i\omega t} \tag{4.13}$$

$$\tau_m(X = h_m) = ik_m G_m^*(E_m e^{ik_m h_m} - F_m e^{-ik_m h_m}) e^{i\omega t} \tag{4.14}$$

Both stresses and displacements must be equal between each layer, such that

$u_m(X = h_m) = u_{m+1}(X = 0)$ and $\tau_m(X = h_m) = \tau_{m+1}(X = 0)$. By using Equations 4.9, 4.10, 4.13, and 4.14, E_{m+1} and F_{m+1} can be written as:

$$E_{m+1} = \frac{1}{2}[E_m(1 + \alpha_m)e^{ik_m h_m} + F_m(1 - \alpha_m)e^{i-k_m h_m}] \tag{4.15}$$

$$F_{m+1} = \frac{1}{2}[E_m(1 - \alpha_m)e^{ik_m h_m} + F_m(1 + \alpha_m)e^{i-k_m h_m}] \tag{4.16}$$

Where α_m is the complex impedance ratio,

$$\alpha_m = \frac{k_m G_m^*}{k_{m+1} G_{m+1}^*} = \left(\frac{\rho_m G_m^*}{\rho_{m+1} G_{m+1}^*}\right)^{1/2} \tag{4.17}$$

Applying boundary conditions, the shear stress at the surface must be zero, requiring that

$$E_1 = F_1 \tag{4.18}$$

Equation 4.18 also means that the incident and reflected waves are equal at a free surface.

By starting at the surface and solving layer by layer for Equations 4.15 and 4.16, the following relationships between the amplitudes at any layer m and the surface layer can be developed:

$$E_m = e_m E_1 e^{i\omega t} \tag{4.19}$$

$$F_m = f_m E_1 e^{i\omega t} \tag{4.20}$$

Where e_m and f_m are transfer functions that can be determined by using the recursion equations 4.15 and 4.16. Additionally, the transfer functions can be used to generate displacement transfer functions, where $A_{n,m}$ can be used to define displacements at layer m relative to layer n by:

$$A_{n,m}(\omega) = \frac{u_m}{u_n} = \frac{e_m + f_m}{e_n + f_n} \tag{4.21}$$

Acceleration can be derived from the displacement function in Equation 4.3, resulting in the following equation:

$$\ddot{u}(x,t) = \frac{\partial^2 u}{\partial t^2} = -\omega^2 \left(E e^{i(kx+\omega t)} + F e^{-i(kx-\omega t)} \right) \tag{4.22}$$

4.2 Solution for 2-Layer System

In order to develop response spectra for various soil profiles, it is necessary to determine the amplitude at various frequencies. The displacement and accelerations are related by ω for harmonic motion, thus the amplitude, or the values of E and F at each layer, defines the peak response at each frequency. In order to determine any trends or inconsistencies in a theoretical system, the solution for a two soil layer system, as shown in Figure 4.2, is used to isolate variations that may be masked in a larger soil profile.
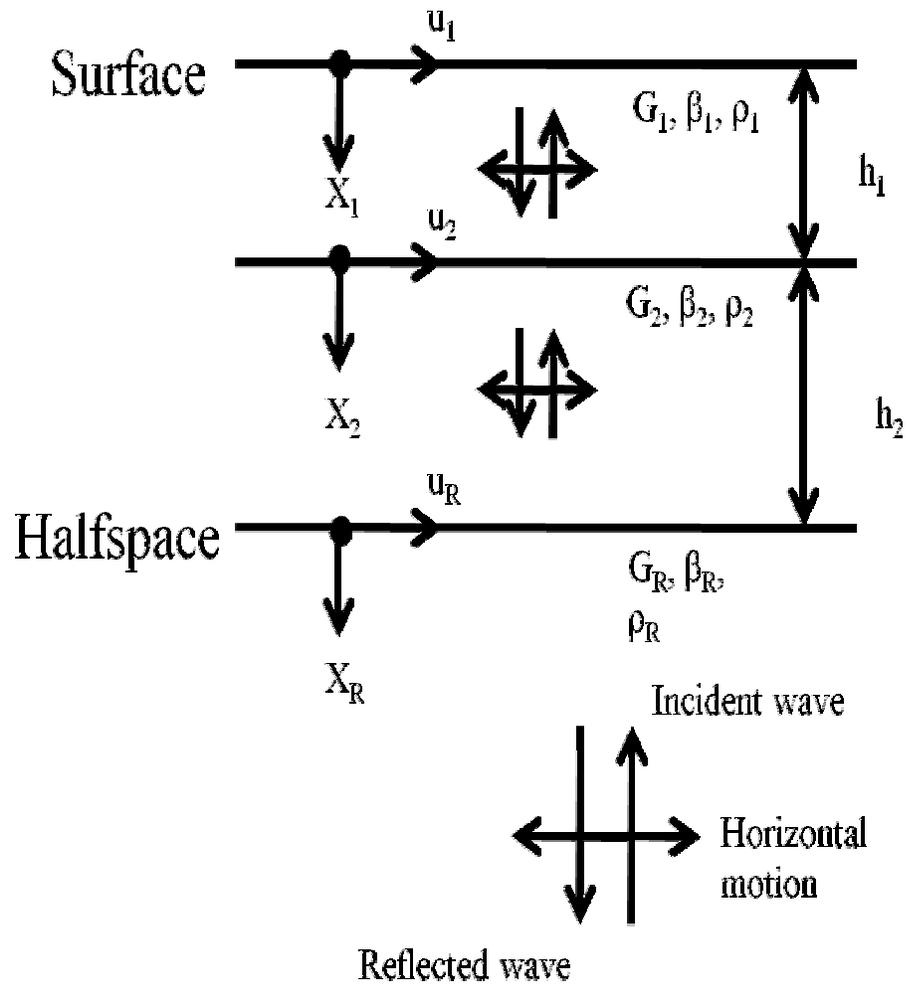
Figure 4.2: 2-Layer Theoretical Soil Profile

To develop a closed-form solution for a 2-layer system, the equation of motion is found at the free surface by using Equation 4.9 with the applied boundary condition from Equation 4.18. The amplitude coefficients representing the two directional waves can be solved for each subsequent layer using Equations 4.15 and 4.16 at each elevation. The equation of motion for the top layer is solved as:

$$u_1 = (E_1 + F_1)e^{i\omega t} = 2E_1 e^{i\omega t} \tag{4.23}$$

Using Equation 4.9, the amplitude coefficients and displacements can be defined at the top of the second layer where:

$$E_2 = \frac{1}{2}\left[E_1(1 + \alpha_1)e^{ik_1h_1} + F_1(1 - \alpha_1)e^{-ik_1h_1}\right]$$

$$\tag{4.24}$$

$$F_2 = \frac{1}{2}\left[E_1(1 - \alpha_1)e^{ik_1h_1} + E_1(1 + \alpha_1)e^{-ik_1h_1}\right]$$

$$u_2 = (E_2 + F_2)e^{i\omega t}$$

$$u_2 = \frac{1}{2}\left[(E_1)e^{ik_1h_1}(2 + \alpha_1 - \alpha_1) + F_1 e^{-ik_1h_1}(2 + \alpha_1 - \alpha_1)\right]e^{i\omega t} \tag{4.25}$$

$$u_2 = \left(E_1 e^{ik_1h_1} + F_1 e^{-ik_1h_1}\right)e^{i\omega t}$$

The displacement u_2 can be compared to Equation 4.9, and the coefficients E_2 and F_2 can be solved as:

$$E_2 = E_1 e^{ik_1h_1}$$

$$\tag{4.26}$$

$$F_2 = F_1 e^{-ik_1h_1}$$

Again, Equations 4.15, 4.16, and 4.9 can be used to determine the displacements at the top of the bedrock layer by solving for the corresponding coefficients E_R and F_R

$$E_R = \frac{1}{2}\left[E_2(1 + \alpha_2)e^{ik_2h_2} + F_2(1 - \alpha_2)e^{-ik_2h_2}\right]$$

<div align="right">4.27</div>

$$F_R = \frac{1}{2}\left[E_2(1 - \alpha_2)e^{ik_2h_2} + F_2(1 + \alpha_2)e^{-ik_2h_2}\right]$$

$$u_R = (E_R + F_R)e^{i\omega t}$$

$$u_R = \left(E_2e^{ik_2h_2} + F_2e^{-ik_2h_2}\right)e^{i\omega t}$$

<div align="right">4.28</div>

$$u_R = \left(E_1e^{i(k_1h_1+k_2h_2)} + F_1e^{-i(k_1h_1+k_2h_2)}\right)e^{i\omega t}$$

$$E_R = E_1e^{i(k_1h_1+k_2h_2)}$$

<div align="right">4.29</div>

$$F_R = F_1e^{-i(k_1h_1+k_2h_2)}$$

The displacements at the top of the bottom layer and the top of the bedrock can be simplified in terms of E_1 by applying the boundary condition from Equation 4.18, where

$$u_2 = E_1\left(e^{ik_1h_1} + e^{-ik_1h_1}\right)e^{i\omega t}$$

<div align="right">4.30</div>

$$u_R = E_1\left(e^{i(k_1h_1+k_2h_2)} + e^{-i(k_1h_1+k_2h_2)}\right)e^{i\omega t}$$

<div align="right">4.31</div>

If the displacements found in Equations 4.23, 4.30, and 4.31 are compared to the form of the amplitudes found in Equations 4.19 and 4.20, then the transfer functions e_m and f_m, can be solved at each elevation as

$$e_1 = 1, f_1 = 1$$

<div align="right">4.32</div>

$$e_2 = e^{ik_1h_1}, f_2 = e^{-ik_1h_1}$$

<div align="right">4.33</div>

$$e_R = e^{i(k_1 h_1 + k_2 h_2)}, f_R = e^{-i(k_1 h_1 + k_2 h_2)} \qquad \text{4.34}$$

Using Euler's Formula for complex analysis, the terms e^{ix} and e^{-ix} can be written as

$$e^{ix} = \cos x + i \sin x$$

$$e^{-ix} = \cos x - i \sin x \qquad \text{4.35}$$

$$e^{ix} + e^{-ix} = 2 \cos x$$

For the determination of transfer functions such as Equation 4.21, the coefficients found in Equations 4.32, 4.33, and 4.34 can be written as

$$e_1 + f_1 = 2$$

$$e_2 + f_2 = 2 \cos(k_1 h_1) \qquad \text{4.36}$$

$$e_R + f_R = 2 \cos(k_1 h_1 + k_2 h_2)$$

In order to solve without complex forms, it is necessary to rewrite the solution in terms of shear wave velocity v_s, damping ratio ξ, and natural frequency ω. Using soil relationships found in Kramer (1996), the following simplifications can be made.

$$v_s = \sqrt{\frac{G}{\rho}} \qquad \text{4.37}$$

Similarly,

$$v_s^* = \sqrt{\frac{G^*}{\rho}} \qquad \text{4.38}$$

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As per Equation 4.7, the complex shear modulus can be removed from the solution by using shear wave velocity, calculated as:

$$v_s^* = \sqrt{\frac{G^*}{\rho}} = \sqrt{\frac{G(1 + 2i\xi)}{\rho}}$$

4.39

$$v_s^* = v_s(1 + i\xi)$$

$$k^* = k(1 - i\xi)$$

4.40

$$k = \frac{\omega}{v_s} \therefore k^* = \frac{\omega}{v_s}(1 - i\xi)$$

For example, using the transfer function coefficients from Equation 4.36 for the second layer, the sum of the transfer functions can be written as:

$$e_2 + f_2 = 2\cos(k_2 h_2(1 - i\xi))$$

4.41

Using the identity $\cos(x + iy) = \sqrt{\cos^2 x + \sinh^2 y}$ where $\sinh^2 y \approx y^2$ for small y, Equation 4.41 can be rewritten as

$$e_2 + f_2 = 2\sqrt{\cos^2 k_1 h_1 + (k_1 h_1 \xi)^2}$$

4.42

$$e_2 + f_2 = 2\sqrt{\cos^2\left(\omega h_1 / v_{s1}\right) + \left(\omega\left(\xi h_1 / v_{s1}\right)\right)^2}$$

Similarly, $e_R + f_R$ from Equation 4.36 can be written as:

$$e_R + f_R = 2cos[k_1 h_1(1 - i\xi_1) + k_2 h_2(1 - i\xi_2)]$$

$$e_R + f_R = 2\,cos[(k_1 h_1 + k_2 h_2) \cdot 1 - (k_1 h_1 \xi_1 + k_2 h_2 \xi_2)i]$$

$$e_R + f_R = 2\sqrt{cos^2(k_1 h_1 + k_2 h_2) + [(k_1 h_1 \xi_1 + k_2 h_2 \xi_2)\omega]^2} \qquad \text{4.43}$$

$$e_R + f_R = 2\sqrt{cos^2\omega \left(\frac{k_1}{v_{s1}} + \frac{h_2}{v_{s2}}\right) + \left[\omega \left(\frac{k_1 \xi_1}{v_{s1}} + \frac{k_2 \xi_2}{v_{s2}}\right)\right]^2}$$

Using the same form for the two layer solution, for any soil profile with n layers, the coefficient $e_m + f_m$ for layer m (where $m \le n$) can be found as:

$$e_m + f_m = 2\sqrt{cos^2\omega \sum_{i=1}^{m-1}\left(\frac{h}{v_s}\right)_i + \left[\omega \sum_{i=1}^{m-1}\left(\frac{h\xi}{v_s}\right)_i\right]^2} \qquad \text{4.44}$$

For the two layer case, two displacement transfer functions for transforming a bedrock amplitude to the surface of layers 1 and 2 can be determined such that:

$$A_{R,1} = \frac{e_1 + f_1}{e_R + f_R} \qquad \text{4.45}$$

and

$$A_{R,2} = \frac{e_2 + f_2}{e_R + f_R} \qquad \text{4.46}$$

Substituting in Equations 4.36, 4.42, and 4.43, the two displacement transfer functions defined by Equations 4.45 and 4.46 can be formed:

$$A_{R,1} = \frac{1}{\sqrt{\cos^2\omega \left(k_1/v_{s1} + h_2/v_{s2}\right) + \left[\omega \left(k_1\xi_1/v_{s1} + k_2\xi_2/v_{s2}\right)\right]^2}}$$

4.47

$$A_{R,2} = \sqrt{\frac{\cos^2\left(\omega h_1/v_{s1}\right) + \left(\left(\omega h_1/v_{s1}\right)\xi\right)^2}{\cos^2\omega \left(k_1/v_{s1} + h_2/v_{s2}\right) + \left[\omega \left(k_1\xi_1/v_{s1} + k_2\xi_2/v_{s2}\right)\right]^2}}$$

4.48

Additionally, if layer 1 of the soil is assumed to be removed creating a free surface at the top of layer 2, the displacement transfer function for the truncated column can be defined as:

$$A_{R,F} = \frac{e_2' + f_2'}{e_R' + f_R'}$$

4.49

Where,

$$e_2' + f_2' = 2$$

4.50

$$e_R' + f_R' = 2\sqrt{\cos^2\left(\omega h_2/v_{s2}\right) + \left(\omega \left(\xi h_2/v_{s2}\right)\right)^2}$$

Thus,

$$A_{R,F} = \frac{1}{\sqrt{\cos^2\left(\omega h_2/v_{s2}\right) + \left(\omega \left(\xi h_2/v_{s2}\right)\right)^2}}$$

4.51

In order to deconvolve from the truncated surface to the bedrock motion, the displacement transfer function would be:

$$A_{F,R} = \frac{e_R' + f_R'}{e_2' + f_2'}$$

<div align="right">4.52</div>

$$A_{F,R} = \sqrt{\cos^2\left(\frac{\omega h_2}{v_{s2}}\right) + \left(\omega\left(\frac{\xi h_2}{v_{s2}}\right)\right)^2}$$

Effectively, if $A_1 = A_{R,F}$ then $A_{F,R} = \frac{1}{A_1}$, then by Equations 4.51 and 4.52,

$$A_{R,F} * A_{F,R} = A_1 * \frac{1}{A_1} = 1.0$$

<div align="right">4.53</div>

For a single two layer soil profile, if the amplitude of the bedrock motion is defined as u_{amp},

then

$$u_{surface} = A_1 * u_{amp}$$

<div align="right">4.54</div>

$$u_{bedrock} = u_{surface} * \frac{1}{A_1} = A_1 * u_{amp} * \frac{1}{A_1} = u_{amp}$$

<div align="right">4.55</div>

Equation 4.55 illustrates an important behavior in wave propagation: "profile-motion consistency." In other words, if a single soil profile is used to develop a ground motion and then return the motion back to the source, it will recover the input only if the soil profile and ground motion are the same. If either is changed then the responses of the system will be modified.

4.3 Results and Key Observations for a 2-Layer System

In order to demonstrate the theoretical profile-motion consistency, the transfer functions are calculated for a uniform soil column and a soil column with a softer top soil layer.

Assuming an initial u_{amp} of 1.0 at the bedrock layer, the surface, in-layer, and foundation

surface transfer function response spectra are determined using Equations 4.47, 4.48, and

4.51 respectively, and shown in Figure 4.3. By using the foundation surface output times

the transfer function $A_{R,F}$, defined by Equation 4.52, the foundation input is recovered, as

shown in Figure 4.4. For the non-uniform column, the softer top soil layer resulted in a

different free-field surface and foundation elevation in-layer responses, as shown in Figure

4.5, but does not alter the foundation elevation response. Again, profile-motion consistency

is demonstrated as the foundation surface response multiplied by the downward transfer

function returns the input motion in the complete frequency range.
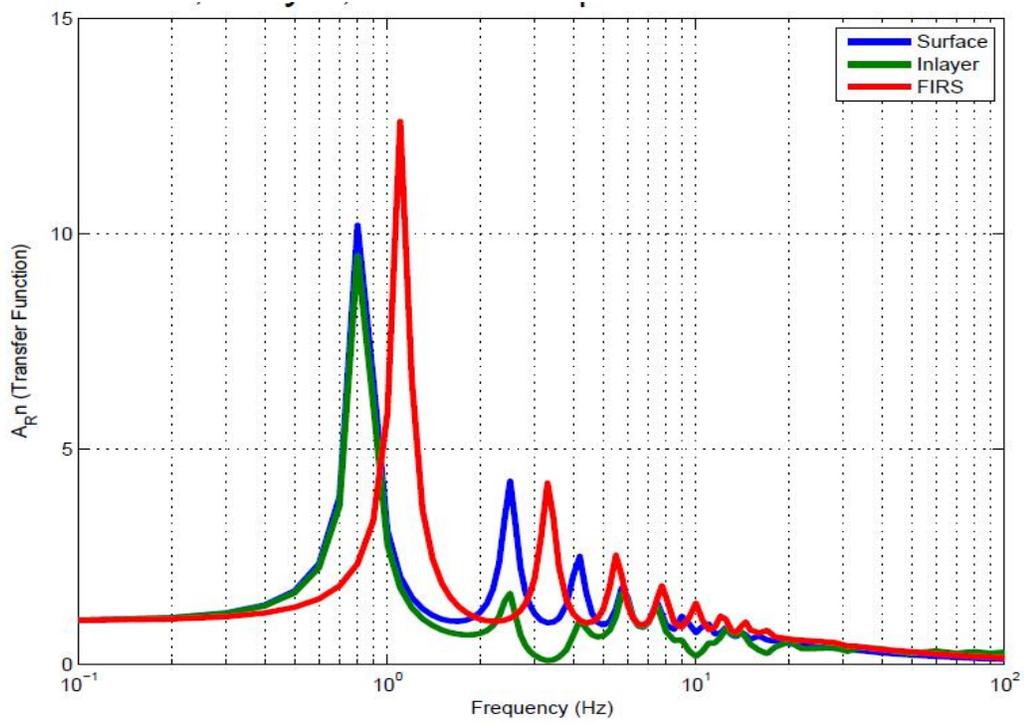
**Figure 4.3: Theoretical Responses for Uniform Soil Column, Various Elevations**
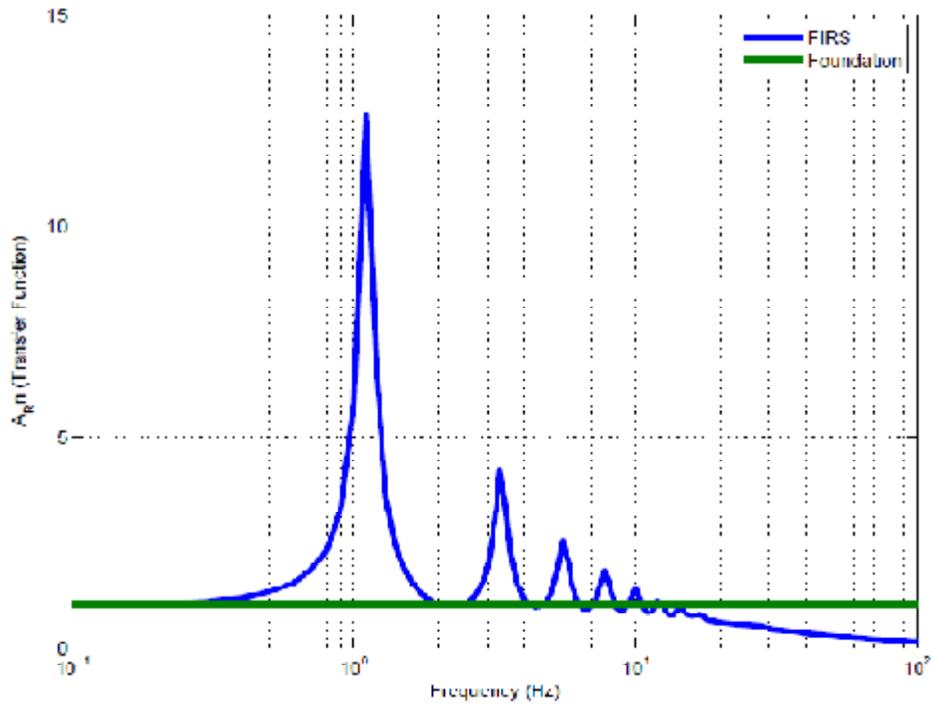
**Figure 4.4: Theoretical Foundation Spectra at Truncated Surface and Bedrock**
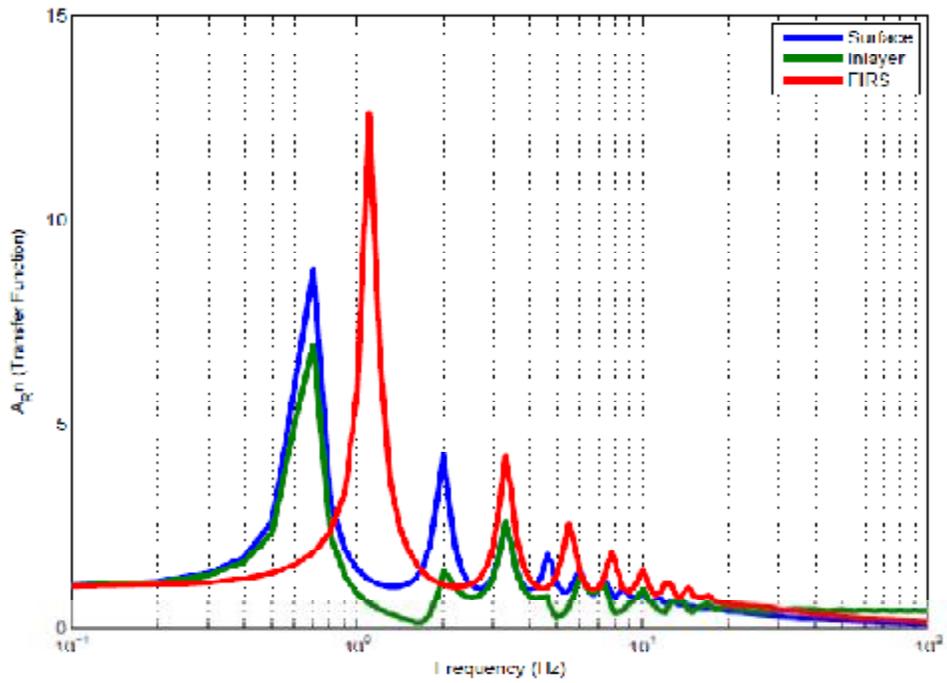
Figure 4.5: Theoretical Responses for Non-Uniform Soil Column, Various Elevations

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The closed-form solution is next used to examine the behavior for a suite of 60 soil

profiles. In order to obtain a variety of strain iterated soil profiles, the Western US soil is

examined for different behaviors across two layer soil sections. Three different soil

behaviors are selected, where 20 profiles have a significantly softer surface layer, 20 profiles

have a significantly harder surface layer, and 20 layers have similar layer stiffness. As a

result, the BLU profiles are relatively uniform with a significant standard deviation. The

transfer functions (Equations 4.47, 4.48, and 4.51) are found for all 60 profiles as well as

three BLU soil profiles, with the mean foundation surface response shown in Figure 4.6.

For each individual profile, if profile-motion consistency is maintained, the inverse of the

$A_{R,F}$ transfer function times the individual foundation surface response returns the original

input. If instead the mean foundation surface response is used as the input for all soil layers

at the truncated surface, as is the case in the current-state-of-practice, and multiplied by the

individual transfer functions, the solution fails to return to the original input for either 60

profiles or the BLU soil profiles, as shown in Figure 4.7.
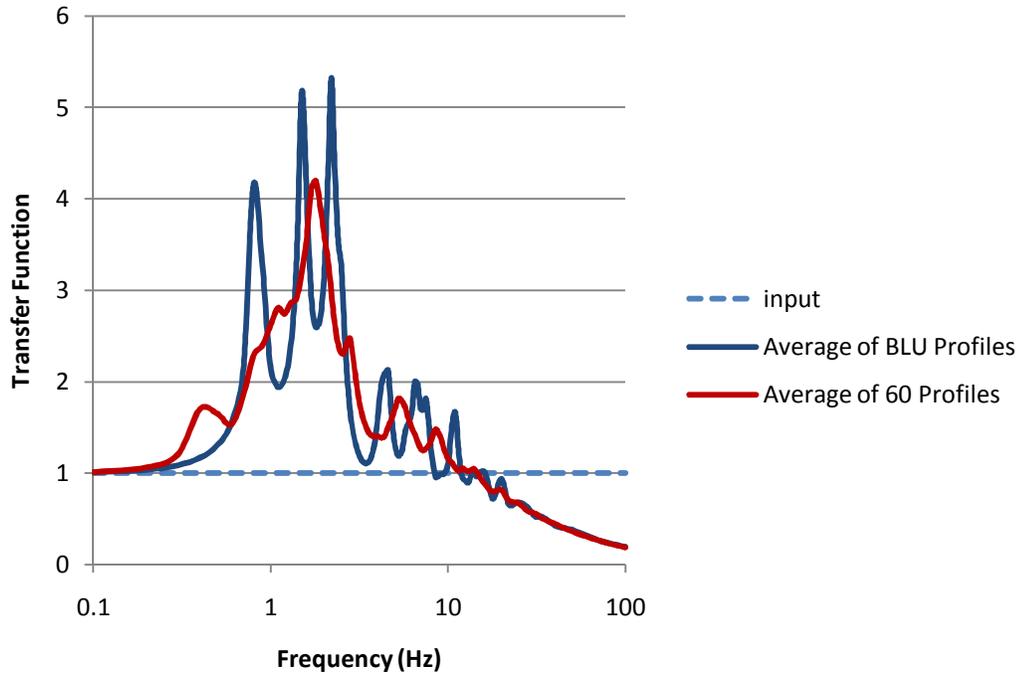
Figure 4.6: 2-Layer Solution Foundation Elevation Transfer Function Responses
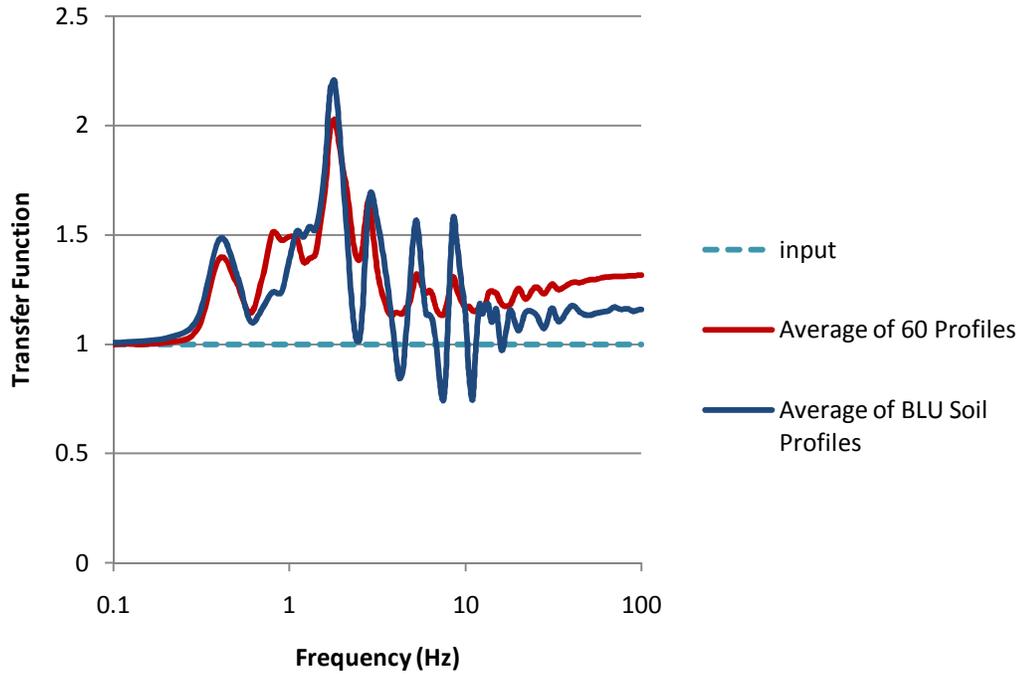

Figure 4.7: 2-Layer Solution Bedrock Responses

66

The failure of the two layer system to return the original input motion illustrates a central tenet in wave propagation, consistency in soil profiles and ground motion. If one soil column is used to generate a ground motion, this motion can be returned along the original soil column, recovering the original input. This profile-motion consistency is critical for developing a consistent bedrock response from a surface motion or design spectrum. Where the single column has a consistent transfer of motion, if either the soil column or the ground motion are altered, this consistency is lost. As the current-state-of-practice uses n soil profiles to generate a mean ground motion such as FIRS or GMRS, this average motion defined at the free surface is no longer consistent with the individual soil columns. Further, typically only the three BLU soil profiles are used for motion transfer. While they may approximate the behavior of the full set of soil columns, they are not consistent with any ground motion inputs. Thus, the only consistent transfer of motion from the bedrock to the surface and back is with the same soil column and an unaltered surface motion. This is true for any number of n soil profiles, where an increased number of profiles may mitigate the error introduced by violating profile-motion consistency.

This theoretical exercise illustrates the problems caused by using altered surface responses that violate profile-motion consistency and then deconvolving to the bedrock surface. Additional observations include:

- Using only the three BLU soil profiles results in extremely sharp peaks in the average response at several key frequencies. As a result, the response of the BLU soil profiles does closely match the average response of the 60 soil profiles,

a trend that is not evident when considering real-life soil examples due to the complex nature of the frequency content.

- There is a roughly two-fold reduction from the foundation surface response down to the bedrock level, but some frequency-specific peaks are further accentuated, suggesting that the deconvolution process might pick up and accentuate key soil frequencies across the 60 soil profiles. These small accentuations have the potential to increase at each step in generating the in-layer motions for SSI inputs. However, these peaks are relatively minor when compared to the added response due to violating profile-motion consistency.

CHAPTER 5: PROPOSED CORRECTION AND LIMITATIONS

5.1 Development of a Correction Factor

As discussed in Section 4.3, it is vital to maintain profile-motion consistency in order to develop consistent motions. If a ground motion is input to a soil column and convolved both up to the surface and back down to the bedrock elevation, the output will be consistent only if the soil profile and the surface motion are unaltered. Similarly, this can be done in reverse, where a surface motion is input, transferred to the bedrock, and recovered back at the surface as long as the profile-motion consistency is not violated. However, the current-state-of-practice violates consistency in two ways: first when responses at the free surfaces are averaged to create a design spectrum, and again when the three BLU soil profiles are used as a substitute for the full soil suite.

If the desired SSI in-layer inputs are the results of a bedrock input, then they can be obtained directly at the same time as the GMRS and FIRS. However, there is still the need to transfer a different design spectrum, such as the CSDRS, that is not developed during the site-specific seismic characterization. These design curves cannot maintain consistency, as they are often not generated using the site-specific soil profiles or input ground motion. As a result, these spectra must be modified in some way to create soil suite specific spectra that can then be used to generate consistent motions, and when transferred, consistent in-layer response.

In order to generate a consistent motion starting with an inconsistent design curve or soil suite, it is necessary to scale the given response by some frequency dependent ratio. This ratio will be defined as the "correction factor" α, which is an elevation specific, frequency dependent factor.

In order to determine the correction factor, we will begin by using the closed-form theoretical solution presented in Chapter 4. For any soil profile undergoing harmonic excitation, the amplitude of the motion at various elevations can be found using transfer functions, such as $A_{R,F}$ found by Equation 4.51. Also, the transfer function for any soil profile made of n layers can be found by using the coefficients developed in Equation 4.44. Additionally, if the bedrock motion is defined as u_{amp}, with some transfer function A_1, the input response can be recovered for a consistent system as shown by Equation 4.55. If we instead use two soil profiles with transfer functions A_1 and A_2, the surface motions from a bedrock amplitude u_{amp} can be characterized as

$$u_{S,1}(\omega) = A_1 * u_{amp} \qquad \textbf{5.1}$$

$$u_{S,2}(\omega) = A_2 * u_{amp}(\omega)$$

Then the mean surface response can be defined as

$$u_{S,avg}(\omega) = \frac{u_1(\omega) + u_2(\omega)}{2} = \left(\frac{A_1 + A_2}{2}\right) * u_{amp}(\omega) \qquad \textbf{5.2}$$

The mean surface motion can be transferred back to the bedrock using Equation 4.55, and if we take u_{amp} as 1.0, the average bedrock response would be

$$u_{BR,avg}(\omega) = \frac{1}{2}\left[\frac{A_1 + A_2}{2} * \frac{1}{A_1} + \frac{A_1 + A_2}{2} * \frac{1}{A_2}\right]$$ 5.3

which simplifies to

$$u_{BR,avg}(\omega) = \frac{1}{4}\left[2 + \frac{A_1}{A_2} + \frac{A_2}{A_1}\right]$$ 5.4

This bedrock response is not equal to the original input of 1.0 for any soil columns where $A_1 \neq A_2$. In order to restore profile-motion consistency, the correction ratio, α_{BR}, can be defined as

$$\alpha_{BR}(\omega) = \frac{u_{amp}}{u_{BR,avg}}$$ 5.5

or

$$\alpha_{BR}(\omega) = \frac{4}{2 + \frac{A_1}{A_2} + \frac{A_2}{A_1}}$$ 5.6

If the average surface motion is multiplied by this correction factor, then a corrected surface motion will be generated,

$$u_{S,avg}^*(\omega) = \frac{2(A_1 + A_2)}{2 + \frac{A_1}{A_2} + \frac{A_2}{A_1}}$$ 5.7

In order to recover the original input, $u_{S,avg}^*$ can be deconvolved back to the bedrock as per Equation 4.55 by multiplying $u_{S,avg}^*$ by $\frac{1}{A_1} + \frac{1}{A_2}$, such that

$$u_{BR,avg}^*(\omega) = \frac{1}{2}\left[u_{S,avg}^* * \left(\frac{1}{A_1}+\frac{1}{A_2}\right)\right]$$

$$= \frac{1}{2}\left[2\left(\frac{2+\frac{A_1}{A_2}+\frac{A_2}{A_1}}{2+\frac{A_1}{A_2}+\frac{A_2}{A_1}}\right)\right] \qquad \text{5.8}$$

$$= 1.0$$

Thus, if the surface motion is scaled by the correction factor developed at the bedrock

motion for all frequencies, then the original input motion can be recovered.

5.2 Correction Factor for Multiple Soil Profiles

For any set of n soil profiles and a bedrock input of 1.0, then the average surface motion

can be defined as

$$u_{S,avg}(\omega) = \frac{\sum A_i(\omega)}{n} \qquad \text{5.9}$$

Similarly, the average surface motion can be deconvolved back to the bedrock as

$$u_{BR,avg}(\omega) = \frac{\sum A_i(\omega)}{n}\frac{\sum\frac{1}{A_i(\omega)}}{n}$$

$$u_{BR,avg}(\omega) = \frac{1}{n^2}\sum A_i(\omega)\sum\frac{1}{A_i(\omega)} \qquad \text{5.10}$$

For the correction factor defined by Equation 5.5,

$$\alpha_{BR}(\omega) = \frac{1}{u_{BR,avg}(\omega)}$$

<div align="right">5.11</div>

$$\alpha_{BR}(\omega) = \frac{n^2}{\sum A_i(\omega) \sum \frac{1}{A_i(\omega)}}$$

Applying the correction factor to the average surface motion would result in

$$u_{S,avg}^*(\omega) = \frac{\sum A_i(\omega)}{n} * \frac{n^2}{\sum A_i(\omega) \sum \frac{1}{A_i(\omega)}}$$

<div align="right">5.12</div>

$$u_{S,avg}^*(\omega) = \frac{n}{\sum \frac{1}{A_i(\omega)}}$$

Finally, if this corrected surface motion is deconvolved back to the bedrock elevation, then the average bedrock motion would be

$$u_{BR,avg}^*(\omega) = u_{S,avg}^*(\omega) * \frac{\sum \frac{1}{A_i(\omega)}}{n}$$

$$u_{BR,avg}^*(\omega) = \frac{n}{\sum \frac{1}{A_i(\omega)}} * \frac{\sum \frac{1}{A_i(\omega)}}{n}$$

<div align="right">5.13</div>

$$u_{BR,avg}^*(\omega) = 1.0$$

Specifically, for a set of three soil profiles, such as the three BLU soil profiles, and corresponding transfer functions A_1, A_2, and A_3, the generic forms generated in Equations 5.9 through 5.13 can be solved as

$$u_{S,avg} = \frac{A_1 + A_2 + A_3}{3} \qquad \text{5.14}$$

$$u_{BR,avg} = \frac{3A_1 A_2 A_3 + A_1^2(A_2 + A_3) + A_2^2(A_1 + A_3) + A_3^2(A_1 + A_2)}{9A_1 A_2 A_3} \qquad \text{5.15}$$

$$\alpha_{BR} = \frac{9A_1 A_2 A_3}{3A_1 A_2 A_3 + A_1^2(A_2 + A_3) + A_2^2(A_1 + A_3) + A_3^2(A_1 + A_2)} \qquad \text{5.16}$$

$$u_{S,\alpha} = \frac{3A_1 A_2 A_3(A_1 + A_2 + A_3)}{3A_1 A_2 A_3 + A_1^2(A_2 + A_3) + A_2^2(A_1 + A_3) + A_3^2(A_1 + A_2)} \qquad \text{5.17}$$

$$
\begin{aligned}
u_{BR,avg} \\
= \frac{1}{3} * 3 & \left[\frac{3A_1 A_2 A_3 + A_1^2(A_2 + A_3) + A_2^2(A_1 + A_3) + A_3^2(A_1 + A_2)}{3A_1 A_2 A_3 + A_1^2(A_2 + A_3) + A_2^2(A_1 + A_3) + A_3^2(A_1 + A_2)} \right] \\
= 1.0 &
\end{aligned}
\qquad \text{5.18}
$$

5.3 *Correction Factor from Surface Motions*

Furthermore, the position transfer function formulation can be rewritten in an inverse form for a case where the input motion could be a design curve at a free surface instead of a bedrock motion. Assuming a surface motion amplitude of 1.0, the bedrock response could be found as

$$u_{BR,avg}(\omega) = \frac{\Sigma \frac{1}{A_i(\omega)}}{n} \qquad \text{5.19}$$

This motion would then be convolved back to the surface, where

$$u_{S,avg}(\omega) = \frac{1}{n^2} \sum A_i(\omega) \sum \frac{1}{A_i(\omega)} \qquad \text{5.20}$$

The surface correction factor can be found in the same way as the bedrock correction factor, where

$$\alpha_S(\omega) = \frac{1}{u_{S,avg}(\omega)}$$

$$\text{5.21}$$

$$\alpha_S(\omega) = \frac{n^2}{\sum A_i(\omega) \sum \frac{1}{A_i(\omega)}}$$

Thus the corrected bedrock motion would be found as

$$u^*_{BR,avg}(\omega) = \frac{\sum \frac{1}{A_i(\omega)}}{n} * \frac{n^2}{\sum A_i(\omega) \sum \frac{1}{A_i(\omega)}}$$

$$\text{5.22}$$

$$u^*_{BR,avg}(\omega) = \frac{n}{\sum A_i}$$

And if the corrected motion is transferred back to the surface, then the corrected average surface motion would be

$$u^*_{S,avg}(\omega) = \frac{\sum A_i}{n} * \frac{n}{\sum A_i}$$

$$\text{5.23}$$

$$u^*_{S,avg}(\omega) = 1.0$$

It is important to note that the correction factors α_{BR} and α_S are found from two different directions, and result in the same correction factor. Thus by Equations 5.11 and 5.21,

$$\alpha(\omega) = \alpha_S(\omega) = \alpha_{BR}(\omega)$$

$$\alpha(\omega) = \frac{n^2}{\sum A_i(\omega) \sum \frac{1}{A_i(\omega)}}$$

It is important to note that while the correction factor can be applied at either elevation, it is specific to the two bounding surfaces where it is developed, i.e., the correction factor for the free-field surface to the bedrock is only valid at those two elevations, whereas another correction factor must be made for a foundation level.

5.4 Calculation of Correction Factor

In order to calculate the correction factor, two independent approaches are possible, allowing for verification. First, for some set of soils, the correction factor can be found numerically by applying a unit amplitude ground motion, transferring the motion from a bedrock motion to the surface and vice versa, where the resulting response can be used to determine the factor. A second, direct approach can be found by developing n separate transfer functions, and summing the individual functions as shown in Equation 5.24.

5.4.1 Numerical Approach

To numerically illustrate the procedure for calculating the correction factor from Section 5.2, the correction factor is calculated for the 60 two layer soil profiles considered in Chapter 4. Initially, bedrock amplitude of 1.0 is used as the input to the soil profiles, and using the individual transfer functions for each profile results in 60 individual truncated surface responses. These responses are then averaged, generating a mean surface response,

which is used as the input to then transfer down the truncated column. Again 60 individual

responses are found, now at the bedrock, and the average of the 60 is calculated. Figure 5.1

shows the original input amplitude, the mean surface response, and the resulting mean

bedrock response. As stated in Equation 5.11, $\alpha_{BR} = \frac{1}{mean\ bedrock\ response}$. Thus the

correction factor is calculated for all frequencies as the inverse of the mean bedrock

response.

To check the solution, the mean truncated surface response is then multiplied by the

correction factor α_{BR}, generating corrected mean truncated surface response, shown in

Figure 5.2. The corrected mean truncated surface motion is then deconvolved down to the

bedrock, where the mean corrected bedrock response equaled the original input amplitude of

1.0, as shown in Figure 5.3.
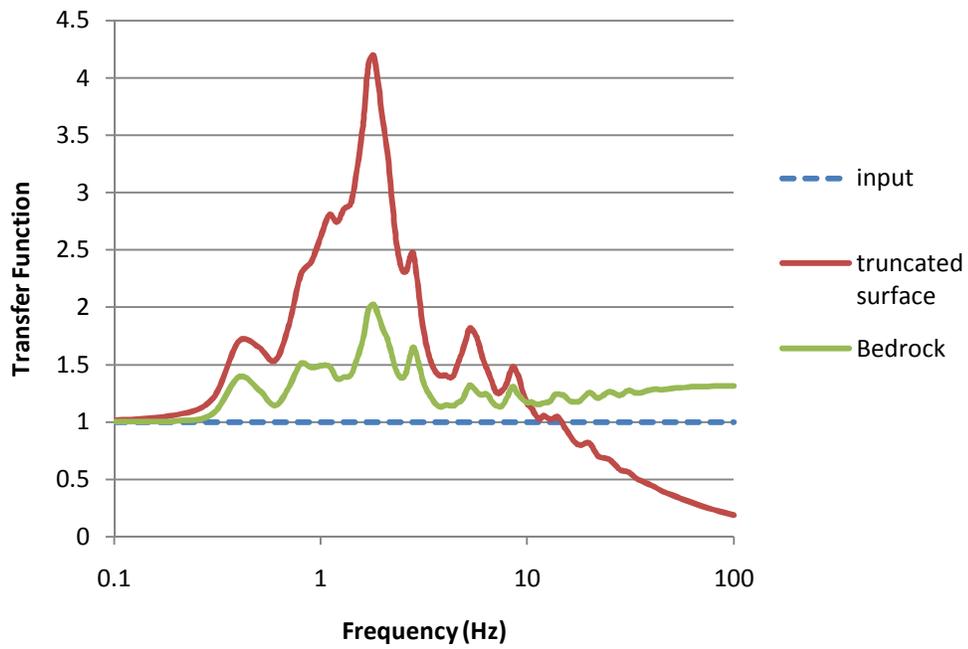
**Figure 5.1: Average Truncated Surface and Bedrock Response for 2-Layer System**
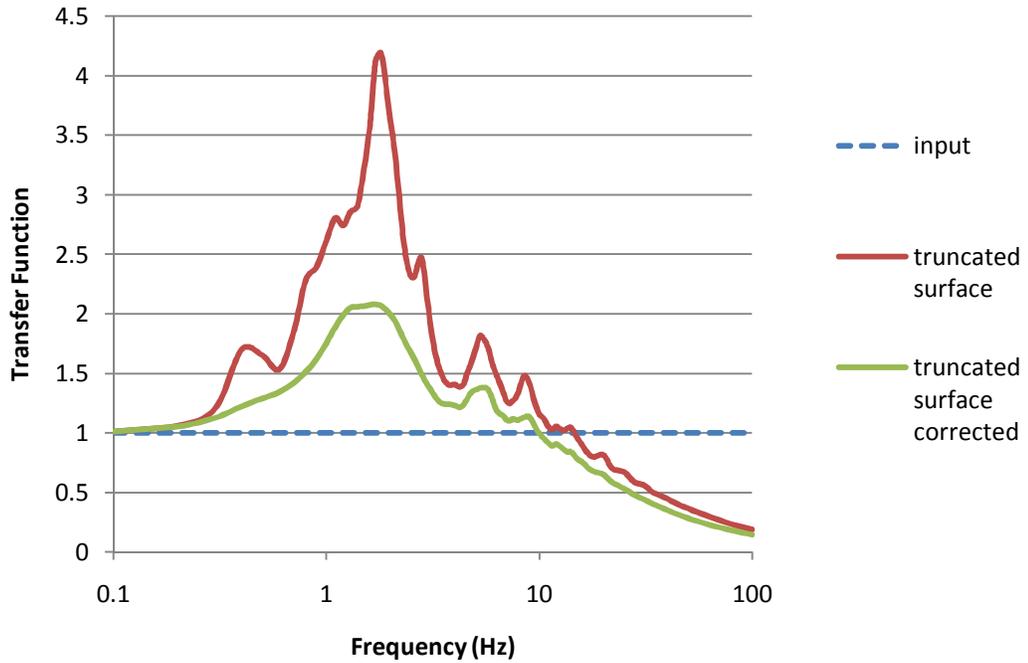
Figure 5.2: Average Truncated Surface and Corrected Truncated Surface Response for 2-Layer System
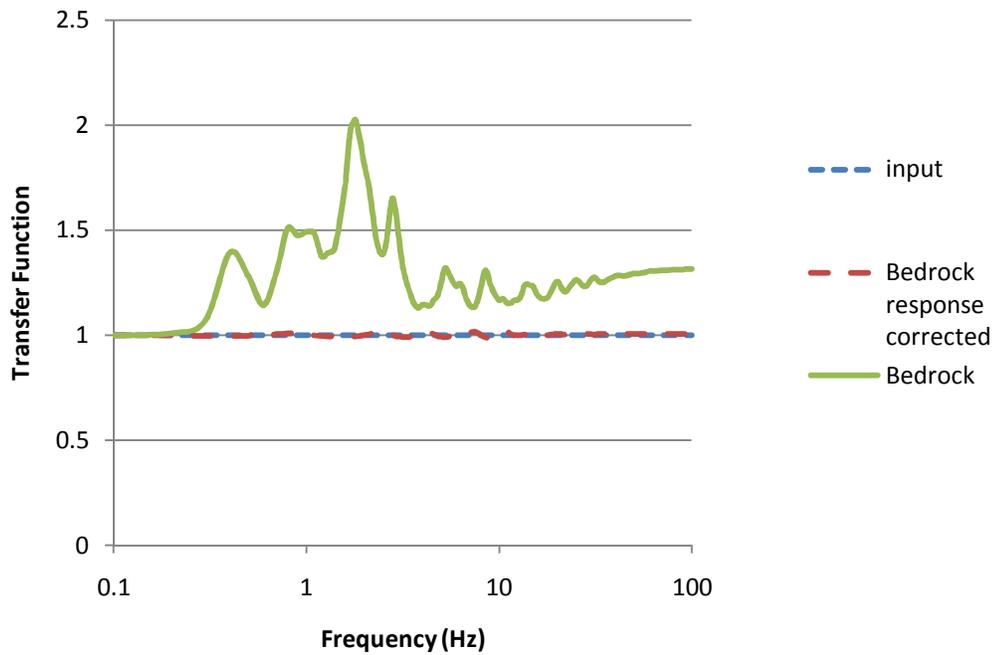
Figure 5.3: Average Bedrock Response and Corrected Bedrock Response for 2-Layer System

While all of the transfers and corrections given above are made for a full set of 60 soil profiles, the current-state-of-practice is to use the three BLU soil profiles to reduce computational effort. However, care must be taken when using the BLU soil profiles to ensure consistency. As previously mentioned the correction factor is elevation specific and also soil profile specific. If the corrected mean truncated surface response is transferred down to the bedrock using the three BLU soil profiles, there is actually an increase in response, as seen in Figure 5.4. If instead, the correction factor is determined using the BLU soil profiles rather than the 60 profiles, the corrected surface average BLU soil profile response can be deconvolved to bedrock level, and returns the original input, as shown in Figure 5.5. While a consistent correction factor can be generated for the BLU soil profiles,

there is a wide discrepancy between the average values of the correction factor for the BLU

soil profiles and the average values of the correction factor for the complete 60 soil profile

suite, as shown by Figure 5.6.
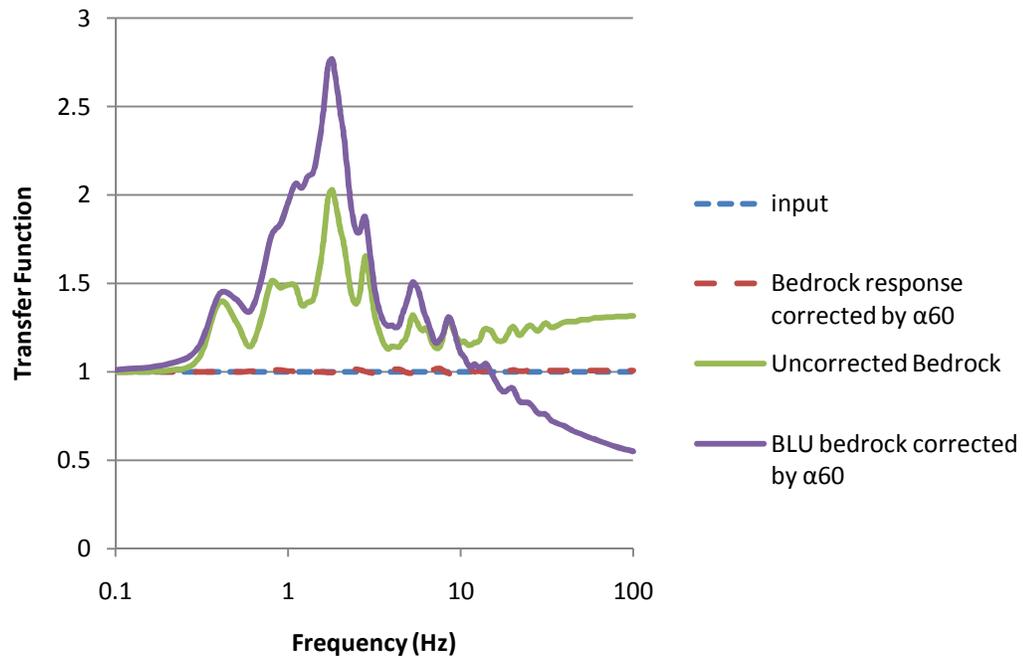
**Figure 5.4: Bedrock Response Using 60 Profile Correction Factor for 2-Layer System**
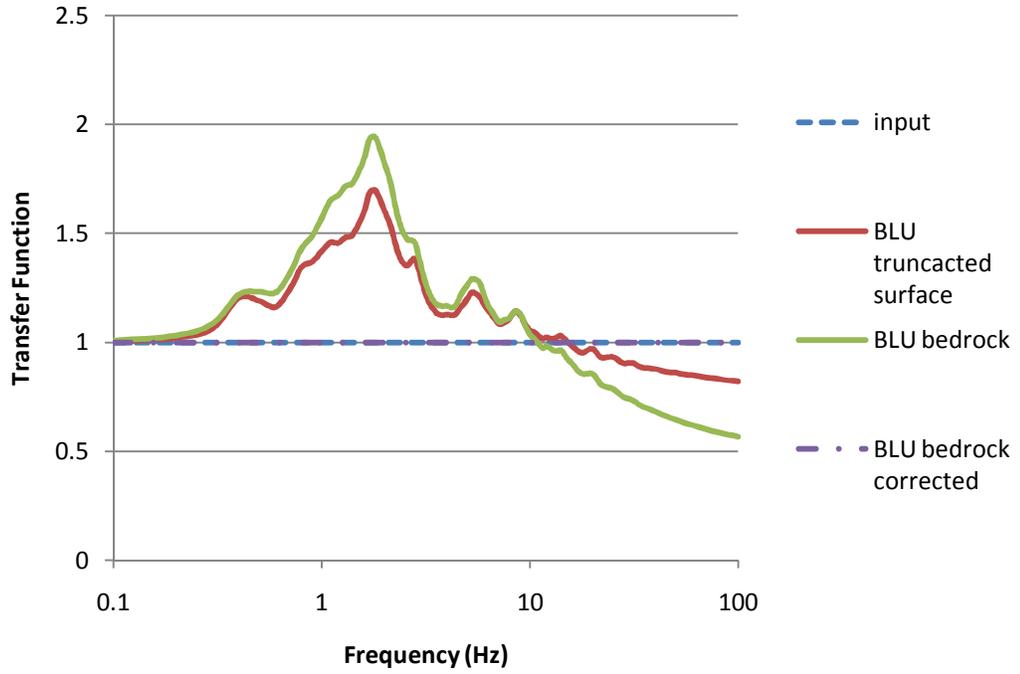
**Figure 5.5: Responses using BLU Soil Profiles and BLU Correction Factor for 2-Layer System**
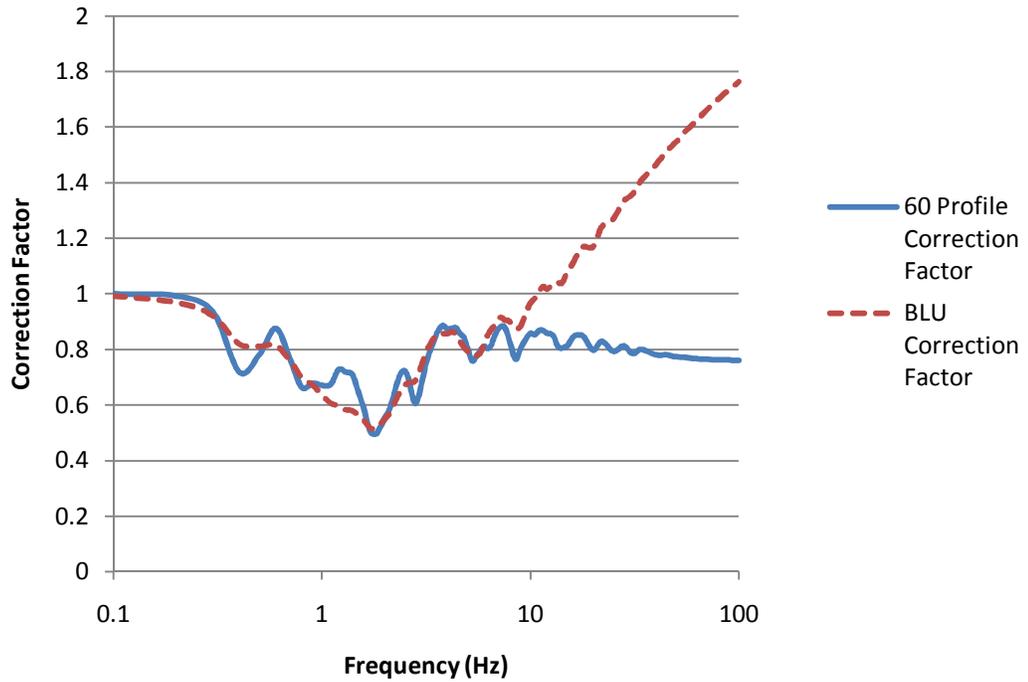
Figure 5.6: 2-Layer Correction Factors

5.4.2 *Direct Approach*

In order to directly calculate the correction factor, the individual transfer functions must be determined. For any soil suite with n soil profiles, the transfer function between the surface and the bedrock will be a based on the properties for the individual layers, as defined in Equation 4.44. For each layer, the terms $\left(h/v_s\right)_i$ and $\left(h\xi/v_s\right)_i$ can be determined and added for the entire profile. Next, the transfer function for each profile can be determined as a function of frequency. Finally, these n functions are input in Equation 5.21 to determine the correction factor.

As a check, the correction factor is determined by both methods for the 60 two-layer soil profiles.   Figure 5.7 shows the numerical method for calculating the correction factor, including the mean free-field surface response and resulting mean bedrock response used to calculate the correction factor. Both the numerical approach and the direct approach produce the same correction factor when using the 60 soil profiles. If instead the BLU soil profiles are used, the correction factor has significant differences, as shown by Figure 5.8. While differences are expected, the three profiles do not produce a sufficient number of responses to average out a smoother curve for the correction factor. Instead, there is the potential of a single profile driving the response and generating inaccurate response spectra. While the BLU soil profiles are preferred for reduced computational effort, it may be better to apply the correction factor to a suite of 60 soil profiles, rather than the BLU soil profiles.
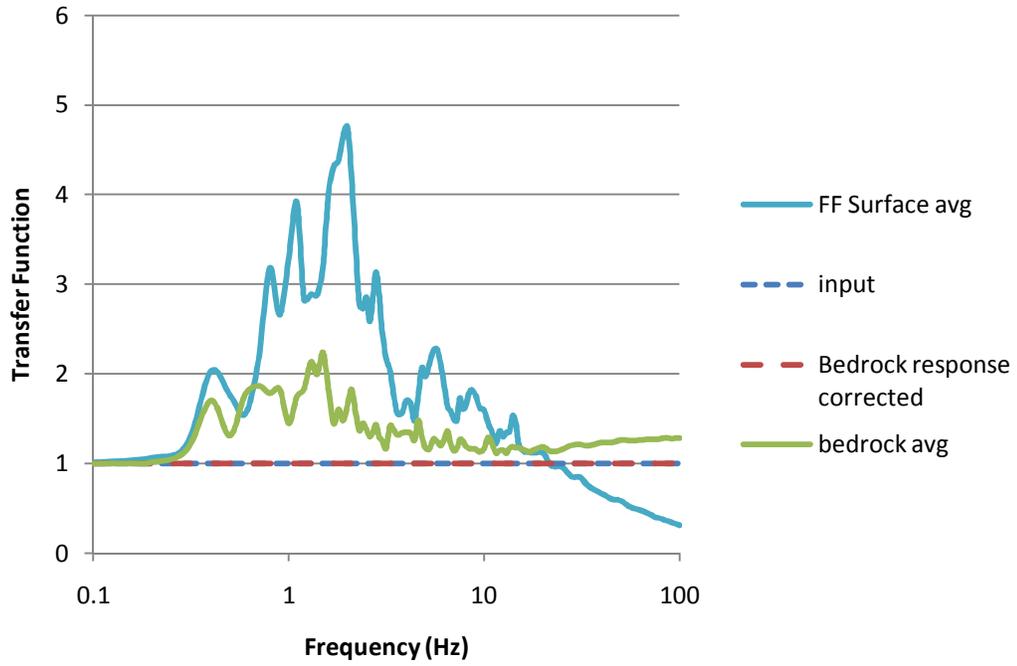
Figure 5.7: 2-Layer Responses for Determining the Correction Factor
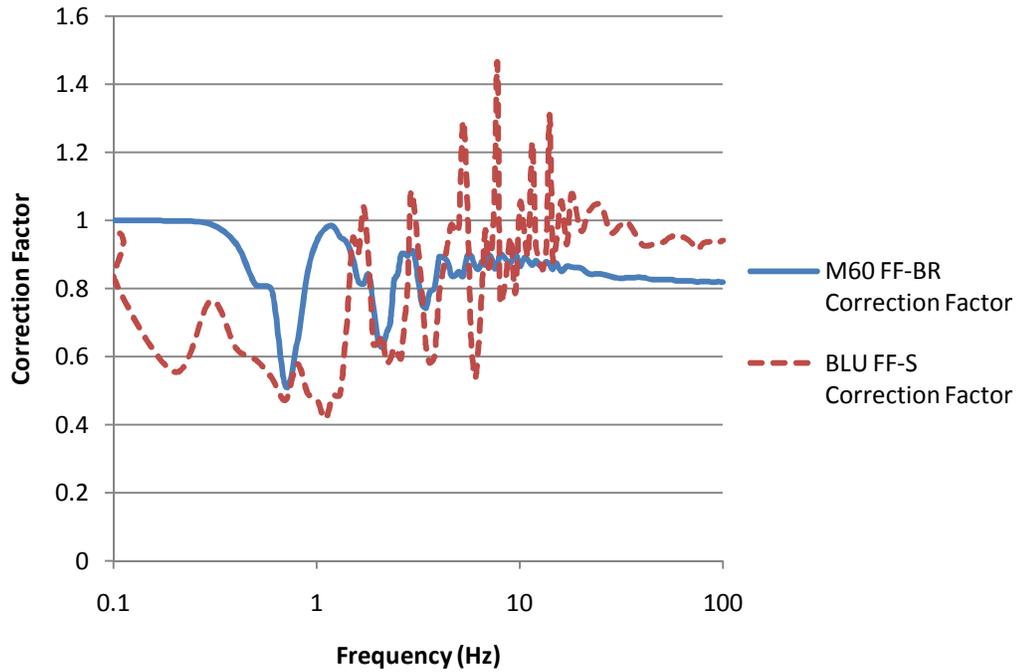

Figure 5.8: Western Soil Free-Field to Bedrock Correction Factors

Finally, the correction factor is applied to the 60 Western US soil profiles including all 47 layers. Using a bedrock input of 1.0, the average surface motion is found using the displacement transfer functions. The average surface motion is then used to deconvolve back to the bedrock elevation. At the bedrock elevation, the response is again double that of the original input. Finally, the correction factor calculated directly can be applied to the average surface response, returning the 1.0 bedrock input. It is important to note that using the full 47 layers introduced minor numerical error, where the corrected response averaged within 1% of the input, as compared to a 20% average error for the uncorrected response. Figure 5.9 shows the average surface response and resulting average bedrock response. Figure 5.10 shows the average bedrock response before and after correction.
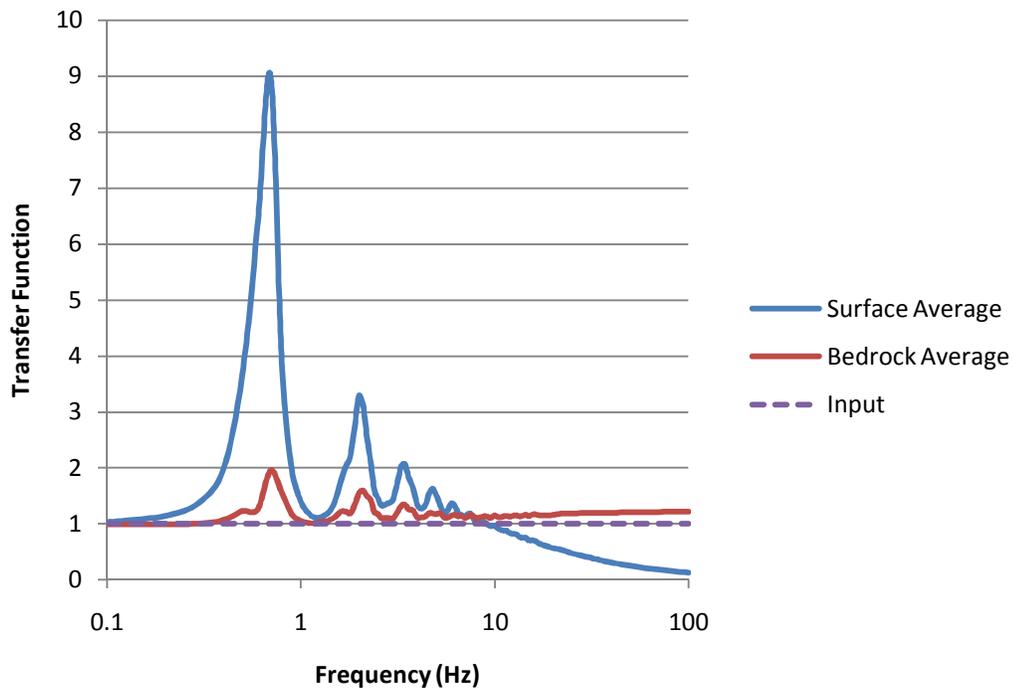

Figure 5.9: Western Soil Input and Average Surface Response
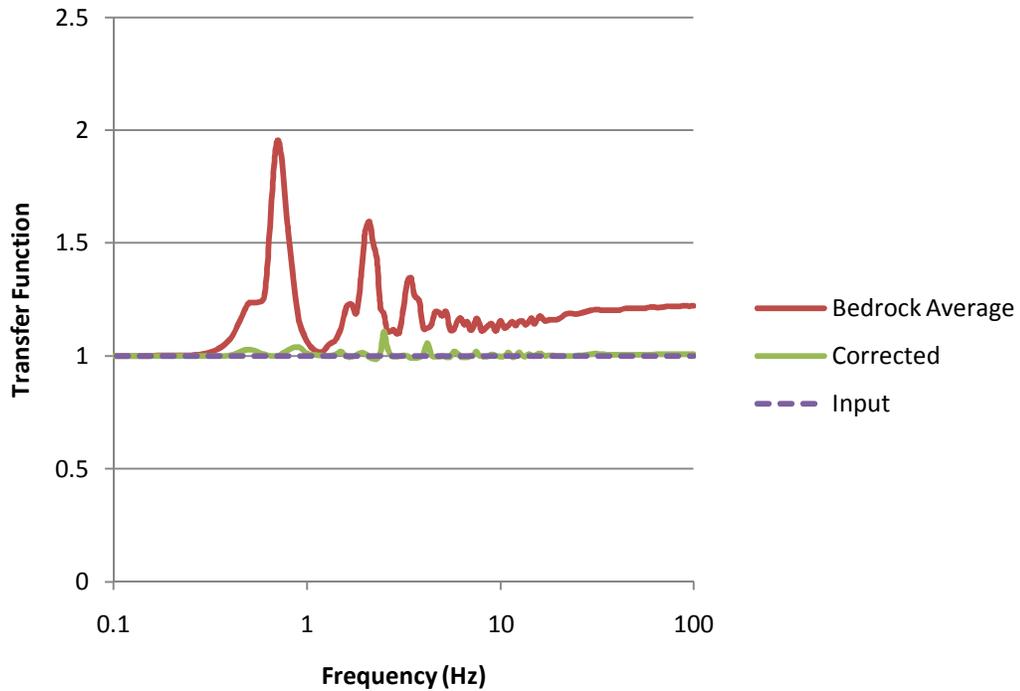
Figure 5.10: Western Soil Bedrock Responses for Harmonic Excitation

5.5 Real-life Application of Correction Factor

Using the direct approach from Section 5.4.2 and Equation 5.24, the correction factor can be readily calculated for any soil profile suite and elevation, without added computational effort. The ease in calculating the correction factor makes it practical to modify the current-state-of-practice. The current-state-of-practice takes a surface design spectrum, deconvolves a corresponding surface motion to the bedrock, and then uses the bedrock motion as an input for the full soil column to generate SSI inputs. Thus, there are two potential elevations to apply the correction factor: either at the surface or the bedrock elevation.

Using the data generated by the study in Chapter 3, surface motions generated using the 60 soil profiles, such as the GMRS and FIRS, as well as the corresponding bedrock motions can be determined for each soil suite. As a result, the input motion at the bedrock will be independent of deconvolving the corrected surface motions. Next, the correction factor is determined for each soil column in accordance with Equation 5.24, such that separate correction factors are generated for the full soil column and the truncated soil column. These correction factors are developed specifically for surface to bedrock motion transfers, using the full set of 60 soil profiles. As established in Section 5.3, the correction factors can be applied at either the bedrock or at the surface.

The application of the correction factor will be analyzed for three specific scenarios:

- Case 1: Free-field surface motion deconvolved to the bedrock, correcting the bedrock motion, and used as the input to determine the in-layer responses

- Case 2: Truncated surface motion deconvolved to the bedrock, correcting the bedrock motion, and used as the input to determine in-layer responses

- Case 3: Correction factor applied at the surface, then deconvolved to the bedrock

5.5.1 *Correction Factor Case 1*

The simplest application of the correction factor is on the free-field surface motion that is deconvolved to the bedrock. These bedrock motions are then multiplied by the full soil column correction factors, generating bedrock corrected responses for each soil set. Again, RASCAL is used to fit an earthquake seed to generate corresponding ground motions. These motions are then convolved to the surface elevation and the inlayer responses.

For the Western US and Eastern US soils, the corrected bedrock response improved the accuracy and consistency significantly compared to the current-state-of-practice. The benchmark response and the corrected response are close to each other at the peak frequencies where the maximum difference occurs in the current-state-of-practice. Figure 5.11 shows the Western US soil in-layer responses at the 50 foot foundation elevation, and Figure 5.12 shows the Eastern US soil in-layer response at the 75 foot foundation elevation. For the Midwestern soil, the results showed an improvement, but they are not as close as in the case of the Western and Eastern US soil profiles. Figure 5.13 shows the Midwestern US soil in-layer response at the 75 foot foundation elevation.
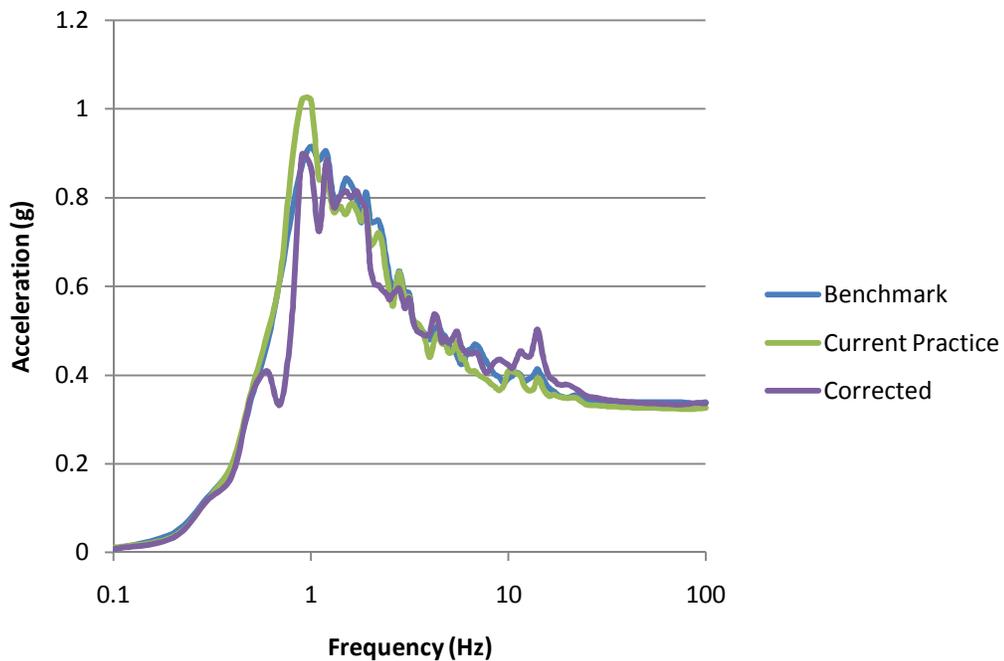
Figure 5.11: In-layer Responses for the Western Soil at the 50ft Foundation Elevation
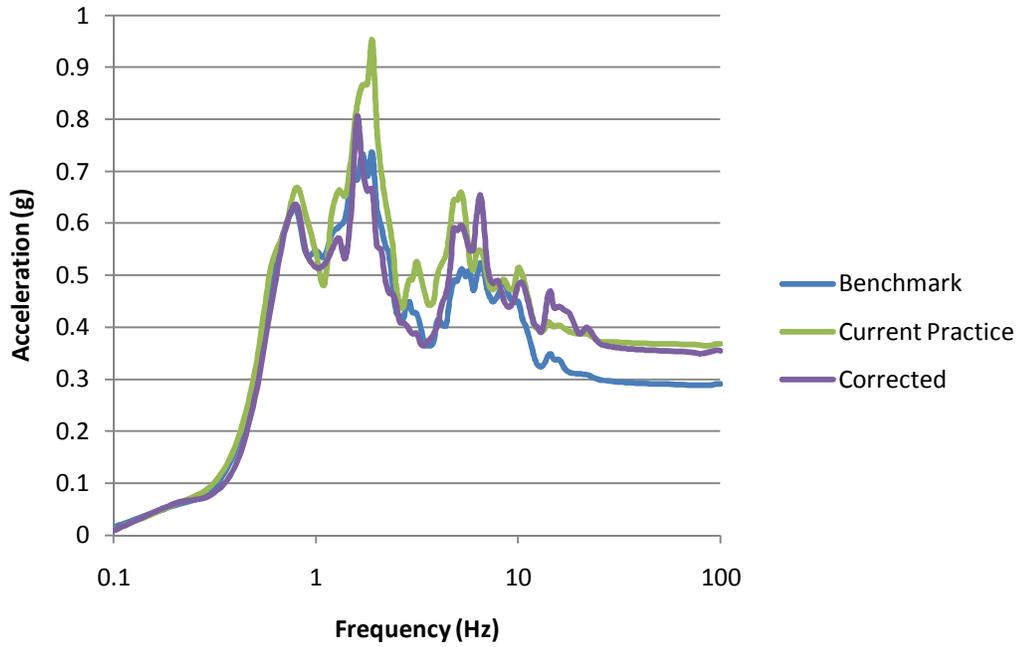
Figure 5.12: In-layer Responses for the Eastern Soil at the 75ft Foundation Elevation
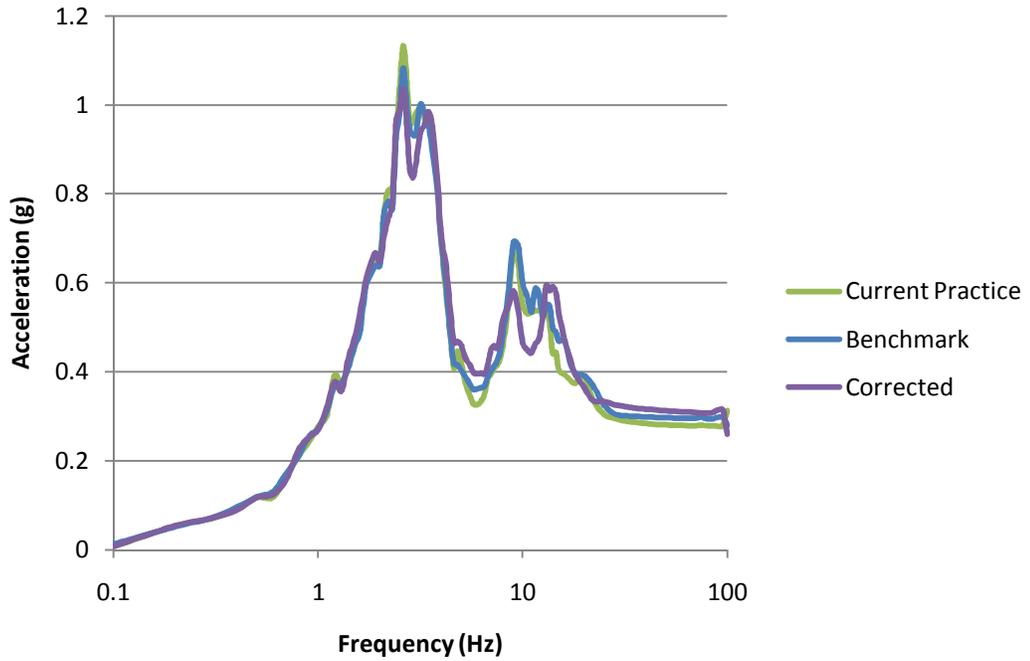
Figure 5.13: In-layer Responses for the Midwestern Soil at the 75ft Foundation Elevation

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5.5.2 *Correction Factor Case 2*

The correction factor can also be applied to the bedrock response deconvolved from the mean truncated surface responses. These responses can be used as inputs for the full column, and then compared to the benchmark and corresponding uncorrected responses calculated using the current practice. Generally, the corrected responses are similar to the corrected responses calculated in Case 1. The responses for the 50 foot foundation elevation in-layer responses are presented for each soil type in Figure 5.14, Figure 5.15, and Figure 5.16.
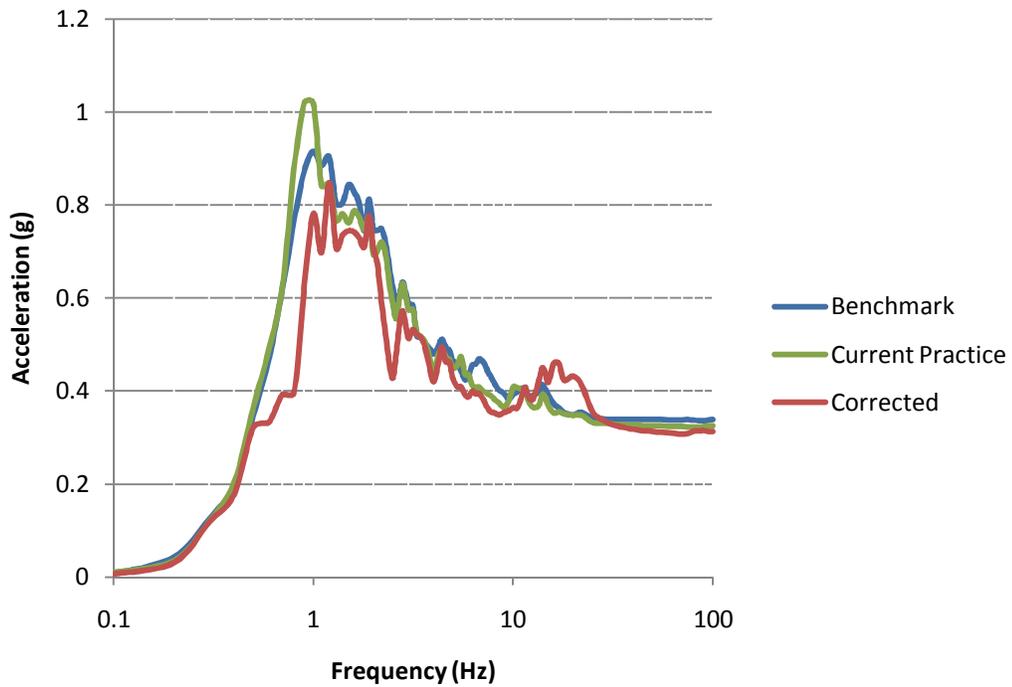
**Figure 5.14: In-layer Responses for Western Soil at the 50ft Foundation**
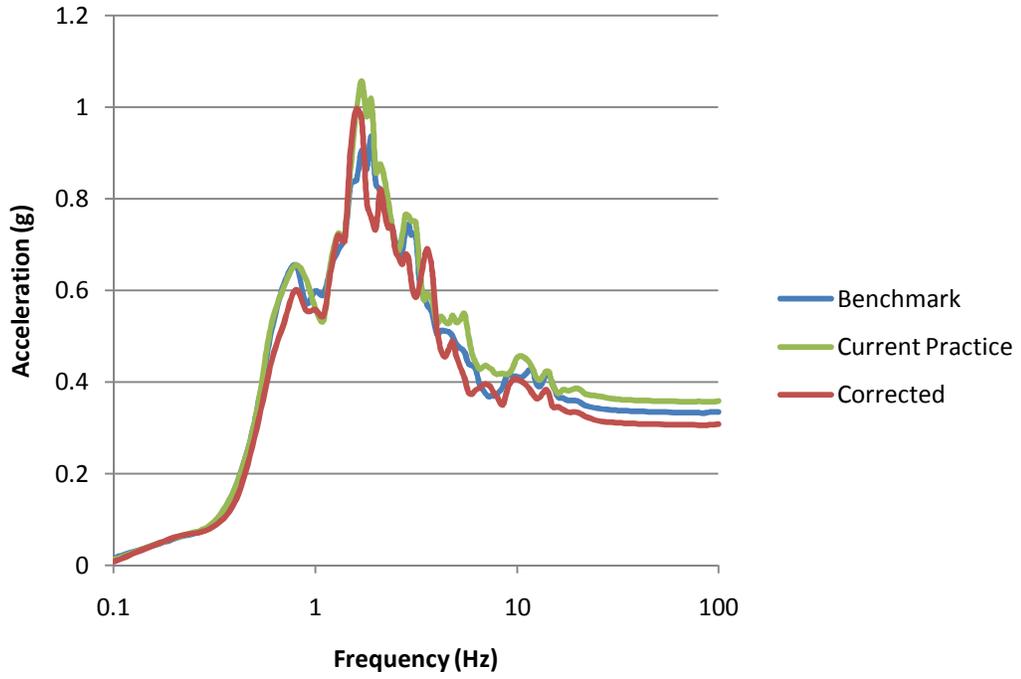
Figure 5.15: In-layer Responses for Eastern Soil at the 50ft Foundation Elevation
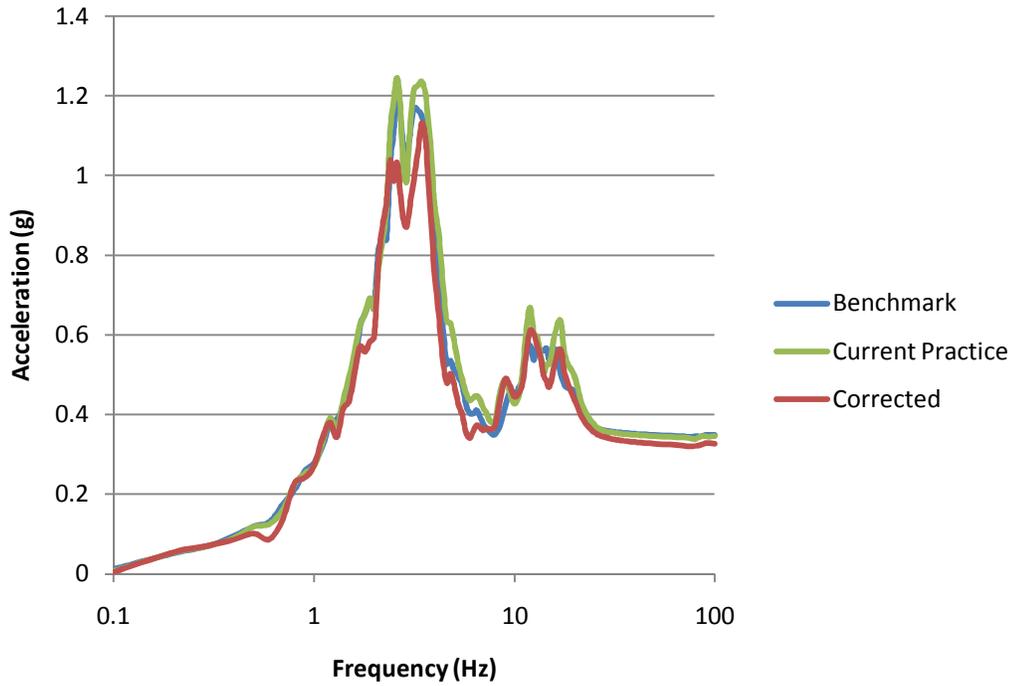
**Figure 5.16: In-layer Responses for Midwestern Soil at the 50ft Foundation Elevation**

5.5.3 *Correction Factor Case 3*

Unlike the previous two cases in which the correction factor is applied to the bedrock motion from the deconvolved surface motions, the correction factor can also be applied directly to the surface responses. Each surface response spectra is multiplied by the respective correction factors, generating corrected response spectra. RASCAL13 is then used to individually generate corresponding surface motions, which are used as input motions for SHAKE. Each motion is input into the suite of 60 soil profiles at the soil column surface, and deconvolved down to the bedrock elevation, where a mean bedrock response is generated. This procedure is completed for all three soil types using the free field surface and all three foundation elevations.

The surface corrected response deconvolved to the bedrock is compared to the original input, the bedrock response from the current-state-of-practice, and the corrected bedrock response from the previous sections. These comparisons for the Western, Eastern, and Midwestern US soil profiles are shown in Figure 5.17, Figure 5.18, and Figure 5.19, respectively.   Generally, the surface corrected response is between the uncorrected response and the corrected response from the previous sections.
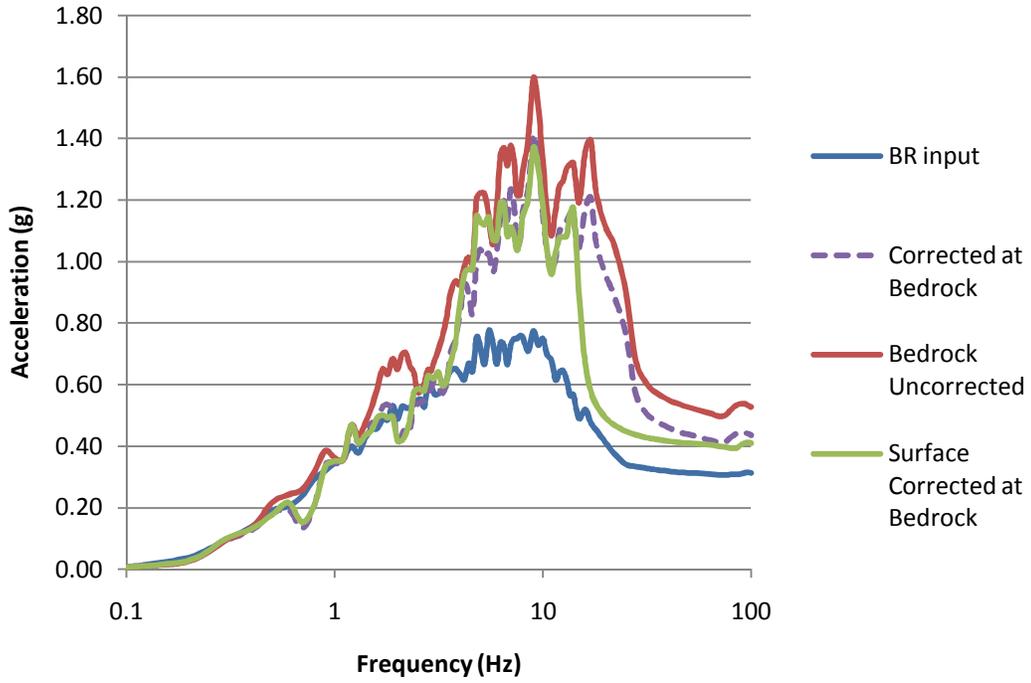
Figure 5.17: Bedrock Response Spectra for Western Soil
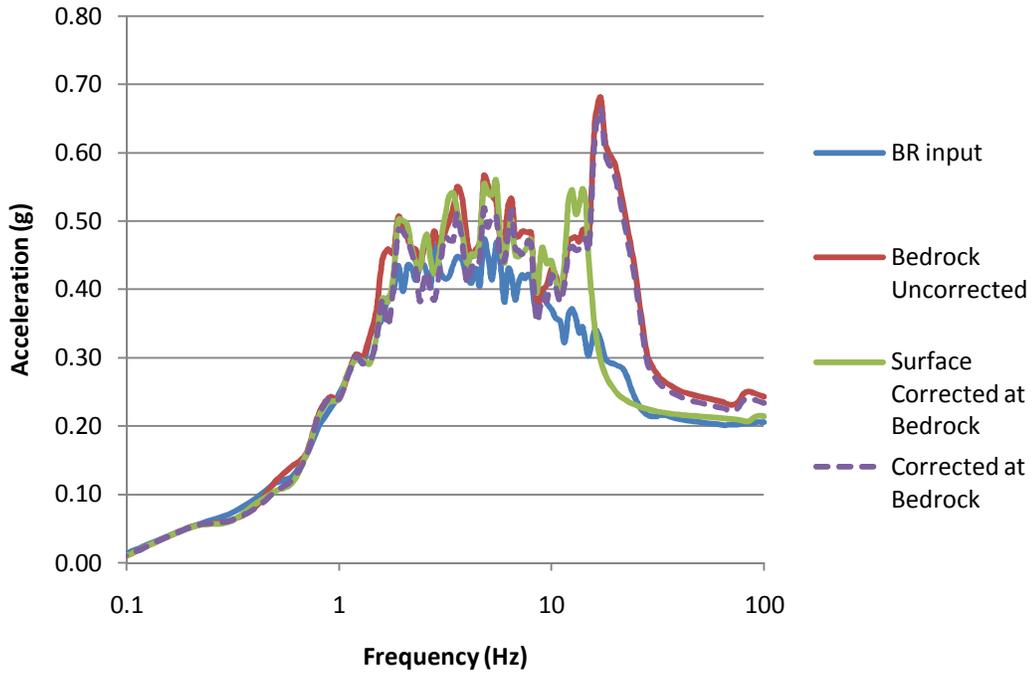
Figure 5.18: Bedrock Responses for Eastern Soil
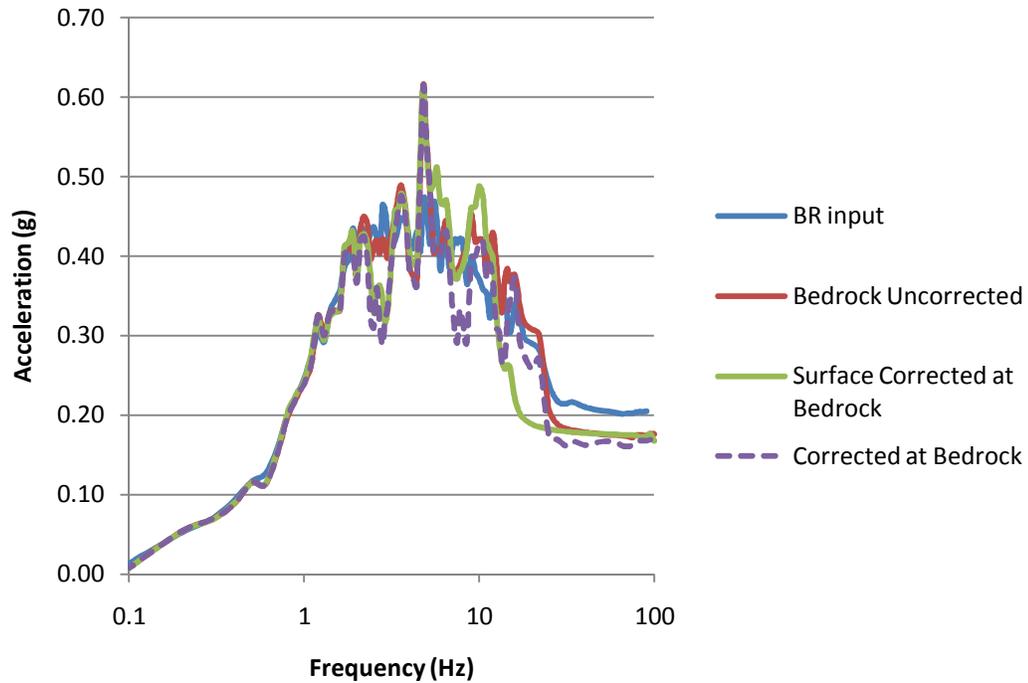
Figure 5.19: Bedrock Responses for Midwestern Soil

5.6 Summary of Key Observations

Generally, the application of the correction factor resulted in an improvement over the current-state-of-practice procedure for developing SSI in-layer responses. However, there are certain limitations regarding wide scale implementation of the correction factor:

- The analysis in Section 5.5 did not use the three BLU soil profiles, as the correction factor is not consistent with the correction factor for the 60 soil profiles.

- The correction factor seemed to work best when applied at the bedrock level rather than at the surface elevations.

- The correction factor can improve the response at the in-layer elevation. Significant improvements can be observed for Case 1, where the input at the free-field surface is corrected at the bedrock level, and some improvement is evident for Case 2 where the input at the surface of the truncated column is corrected at the bedrock level.

- The complex numerical nature of the problem and analysis techniques introduces variability that prevented similar correction as found in the harmonic motion analysis presented in Section 5.4.2.

The in-layer responses are relatively more accurate for the case when the input is defined at the free-field surface of the full soil column compared to the case when the input is defined at the surface of the truncated soil column. The reason for this difference is once again the profile-motion consistency.

In Case 1, where the input is defined at the free-field surface of the full soil column, both the downward propagation to the bedrock level and the subsequent upward propagation to the in-layer location correspond to the same sets of consistent full soil column profiles. However, this does not occur in Case 2, where the input is defined at the surface of the truncated soil column. This is because the downward propagation to the bedrock level occurs through the truncated soil column profiles and the upward propagation to the in-layer is through the full soil column profiles. In this case, the key question that needs to be addressed is related to the calculation of correction factors. Essentially, the correction factor can be calculated using either the truncated soil column or the full soil column. For Case 2,

we used the correction factor corresponding to the truncated soil column because the input is

defined at the surface of the truncated column. Additional work is needed to address this

inconsistency for seismic analyses such as Case 2.

CHAPTER 6: SUMMARY AND CONCLUSIONS

6.1 Summary

This thesis presents a study of the consistency in the current-state-of-practice for the generation of site-specific earthquake response motions. Currently, the evaluation of the site-specific seismic demands may require a SSI analysis, where the design motions must be transferred between elevations. Often this transfer process results in excess response for SSI inputs, and consistency is lost between design spectra and ground motion inputs. This thesis focuses on identifying the reasons for inconsistency in the current-state-of-practice for transferring seismic response between elevations as well as a proposed solution to generate consistent site-specific motions.

The current trend in nuclear power plant design and licensing is for the vendors of new power plants to apply for regulatory approval of a generic seismic design. Ideally, the generic design could be constructed at multiple locations, where the utility only needs to focus on site-specific demands, without a complete relicensing process. However, it is necessary to compare the site-specific demand to the plant capacity. The design capacity of a nuclear power plant is based on the CSDRS, which is a smooth, broad banded spectrum typically defined at the foundation elevation. This spectrum can be generated in several ways, including PSHA analysis or using RG 1.60.

The evaluation of the site-specific seismic demand begins with a geologic survey to identify both soil properties and the locations of any seismic source zones. Next, the survey

is used in a PSHA, which determines the probability of an earthquake of a certain magnitude at the site. The PSHA is designed to account for the risk and uncertainty in the seismic design, and is typically used to generate several UHRS curves for varying return periods. These UHRS can be combined with performance-based design to generate the surface design response spectrum, or DRS.

Demand is defined at one of two elevations, either the free-field surface or at some foundation elevation. For the free-field surface, the seismic demand is defined as the GMRS, which is calculated from the UHRS using performance-based design factors, and typically the GMRS is equivalent to the SSE. For a surface founded plant, the GMRS can be compared directly to the CSDRS. However, for many plants the foundation elevation is at some embedded depth, such that the CSDRS and the GMRS are no longer at the same elevation. In order to compare demand and capacity, the FIRS is generated as a surface motion at the foundation elevation, allowing for a capacity acceptance check. The FIRS can be generated using the same process as the GMRS, with the exception that the soil above the foundation is removed from the analysis.

In the event that the site-specific seismic demands exceed the design capacity of the plant, it may be necessary to perform a SSI analysis. Traditionally, SSI analysis is conducted using the impedance method, where the soil surrounding the structure foundation is simplified into a set of springs and dashpots to model the equivalent stiffness. Typical SSI programs using the impedance method require foundation level in-layer motions, which require transferring the design motion from the foundation elevation. Current-state-of-

practice deconvolves the surface design response down to the bedrock elevation using the three BLU soil profiles. The bedrock motion is then used as an input for the full column, and the in-layer responses can be developed at any required elevation. This study is focused on the transfer of motion for SSI inputs for several reasons. First, USNRC has identified the use of the three BLU soil profiles as a potential source of error, primarily by reducing the variability across the full suite of soil profiles. Additionally, some practitioners have seen significant changes in response during the transfer of surface motion to in-layer motions.

After exploring the current-state-of-practice as described in RG 1.208 and other current regulatory guides, a study is undertaken to identify trends in the motion transfer methods currently available using real-life soil and earthquake data. The surface motions and corresponding design spectra are first determined from a consistent bedrock design spectrum. The response is calculated at the free-field surface, the foundation surface, and the foundation elevation in-layer motions by averaging the response of the 60 soil profiles. The surface design motions are then transferred to the bedrock elevation and then up to the in-layer elevations, according to the current-state-of-practice. All of the elevation responses are compared in order to identify significant trends or differences.

Several observations are made from the study, primarily:

- The average response for the BLU soil profiles is similar to the average response using 60 soil profiles, suggesting that the process of using the three BLU soil profiles is acceptable for motion transfer.

- The motion transfer processes are inconsistent as compared to the direct calculation from the given bedrock motion.

- The inconsistency in the motion transfer is first seen at the bedrock elevation that is obtained from deconvolving the various surface motions to the bedrock.

A closed-form solution for a two layer soil system subjected to harmonic motion is used to isolate the source of inconsistency. Using the displacement transfer functions derived in Equation 4.21 and Equation 4.44, the amplitude of some bedrock input can be transferred to both surface and in-layer motions, following the procedures in the current-state-of-practice.

For a single soil profile, if a bedrock motion is transferred to the free surface and back to the bedrock, then it returns the original input. Similarly, if a suite of soil profiles is used to transfer the motion to the surface, then as long as each individual response is used as an input for the same soil profile, then all of the soil profiles return the original input. However, if the suite of responses at the surface is averaged, and then used as a single input for all of the soils to transfer back to the bedrock, the profiles fail to converge back to the input individually or averaged across all soils.

This identifies a key relationship in soil wave mechanics, a profile-motion consistency. In other words, as long as the motion generated from a specific profile is used with the same profile, then the inputs and outputs will be the same. This is true even for a complex soil column in SHAKE, where an output acceleration used directly as the surface input returns

the original bedrock motion[2]. Fundamentally, the current-state-of-practice violates this consistency by both averaging responses at the surfaces and by using the three BLU soil profiles instead of the complete suite of 60 soil profiles. Typically, transferring the FIRS design spectra is done in a separate analysis, making it necessary to transfer the design motion without the original input arguments and avoiding any added response.

As the violation of profile-motion consistency is the key problem in the current-state-of-practice, a method must be determined to account for the added response in the system. In this thesis we develop a frequency dependent correction factor to account for the added response of a particular soil suite. This correction factor is defined using fundamental wave propagation, as in Equation 5.24.

Additionally, the correction factor is validated numerically. First, a bedrock motion with a unit amplitude is applied to a suite of soil profiles. The individual transfer functions are used to develop 60 surface responses. The mean surface response is then used as the single input amplitude to transfer back to the bedrock, again using all 60 soil columns. The error could be identified directly as the resulting mean bedrock motion does not return the input amplitude. The inverse of this response returns the frequency dependent correction factor developed by Equation 5.5.

Finally, the motion transfer process is repeated, using the correction factor to modify the motion at either the surface or the bedrock. Additionally, the motion transfer is undertaken

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[2] This is true for cases where the output acceleration is used as the input at the top of the soil column. If the output acceleration is transformed into a response spectra and a new corresponding motion is generated, small numerical errors will be introduced into the system.

using all 60 soil profiles, rather than the BLU soil profiles alone. It is found that correcting

the response at the bedrock yields reasonably accurate in-layer motions.

6.2 Conclusions

The key observations made in this study can be used to provide the following primary

conclusions:

- **Inconsistency in Current-State-of-Practice:** As demonstrated in Chapter 4,

  there is inherent consistency between the soil profiles and their respective motion

  generation. This profile-motion consistency is violated in the current-state-of-

  practice both by creating average surface responses and using different soil

  profiles to transfer motion than generate the motion.

- **Development of a Correction Factor:** It is possible to develop a frequency

  dependent correction factor that is soil profile suite and elevation specific. This

  correction factor can be applied to the response at either elevation of the soil

  profile to correctly transfer motion from one surface to another. Additionally,

  the correction factor can be readily calculated for any number of soil layers or

  profiles.

- **Limitations of the Correction Factor**: While the correction factor works

  theoretically, there are two limitations preventing a practical application of the

  correction factor. First, the current processes and procedures used to generate

  and transfer design motions introduces minor numerical error that results in some

variability in the system. Secondly, the correction factor is generated to transfer between two elevations in a soil column, where the current practice needs to transfer between two different soil columns. Thus, the free field motion can be improved, but the correction factor for the FIRS, which is the basis for all SSI analyses, requires further investigation.

- **Use of BLU Soil Profiles:** In the current-state-of-practice, the average responses for the three BLU soil profiles are close to the average response from the 60 soil profiles. However, when the correction factor is determined, the increased number of profiles in the complete soil suite results in a smoother correction factor. The BLU soil profiles do not give a consistent correction factor due to the limited number of soil profiles and their frequency content.

6.3   Recommendations for Future Work

A key area of study would be the potential use of correction factors for the BLU soil profiles as opposed to the complete soil suite. The studies in this thesis suggested a more consistent response by using more profiles. Furthermore, the definition of correction factor can be improved for the case where motion is transferred from a truncated soil column to the full soil column and vice versa. Finally, the results in this thesis have not examined the effects in the higher frequency region.

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Appendices

Appendix A – Development of Current Regulations

Seismic analysis has evolved over the years, changing from a deterministic approach that is updated to a probabilistic approach in the 1990's, and is currently a performance based probabilistic process. The first prevalent approach to seismic analysis is Deterministic Seismic Hazard Analysis (DSHA) (Kramer, 1996). The DSHA can be broken into four steps, starting with an identification of any earthquake sources, either faults or seismic zones and their earthquake potential. Secondly, the distance from the seismic hazard to the site is established, usually the shortest distance from hazard to site. Third, the controlling earthquake is established from the combination of distance and probable earthquake magnitude from each source. Finally, the controlling earthquake is defined at the site in terms of a common analysis parameter such as response spectra, peak acceleration, or peak velocity. There are two main limitations of a DSHA: first that there is no determination of the frequency of earthquake damage or the number of earthquakes to exceed the design earthquake during the lifetime of the site, and secondly, the determination of earthquake potential in the first step can be highly subjective.

The DSHA is replaced by the PSHA which is in many respects similar to the DSHA. The first step is again to define any earthquake sources, but to also determine the probability of an event along each source zone. Often these probabilities can be uniform across a fault or other source, but when combined with the source geometry develop a probability estimate for source and distance, rather than assuming the earthquake to only occur at the closest

point to the site. The second step is to develop a recurrence relationship, which is the

expected rate that some design earthquake will be exceeded, which allows for the

consideration of multiple earthquakes rather than the DSHA focusing only on the maximum

earthquake. Third, the source motion is shifted to the ground motion at the site for any

possible earthquake from any source location, with uncertainty in predictive relationships

included. Lastly, the uncertainties from the first three steps – the earthquake location, size,

and ground motion prediction – are combined to obtain a probability of ground motion

exceedance for a specified return period.

In 1997, the NRC issued Regulatory Guide (RG) 1.165, "Identification and

Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake

Ground Motion," that first defined probabilistic methods for determining the SSE at nuclear

plant sites. Rather than applying the previously deterministic approach, RG 1.165 outlines

an approved approach to determining the site-specific geological properties and seismic

demands, conduct a PSHA, and then determine the SSE that satisfies 10 CFR 100.23

(USNRC, 1997). Following the PSHA, RG 1.165 determines a "Reference Probability" that

is based on the probability that 50% of a set of selected current plants would have an annual

median frequency of exceeding the SSE (thus failure) of the future plant. This reference

probability is then factored into developing the SSE.

The most recent change in seismic analysis is due to ASCE 43-05, *Seismic Design

Criteria for Structures, Systems, and Components in Nuclear Facilities*, and it is adopted in

NRC Regulation 1.208. ASCE 43-05 introduced performance-based goal design, rather than

a deterministic approach to design. Instead of a reference probability, the plant is instead designed to a performance goal, which in the case of nuclear power plants, is the target value of $1x10^{-5}$ mean annual probability of exceedance for inelastic deformation. Instead of referencing other plants' relative response, the code defines a performance goal, hazard exceedance probability, and probability ratio that can be adjusted based on the structure being analyzed.

With the addition of performance goals and the desire for using certified designs, it is necessary to check that the seismic demand at proposed sites does not exceed the CSDRS. Through the process outlined in RG 1.208, the output from the geological investigation and PSHA is the UHRS which is a smoothed response from the strong motion at the bedrock layer. The UHRS is shifted to the surface and scaled by a design factor, becoming the GMRS, a site-specific motion found in the free field on the ground surface, and is the SSE assuming it meets all of the requirements in 10 CFR 100.23 and 10 CFR 50 Appendix S (USNRC, 2007). Furthermore, the GMRS must be enveloped by the CSDRS to show that the demand is less than the design basis.

Since the publication of RG 1.208 there have been several clarifications issued by both USNRC and the private industry. Initially, NRC published *Interim Staff Guidance on Seismic Issues Associated with High Frequency Ground Motion in Design Certification and Combined License Applications*, or ISG-1. In ISG-1, the three primary ground motions – CSDRS, GMRS, and FIRS – are defined as above, detailing that the CSDRS is the design basis of the plant and should not be exceeded, and that the SSE for the site is the GMRS. In

regards to the relationship of the FIRS to the GMRS, both are defined as free field outcrop spectra, and the transfer of the GMRS to the FIRS should include the entire soil column down to the effective uniform half space (USNRC, 2008a).

In order to clarify the process briefly mentioned in ISG-1, two methods are presented for approval in the newly drafted *Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses* or ISG-17. ISG-17, currently open for comments, combines a white paper from the Nuclear Energy Institute (NEI), "Consistent Site-Response/Soil-Structure Interaction Analysis and Evaluation," and Brookhaven National Laboratory (BNL) report, "Consistent Site Response-SSI Calculations." When comparing the CSDRS to the FIRS, ISG-17 references the NEI white paper and three specific structure/foundation situations. If the design requires SSI analysis, either the method presented in the NEI white paper or the BNL report can be used.

In order to compare the varying methods, it is necessary to define additional ground motions. As described earlier, the licensing process depends upon the CSDRS, GMRS, and FIRS. However, based on the elevation of the foundation, GMRS, and FIRS, there are various methods to transfer the motions to consistent elevations. Additionally, it is assumed that the UHRS is defined at the effective uniform halfspace, or the bedrock level. It can be transferred to the surface elevation and scaled to calculate the GMRS. The following motions are defined in the NEI whitepaper and the BNL report (2009):

- *Geologic Outcrop Spectra (GO):* A GO is a generic term for the free field surface response at the top of a soil column. Any soil above the elevation of

interest is removed, such that the response is truly the surface response. FIRS should be defined as a GO at the foundation elevation.

- *Full Column Outcrop Spectra (FCO):* The FCO is an artificial outcrop motion defined at some elevation inside of a full soil column. The ground motion is comprised of an incident and reflected wave, which at a free surface would be equal to twice the incident wave. In SHAKE and other software programs, the FCO is defined as twice the incident wave, but is impacted by the response of the soil above the elevation of interest, typically leading to a non conservative response. Per the discussion in the BNL report, the 'correct' ground motion should be defined as a GO, not an FCO.

- *Full Column Inlayer Spectra (FCI):* The FCI is the true response of the soil at a given elevation, found by summing the response of the incident and reflected wave. For some SSI programs, the preferred input is the foundation level FCI, and not FIRS.

- *Performance-Based Surface Response Spectra (PBSRS):* The PBSRS is the spectra developed at the surface of a full column. When the GMRS is defined at the free surface, then the PBSRS and the GMRS should be the same spectra.

The comparison of CSDRS, GMRS, and FIRS in the NEI and BNL reports can be broken into three main categories: surface structures, embedded structures with CSDRS defined at the foundation elevation, and embedded structures with CSDRS defined at the surface elevation. For the case of surface structures, CSDRS, GMRS, and FIRS are all

defined at the free surface of the soil column. As such, GMRS and FIRS would be equal, so a direct comparison to CSDRS is possible.

For an embedded structure with the CSDRS defined at the foundation elevation, it is necessary to develop FIRS at the foundation level for comparison to the CSDRS as well as convolve the FIRS to the surface to compare to the PBSRS. The process for developing the FIRS is similar to that of developing the GMRS. Starting with a rock elevation UHRS, using 60 randomized profiles, the GMRS is based on the mean surface response, where the FIRS is a GO found by using the same soil columns with the soils above the elevations truncated. It is important to note that as the UHRS and GMRS are developed, the random soil profiles go through strain iterations that account for overburden stresses and the weight of the soil above the foundation. For the development of the FIRS, the strain iterated soil profiles should be used with no strain iteration to properly incorporate the weight of the soil above the foundation level. The FIRS should then be enveloped by the CSDRS at the foundation elevation. Additionally, the FIRS should be convolved to the surface using the BLU soil suite and shown to envelope the PBSRS.

For embedded structures with the CSDRS defined at the surface elevation, the BNL and NEI report diverge, although ISG-17 accepts the NEI position. The NEI procedure dictates that the CSDRS should be deconvolved to the foundation level, and this CSDRS-based foundation spectrum should be compared to the FIRS. In contrast, the BNL report suggests that the CSDRS should be compared to the GMRS at the surface, and the FIRS only checked to meet minimum response requirements. Furthermore, a key difference is that the NEI

paper suggests that FCO spectra should be used during the analysis, which is in direct conflict with the BNL position that FCO spectra are inherently flawed and artificial. This difference is not discussed in ISG-17, and seems to be a critical oversight.

While the two procedures differ in the appropriate evaluation of the CSDRS, the input into the SSI is of critical importance. For a surface structure, the FIRS/GMRS can be used directly for ground motion input. The BNL report also suggests that a surface defined GMRS can be used as the input for SSI analysis. For cases where the SSI program can handle foundation outcrop responses, the NEI white paper uses the FIRS outcrop motion as the input, although it uses an appropriate GO FIRS for foundation level CSDRS versus an FCO FIRS for a surface defined CSDRS.

The more complicated case is when the SSI program needs an in-layer foundation response instead of an outcrop foundation response, a problem for commonly used programs such as SASSI. For this case, both methods deconvolve the FIRS down to the uniform halfspace using the BLU soil profiles. The outcrop response at the halfspace is then used as input in a full soil column where the FCI is again found using the BLU soil profiles and can then be used as the input motion. Another alternative is to use the halfspace inputs and calculate the surface response of the full column and use the corresponding ground motion for the SSI calculations.

Appendix B – Soil Profile Properties
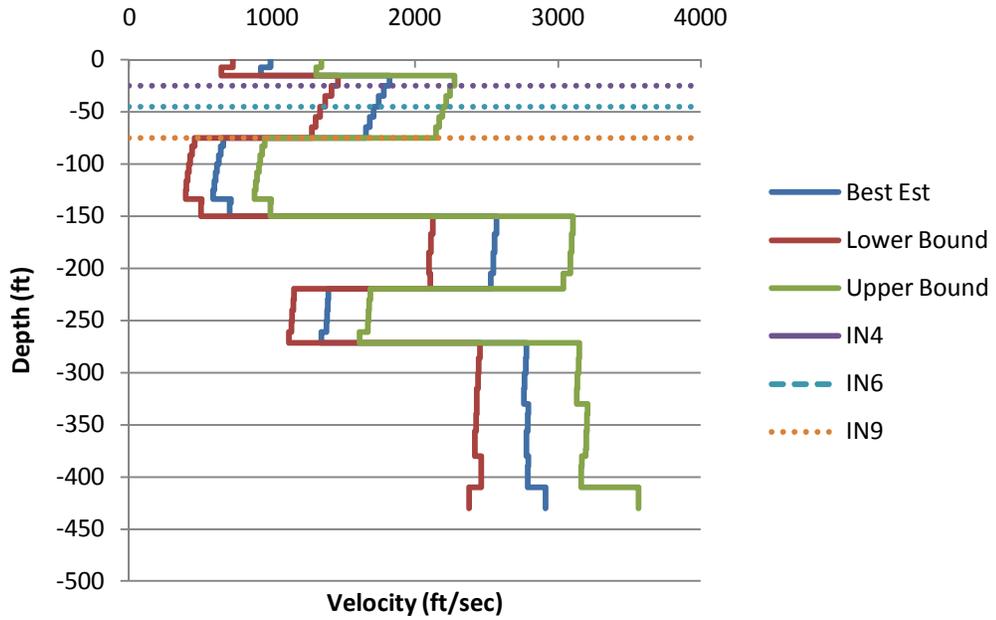
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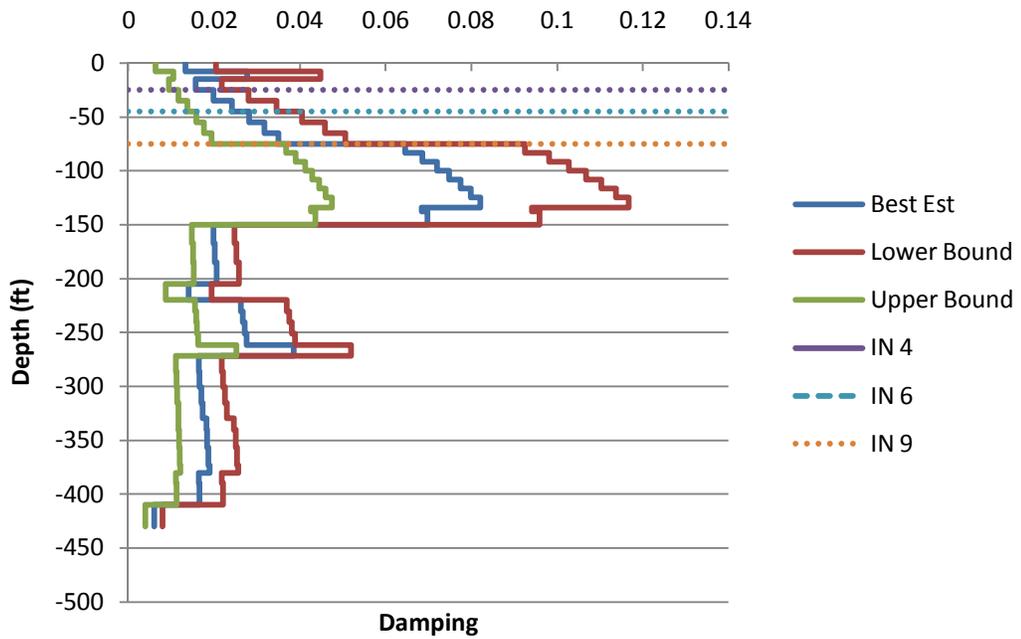

Figure B.2: Western BLU Soil Damping Profiles
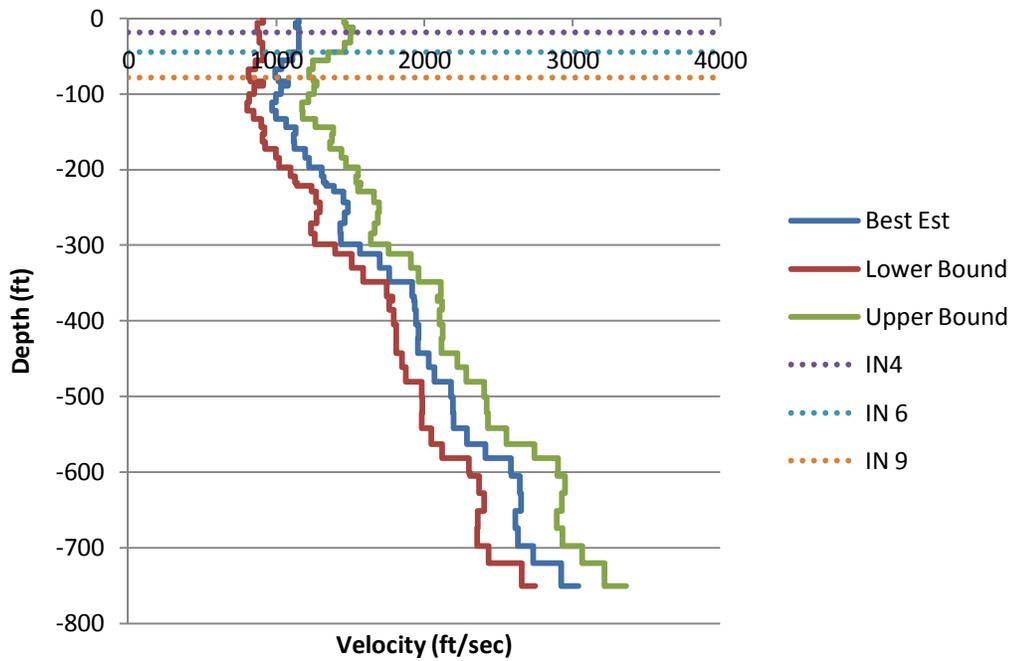
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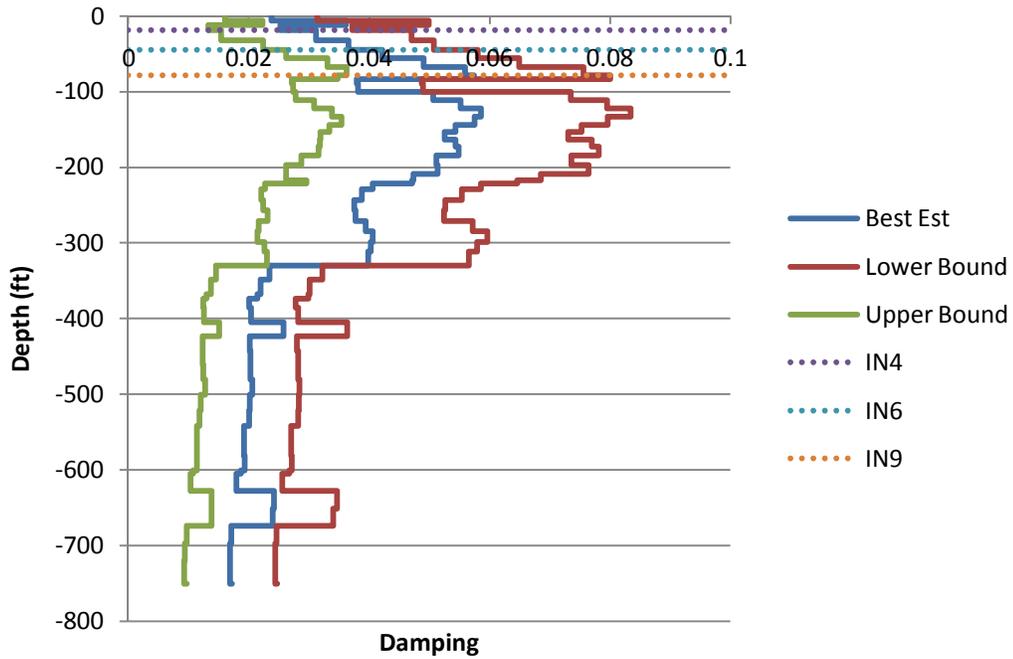

Figure B.4: Eastern BLU Soil Damping Profile

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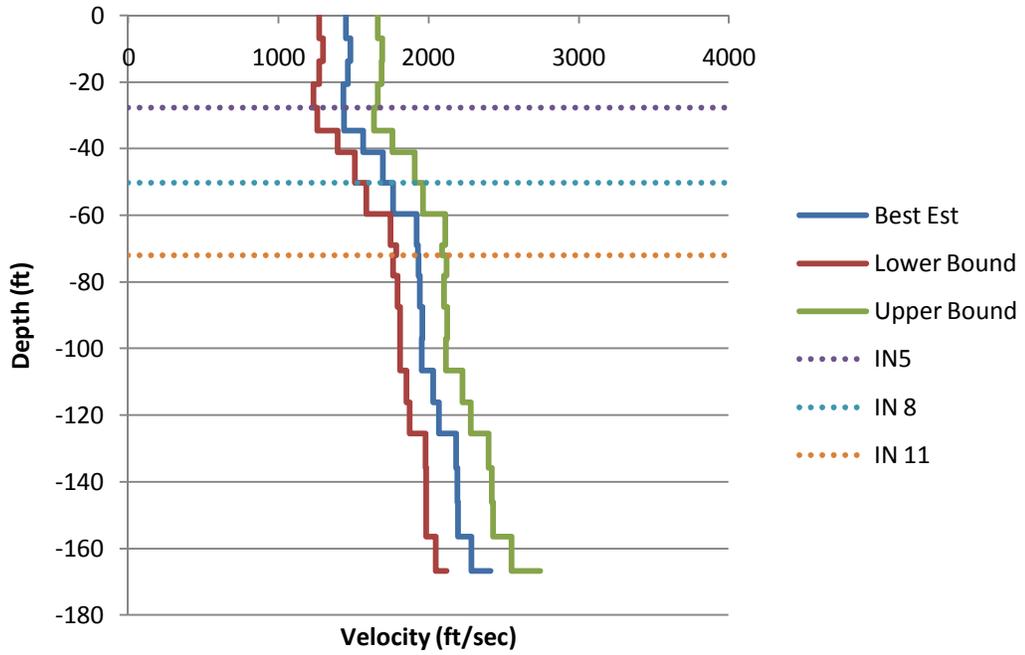
Figure B.5: Midwestern BLU Soil Shear Wave Velocity Profiles
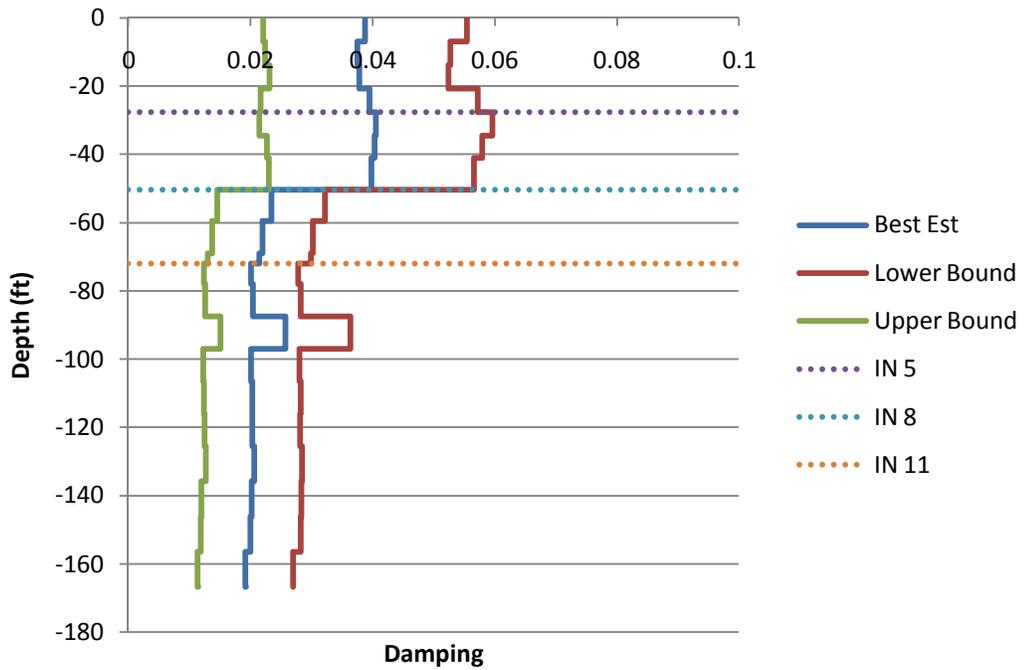
Figure B.6: Midwestern BLU Soil Damping Profiles