

**Site-Specific Seismic Ground Motion  
Analyses for Transportation Infrastructure  
in the New Madrid Seismic Zone**

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## ABSTRACT

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Seismic design forces generally govern the design of costly transportation infrastructure in Northeast Arkansas. This region lies in the heart of the New Madrid Seismic Zone, where extreme seismic demands and unfavorable subsurface conditions present significant seismic hazards that engineers must account for in design. In general, code-based design procedures cannot account for the anticipated short-period attenuation and long-period amplification of earthquake ground motions in the deep, soft sediments of the Mississippi Embayment. As a result, generic, code-based designs may lead to short-period structures being over-designed at a significant cost, and long-period structures being under-designed at a significant risk. Most AHTD bridges are in the short-period range (i.e., 0.1-0.5 seconds), where site specific analyses can potentially allow engineers to reduce seismic design forces by up to 33%. For these reasons, AASHTO explicitly recommends site-specific ground motion response analyses for this part of the country.

Site-specific ground motion response analyses have been conducted for an example site in Blytheville, AR, where a railroad overpass bridge (AHTD Bridge No. 07204) was designed previously using the generic, code-based procedures and recently constructed at a cost of approximately \$11 million. This example site was chosen in order to demonstrate the feasibility of conducting site-specific analyses at a location where the seismic hazard is particularly extreme. Results from the site-specific analyses discussed herein clearly show that the seismic design forces for the example bridge site could have been reduced by 33% if these analyses had been performed prior to design. This reduction could have produced enormous cost savings. Similar results are expected for short-period bridges throughout Northeast Arkansas, where probabilistic seismic hazards are generally dominated by a single earthquake scenario and subsurface conditions are relatively homogenous.

The site-specific ground motion response analyses performed in this study follow a “Level 1” approach to seismic hazard preservation. Wherein, seismic hazard is determined for bedrock, and approximately maintained by using engineering judgment to explicitly quantify uncertainties in soil properties. Seismic hazard was established using the 2002 USGS Seismic Hazard Maps to develop a 7% in 75-year uniform hazard spectrum for the region’s stiff Paleozoic bedrock. A suite of 25 input “rock” motions were developed using ground motions from McGuire et al. (2001), which were modified to represent the expected frequency content of ground motions in the central and eastern United States (CEUS), and spectrally matching to the site-specific uniform hazard “target spectrum”. Four  $V_s$  profiles were combined with three sets of dynamic soil properties to construct 12 subsurface soil profiles, collectively quantifying best estimates and reasonable ranges of soil properties. These 12 soil profiles were utilized in both equivalent-linear (EQL) and fully nonlinear (NL) analyses using DEEPSOIL v5.0. In all, this feasibility study comprises 600 preliminary analyses, 100 sensitivity analyses regarding depth to bedrock, and 600 design-level analyses. The design-level analyses were weighted to produce a sensible and conservative estimate of surface ground motions.

## TABLE OF CONTENTS

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Abstract.....	ii
Table of Contents.....	iii
List of Figures.....	v
List of Tables.....	ix
1. Introduction.....	1
2. Literature Review.....	4
2.1. Soil Profile Development.....	4
2.2. Input Earthquake Ground Motions.....	11
2.2.1. Target Spectrum.....	12
2.2.2. Ground Motion Selection and Scaling.....	13
2.3. Site Response Analyses.....	16
3. Information Needed for Site-Specific Ground Motion Response Analyses.....	20
3.1. Shear Wave Velocity Profiles.....	20
3.2. Unit Weight (Density) Profiles.....	24
3.3. Dynamic Soil Properties: Modulus Reduction and Damping Curves.....	24
4. Input Ground Motions.....	33
4.1. Site-Specific Target Spectrum.....	33
4.2. Input Ground Motion Selection.....	33
4.3. Spectral Matching.....	38
5. Framework for Site Response Analyses.....	43
5.1. Input Alternatives and Analysis Methods Used in Site Response Analyses.....	43
5.2. Weighting Factors for Results from Design-Level Analyses.....	46
6. Equivalent Linear Site Response Analyses.....	49
6.1. Influence of Dynamic Soil Properties.....	49
6.2. Influence of $V_s$ Profile.....	51
6.3. Sensitivity of Surface Response Spectrum to Depth to Bedrock.....	53
6.4. Remarks on EQL Results.....	53
7. Fully Non-Linear Site Response Analyses.....	56
7.1. Influence of Dynamic Soil Properties.....	56

7.2.	Influence of $V_s$ Profile.....	58
7.3.	Sensitivity of Surface Response Spectrum to Depth to Bedrock.....	60
7.4.	Remarks on NL Results.....	60
8.	Development of Design Response Spectrum at Ground Surface.....	62
8.1.	Computation of the Design Response Spectrum.....	62
8.2.	Discussion of the Design Response Spectrum.....	65
9.	Conclusions.....	66
9.1.	Discussion.....	66
9.1.1.	Collection and Modification of Input “rock” Ground Motions.....	66
9.1.2.	Dynamic Soil Properties.....	66
9.1.3.	$V_s$ Profiles.....	67
9.1.4.	Depth to Bedrock.....	67
9.1.5.	Site Response Analyses: NL vs. EQL.....	67
9.2.	Conclusions.....	68
	References.....	70

## LIST OF FIGURES

---

Figure 1.	Design ground motions (5% damped acceleration response spectra for ground motions with a 5% probability of exceedance in 50 years) for Blytheville, AR and other seismically active cities in the United States (all assuming AASHTO Site Class B). .....	2
Figure 2.	AASHTO code-based design acceleration response spectra (5% damped ground motions with a 5% probability of exceedance in 50 years) for Blytheville, AR.....	3
Figure 3.	Schematic of site-specific ground motion response analysis: (a) discretize soil profile and determine soil properties; (b) obtain appropriate input “rock” ground motions, and use computer software to simulate upward propagation through the soil column and compute response; (c) develop design response spectrum based on results and restricted to no less than 2/3 of the AASHTO code-based response spectrum. ....	5
Figure 4.	(a) Plan view of the Mississippi Embayment and major geologic structures and (b) E-W section through Memphis (Hashash and Park, 2001). .....	6
Figure 5.	Compiled and average $V_s$ profiles in the Mississippi Embayment (Romero and Rix, 2001). .....	7
Figure 6.	Average and smoothed $V_s$ profile with assumed geologic layering (Romero and Rix, 2001). .....	8
Figure 7.	Locations of deep shear wave velocity profiles measured in Mississippi Embayment by Rosenblad et al. (2010). .....	10
Figure 8.	Combination of (a) deep $V_s$ profiles from Rosenblad et al. (2010) and (b) full-depth reference $V_s$ profile from Rix and Romero (2005) for (c) site-specific full-depth $V_s$ profile (Rosenblad and Goetz, 2010). .....	10
Figure 9.	EPRI material property curves for (a) shear modulus reduction, and (b) damping (Zheng et al. 2010). .....	12
Figure 10.	UHS for generic reference conditions (Site Class B/C) and the Site Class A UHS (target spectrum for input ground motions, computed using scaling factors in Table 2) for stiff Paleozoic bedrock in Blytheville, AR. ....	14
Figure 11.	Deaggregation of zero-second period (PGA) ground motions with a 5% probability of exceedance in 50 yrs for Blytheville, AR (2002 USGS Probabilistic Seismic Hazard Analysis). .....	15
Figure 12.	Example of (a) scaling and (b) spectrum-matching of input ground motions to a target spectrum.....	17
Figure 13.	Design response spectrum from site-specific ground motion response analysis in Osceola, Arkansas (Zheng et al., 2010). .....	19

Figure 14.	Location of example bridge site for site-specific ground motion response analysis: AHTD bridge No. 07204 from job No. 100705. ....	21
Figure 15.	Locations of four $V_s$ profiles (Site 2, Site 3, Site 4 and Site 11) measured by Rosenblad et al. (2010) relative to the example bridge site.....	21
Figure 16.	650-foot $V_s$ profiles in Northeast Arkansas measured by Rosenblad et al. (2010). ..	22
Figure 17.	Full-depth $V_s$ profiles developed for the example bridge site in Blytheville, Arkansas: lognormal median and plus/minus 20% COV from four $V_s$ profiles developed by Rosenblad et al. (2010).....	23
Figure 18.	Example of procedure for modifying Darendeli (2001) modulus and damping curves to account for implied shear strength at large strains in a cohesive soil layer.....	28
Figure 19.	Example of procedure for modifying Darendeli (2001) modulus and damping curves to account for implied shear strength at large strains in a cohesionless soil layer. ...	29
Figure 20.	Shear strain profiles from preliminary site response analyses, showing large strains in the top 100 feet of the soil column. ....	30
Figure 21.	Shear stress profiles from preliminary site response analyses, indicating possible shear failure in top 100 feet of the soil column. ....	31
Figure 22.	Java ground motion parameter calculator for developing a uniform hazard spectrum: (a) select “Probabilistic hazard curves”; (b) select “Conterminous 48 States”; (c) select “2002 Data”; (d) select the “Lat/Lon” tab and enter the coordinates of the job site; (e) select the desired period and click “Calculate” in the “Basic Hazard Curve” section; (f) select the “Prob. & Time” tab, select “5” for Prob. Of Exceedance and “50” for Exposure Time, and click “Calculate” in the “Single Hazard Curve Value” section; record the calculated ground motion and associated period. ....	34
Figure 23.	Interactive deaggregation application for 2002 USGS Seismic Hazard Maps: (a) enter name of structure or site; (b), (c) enter latitude and longitude in decimal degrees; (d) select “5% in 50 years” as per AASHTO specifications; (e) select period for deaggregation; (f) option for geographic representation of results; (g) option to generate synthetic input ground motions; (h) click button to perform the deaggregation.....	35
Figure 24.	Mean deaggregation results for the Blytheville example bridge site obtained from USGS interactive deaggregation of the 2002 probabilistic seismic hazard analysis at each period available for calculation. ....	36
Figure 25.	Input ground motion response spectra (5% damping) selected for the example bridge site in Blytheville, AR with lognormal median and target spectrum. ....	39
Figure 26.	Settings used in RspMatch 2009 to spectrally match ground motions to the target spectrum for input ground motions.....	40

Figure 27. Effects of spectral matching on input ground motion XXV.....	41
Figure 28. Spectrally matched input ground motion response spectra (5% damping) and target spectrum for site response analyses at the example bridge site in Blytheville, AR ..	42
Figure 29. Equivalent linear analyses with all 25 output response spectra (gray lines) and the computed lognormal median, including lognormal median of the 25 input response spectra and the code-based Site Class E and 2/3 Site Class E Spectra.....	45
Figure 30. Logic tree for calculating a single 5% damped surface response spectrum from the 24 runs corresponding to 600 surface response spectra analyses for the example bridge site in Blytheville, Arkansas.....	47
Figure 31. Acceleration response spectra from equivalent linear site response analyses with fixed $V_s$ profile and varying dynamic soil properties for the example bridge site in Blytheville, AR.....	50
Figure 32. Shear strain profiles from equivalent linear site response analyses with fixed $V_s$ profile and varying dynamic soil properties for the example bridge site in Blytheville, AR.....	51
Figure 33. Acceleration response spectra from equivalent linear site response analyses with varying $V_s$ profiles and fixed dynamic soil properties for the example bridge site in Blytheville, AR.....	52
Figure 34. Shear strain profiles from equivalent linear site response analyses with varying $V_s$ profiles and fixed dynamic soil properties for the example bridge site in Blytheville, AR.....	52
Figure 35. Acceleration response spectra from equivalent linear site response analyses with varying depths to bedrock and fixed dynamic soil properties for the example bridge site in Blytheville, AR.....	54
Figure 36. Shear strain profiles from equivalent linear site response analyses with varying depths to bedrock and fixed dynamic soil properties. $V_s$ profile (not on a logarithmic scale) for the example bridge site in Blytheville, AR.....	54
Figure 37. Acceleration response spectra from non-linear site response analyses with fixed $V_s$ profile and varying dynamic soil properties for the example bridge site in Blytheville, AR.....	57
Figure 38. Shear strain profiles from non-linear site response analyses with fixed $V_s$ profile and varying dynamic soil properties for the example bridge site in Blytheville, AR.....	57
Figure 39. Acceleration response spectra from non-linear site response analyses with varying $V_s$ profiles and fixed dynamic soil properties for the example bridge site in Blytheville, AR.....	59

Figure 40. Shear strain profiles from non-linear site response analyses with varying $V_s$ profiles and fixed dynamic soil properties for the example bridge site in Blytheville, AR. ..	59
Figure 41. Acceleration response spectra from non-linear site response analyses with different depths to bedrock and fixed dynamic soil properties for the example bridge site in Blytheville, AR. ....	61
Figure 42. Shear strain profiles from non-linear site response analyses with varying depths to bedrock and fixed dynamic soil properties (note, $V_s$ profile is not on a logarithmic scale) for the example bridge site in Blytheville, AR. ....	61
Figure 43. Surface response spectra and weighted average from EQL site response analyses for the example bridge site in Blytheville, AR. ....	63
Figure 44. Surface response spectra and weighted average from NL site response analyses for the example bridge site in Blytheville, AR. ....	63
Figure 45. Site-specific surface response spectrum developed from EQL and NL analyses for the example bridge site in Blytheville, AR. ....	64
Figure 46. Spectral ratios of surface response to input “rock” motions for the example bridge site in Blytheville, AR. ....	64
Figure 47. Design surface response spectrum from site-specific ground motion response analysis for the example bridge site in Blytheville, AR, delineated along the AASHTO suggested minimum reduction. ....	65

**LIST OF TABLES**

---

Table 1. Velocity models developed for the Mississippi Embayment (Romero and Rix, 2001) ..... 9

Table 2. Scaling factors for converting AASHTO Site Class “B/C” ground motion amplitudes to AASHTO Site Class “A” ground motion amplitudes ..... 14

Table 3. Assumed relationship used for assigning total unit weight values based on shear wave velocity ..... 25

Table 4. Depths, soil properties and target strength values for five aggregate layers (curve sets 1 – 5) defining the top 100 feet of the soil profiles ..... 32

Table 5. Output from java ground motion parameter calculator and conversion to Site Class A to develop UHS target spectrum for example bridge site in Blytheville, AR ..... 36

Table 6. Summary of input ground motions selected after screening the suite of modified ground motions by McGuire et al. (2001) ..... 37

Table 7. Summary of run numbers assigned to each fully non-linear (NL) site response analysis for the example bridge site in Blytheville, Arkansas..... 44

Table 8. Summary of run numbers assigned to each equivalent linear (EQL) site response analysis for the example bridge site in Blytheville, Arkansas..... 44

Table 9. Summary of run numbers assigned to each NL site response analysis conducted to study sensitivity of depth to bedrock for the example bridge site in Blytheville, AR44

Table 10. Summary of run numbers assigned to each NL site response analysis conducted to study sensitivity of depth to bedrock for the example bridge site in Blytheville, AR45

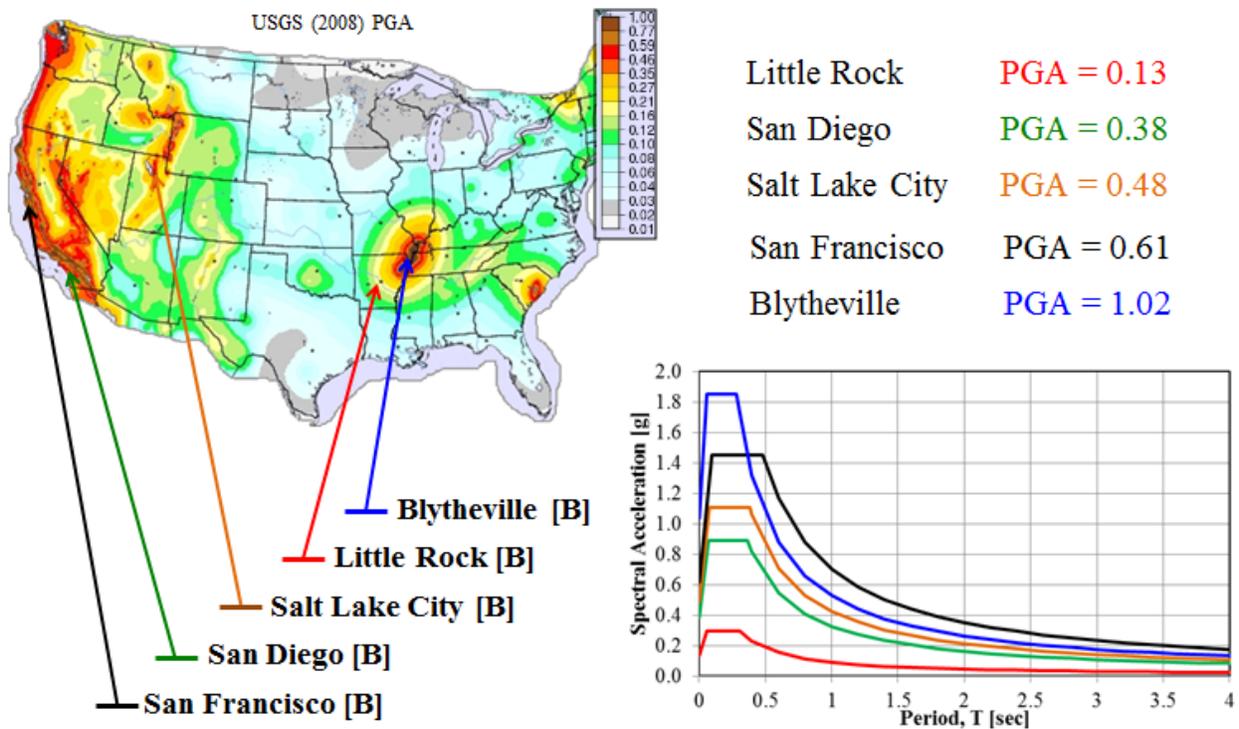
## 1. INTRODUCTION

This study was made possible by funds from the Mack Blackwell Rural Transportation Center (MBTC) and matching funds from the Arkansas State Highway and Transportation Department (AHTD). Both funding agencies have a significant interest in the state of seismic design in Northeast Arkansas because it lies in the New Madrid Seismic Zone (NMSZ). Earthquake design ground motions in this region are very large, even compared to other seismically active areas of the nation (Figure 1). Extreme seismic forces in the NMSZ often govern the design of bridges and other transportation infrastructure. AASHTO Guide Specifications for Seismic Bridge Design (2009) allow standard code-based seismic design forces to be reduced as much as 33% if site-specific ground motion response analyses are performed. The purpose of this study is to develop guidelines for conducting site-specific ground motion analyses in the NMSZ (particularly Northeast Arkansas) that may result in reduced seismic design forces and significant cost savings.

Northeast Arkansas is a unique seismic design region for two reasons: (1) seismic design ground motions in the heart of the NMSZ are very large, and (2) the geology of the Mississippi Embayment is unlike any other seismic region in the nation. Large design ground motions in the area are a direct result of three powerful earthquakes that occurred in the winter of 1811 – 1812. Seismologists generally agree that the magnitudes of those earthquakes were between 7.0 and 8.0. However, there are no ground motion records from the events for verification of the magnitudes or quantification of the shaking intensity. The last significant seismic event in the area was a magnitude-6.0 earthquake in 1895, from which there are also no physical ground motion recordings. Geologic investigations indicate that large earthquake sequences like the 1811-1812 events occur every 500 – 1,000 years (USGS, 2008). However, there is significant uncertainty regarding the magnitude, recurrence interval and shaking intensity for seismic events in the NMSZ.

Geology in the Mississippi Embayment consists of deep, soft soil deposits. The depth of soft soils ranges from several hundred to as much as one thousand meters. After the 1985 Michoacán earthquakes in Mexico City, earthquake engineers realized that soft soils propagate seismic ground motions much differently than stiff soils or rock (Dobry and Vucetic, 1987). For soft soils, high-frequency (short-period) energy is attenuated (damped out) while low-frequency (long-period) energy is amplified. In general, high-frequency energy does more damage to smaller structures while low-frequency energy is most damaging to large structures. In Northeast Arkansas, most standard bridge designs have relatively short natural periods of vibration (i.e., generally in the range of 0.1 – 0.5 seconds). Meaning, the seismic design is governed by high-frequency energy (2 – 10 Hz), most of which should attenuate in the deep soil deposits.

Code-based design ground motion procedures do not account well for the extensive damping and nonlinear soil response that can be expected in Northeast Arkansas, resulting in potentially excessive design ground motions in the short-period range. This is because a typical code-based design requires a site to be assigned into one of five generalized Seismic Site Classifications (i.e., Site Class A – E). These generalized subsurface conditions range from hard

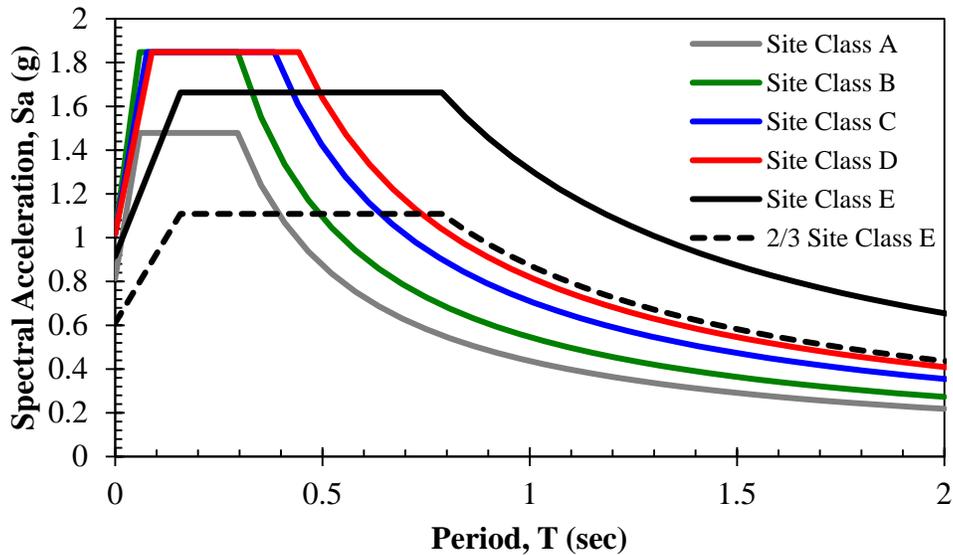


**Figure 1. Design ground motions (5% damped acceleration response spectra for ground motions with a 5% probability of exceedance in 50 years) for Blytheville, AR and other seismically active cities in the United States (all assuming AASHTO Site Class B).**

rock (Site Class A) to soft clay soils (Site Class E). The deep, soft soil conditions of the Mississippi Embayment certainly fall outside the generalized, code-based Seismic Site Classification system, which is based only on the soil properties within the top 100 feet of the ground surface. As shown in Figure 2, the AASHTO Guide Specifications only account for a relatively small amount of high-frequency attenuation at periods less than 0.5 seconds for soft, Site Class E soil deposits, when compared to the other Site Classifications. AASHTO Guide Specifications acknowledge that code-based design ground motions at deep, soft soil sites may be overly conservative for short-period bridges as follows:

“... site conditions may also warrant a site-specific ground motion evaluation, particularly for locations that are outside the range of the original site class development. [...] Deep soil deposits have a pronounced yet not fully understood influence on the amplification or deamplification of ground motions. Areas encountering this effect include central and eastern United States. [...] If deep soil deposits are not considered in developing response spectra for these sites, longer-period bridges may be under-designed and shorter period bridges may be over-designed.”

Site-specific ground motion response analyses are required in order to justify lower short-period/high frequency design ground motions.



**Figure 2. AASHTO code-based design acceleration response spectra (5% damped ground motions with a 5% probability of exceedance in 50 years) for Blytheville, AR (showing Site Classes A – E and 2/3 E).**

AASHTO Guide Specifications allow two types of site-specific analyses. Engineers may either evaluate the seismic hazard (site-specific seismic hazard analysis) or the ground motion response (site-specific ground motion response analysis). The first option is not addressed in this study, but may be of interest for future research. Herein, attention is focused on predicting the response of deep soil deposits in Northeast Arkansas using site-specific ground motion response analyses. As noted above, the AASHTO Guide Specifications allow the code-based design ground motions to be reduced by as much as 33% if site-specific analyses reveal it is appropriate. Meaning, designers could potentially use the 2/3 Site Class E spectrum shown in Figure 2 rather than the standard Site Class E spectrum. This study focuses on a recently constructed bridge in Blytheville, AR in order to demonstrate the feasibility of using site-specific ground motion response analyses to significantly reduce design ground motions for short-period bridges.

An important distinction between *design ground motions* and *seismic hazard* is now drawn: Properly conducted site-specific ground motion response analyses may reduce ground motions for design, but do not alter the seismic hazard. McGuire (2001) and NUREG (2002) describe several approaches to preserving the seismic hazard as seismic energy is modeled from bedrock up to the ground surface. The simplest approach (Level 1) defines hazard according to the bedrock conditions, and attempts to maintain that hazard level by explicitly considering soil uncertainties. The most robust approach (Level 4) comprises a probabilistic assessment of both bedrock and soil hazard at multiple frequencies. The site response analyses conducted for this study are all classified as “Level 1”. Meaning, attempts have been made to account for uncertainty in the input ground motions, dynamic soil properties and methods of analysis. However, these efforts were made based on engineering judgment rather than rigorous attempts at preserving probability in the earthquake ground motions from the bedrock to the ground surface. This Level 1 approach is deemed appropriate for the current study, where a site-specific seismic hazard analysis was not performed.

## 2. LITERATURE REVIEW

Site-specific ground motion response analyses involve: (1) characterizing a soil profile down to bedrock, (2) collecting and adjusting appropriate “rock” input earthquake acceleration time histories (horizontal ground motions), and (3) using numerical analyses to simulate the propagation of the ground motions from bedrock up to the ground surface. If performed rigorously and correctly, the resulting response spectrum can be used in design rather than the basic code-based response spectrum. However, the design response spectrum obtained from site-specific analyses may not be taken as less than 2/3 of the AASHTO code-based spectrum. A schematic of the overall process is presented in Figure 3. Each step requires multiple sources of information and decisions on the part of the engineer.

### 2.1. SOIL PROFILE DEVELOPMENT

Site response studies in the Mississippi Embayment have generally relied on geologic mapping to develop soil profiles for site response analyses. The region comprises a deep basin of stiff Paleozoic rock overlain by soft interbedded sediments. Hashash and Park (2001) illustrated the general layout of the Mississippi Embayment (Figure 4) based on the work of Ng et al. (1989). Dart and Swolfs (1998), Van Arsdale and TenBrink (2000) and others have used well logs and seismic refraction data to develop more precise structural contour maps of the basement floor and overlying stratigraphy. These maps can be used to estimate the depth to bedrock at any given site in the NMSZ.

Numerous investigators have measured shear wave velocity ( $V_s$ ) profiles in the Mississippi Embayment. However, most measurement techniques are incapable of sampling the full depth of soft sediments. Romero and Rix (2001) summarized the full-depth shear wave velocity profiles that had been developed in the Mississippi Embayment at that time (Table 1). As shown in Figure 5, they calculated an average of those  $V_s$  profiles to generate a reference  $V_s$  profile for site response analyses. The key assumption for Romero and Rix (2001) was that shear wave velocity is constant through each layer of the geologic profile developed by Van Arsdale and TenBrink (2000). The average profile was also smoothed to avoid large impedance contrasts between adjacent layers, Figure 6. This smoothed and averaged  $V_s$  profile was proposed as a full-depth reference profile for the entire region.

In the same study, Romero and Rix (2001) developed several characteristic  $V_s$  profiles to a depth of 70 meters for regions surrounding Memphis, TN. The profiles were developed based on a compilation of previously measured data. Characteristic  $V_s$  profiles were developed in far northeast Arkansas, northwest Mississippi and west Tennessee for two geologic site conditions: (1) sites within present river meanders, and (2) backswamp regions or older meanders. The characteristic  $V_s$  profiles for Tennessee and Mississippi were each based on six measured profiles. Eight measured profiles were used for the two characteristic  $V_s$  profiles in Arkansas. The  $V_s$  profiles compiled in that study were originally measured using a variety of test procedures including seismic CPT, downhole, crosshole, seismic refraction and spectral-analysis-of-surface-waves.

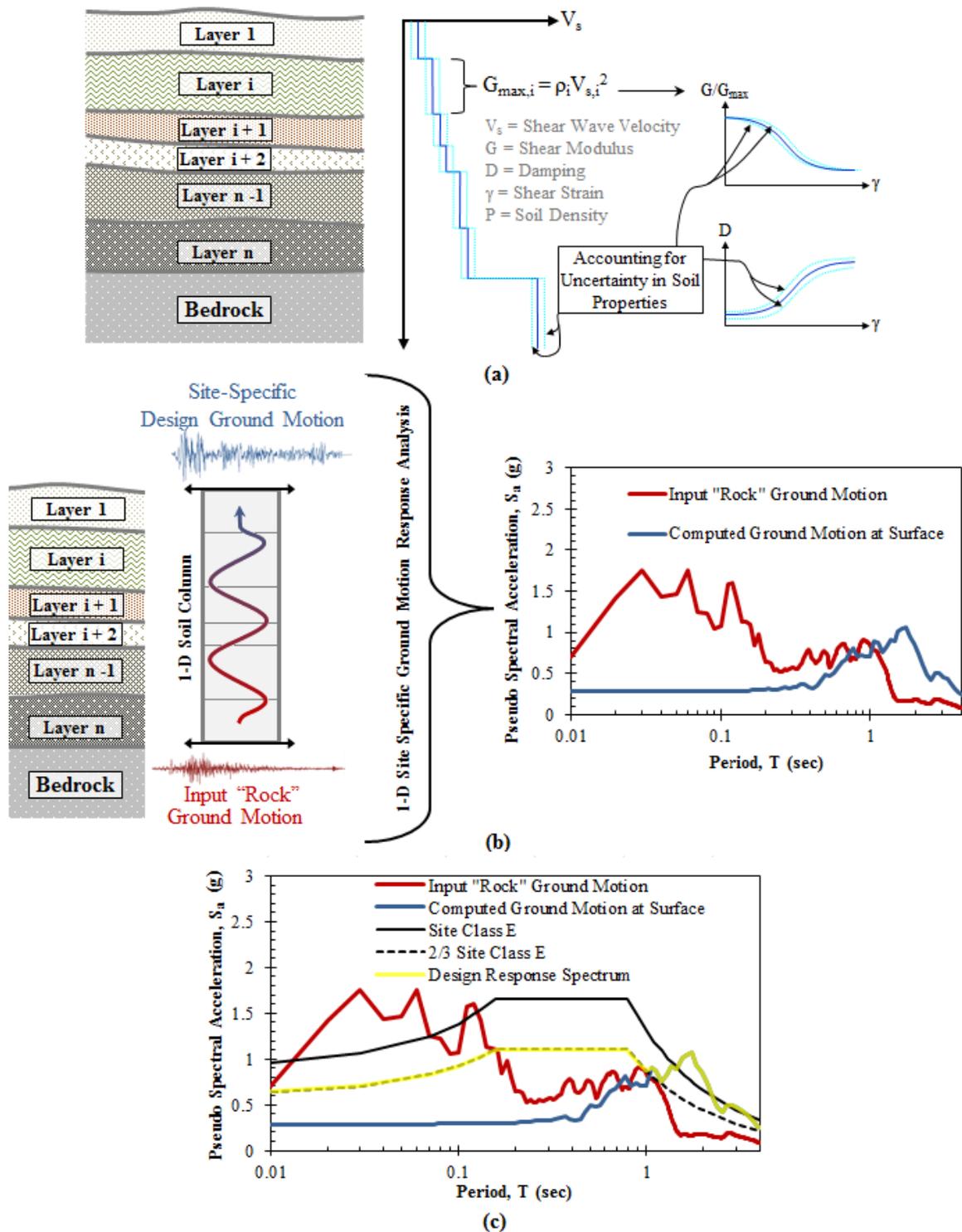
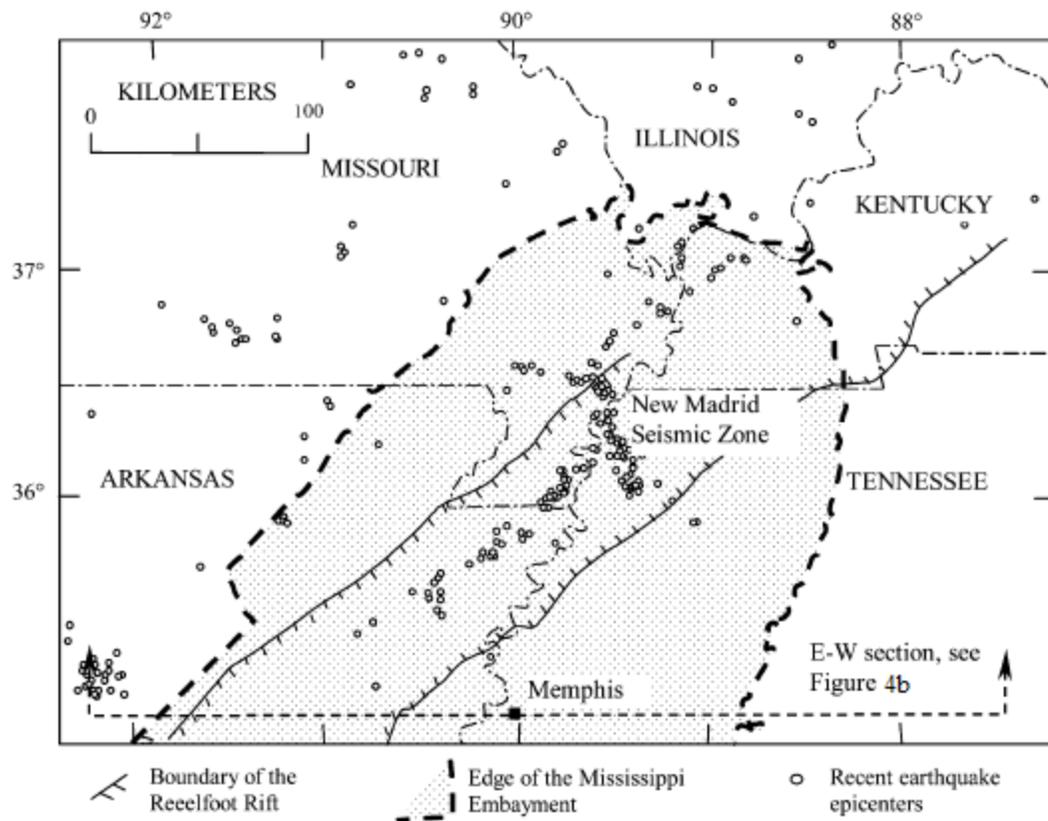
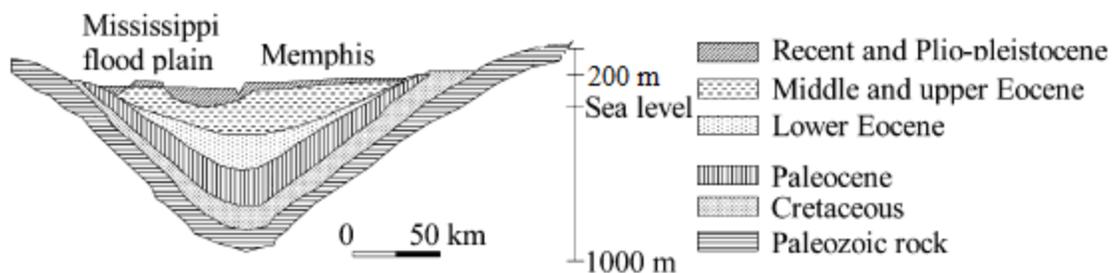


Figure 3. Schematic of site-specific ground motion response analysis: (a) discretize soil profile and determine soil properties; (b) obtain appropriate input “rock” ground motions, and use computer software to simulate upward propagation through the soil column and compute response; (c) develop design response spectrum based on results and restricted to no less than 2/3 of the AASHTO code-based response spectrum.



(a)



(b)

**Figure 4. (a) Plan view of the Mississippi Embayment and major geologic structures and (b) E-W section through Memphis (Hashash and Park, 2001).**

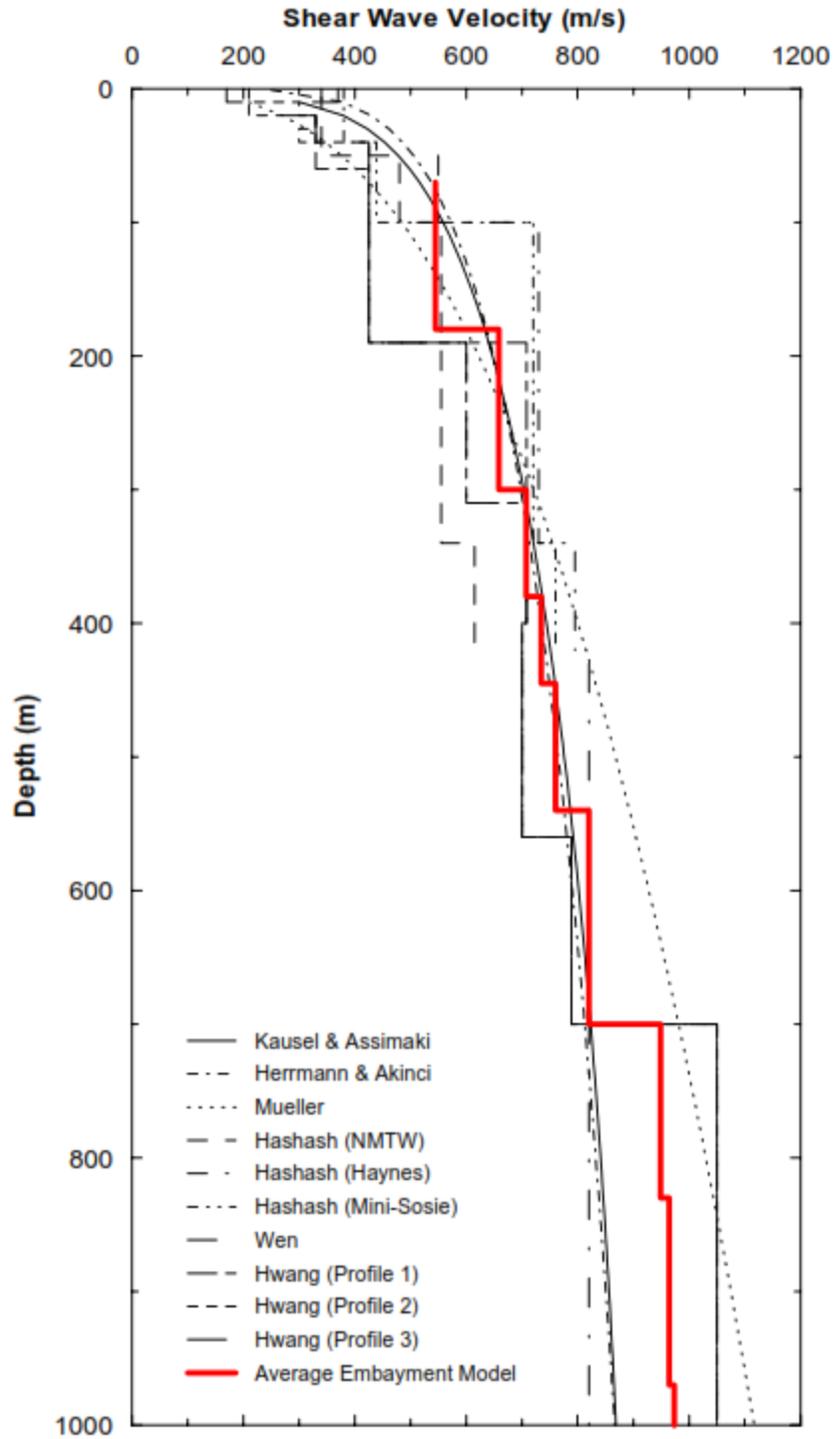


Figure 5. Compiled and average  $V_s$  profiles in the Mississippi Embayment (Romero and Rix, 2001).

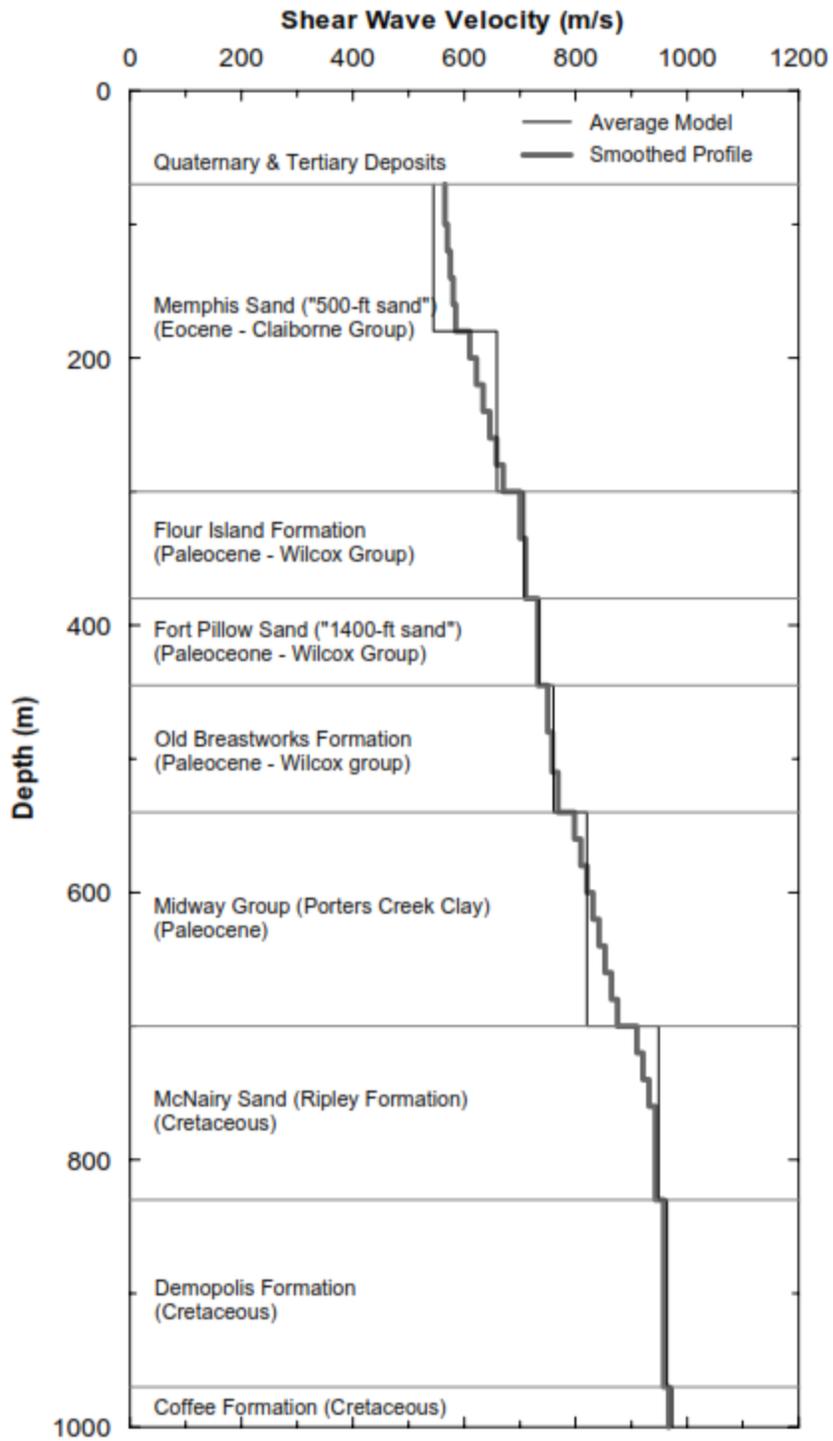


Figure 6. Average and smoothed  $V_s$  profile with assumed geologic layering (Romero and Rix, 2001).

**Table 1. Velocity models developed for the Mississippi Embayment (Romero and Rix, 2001)**

Source	Other Description	Type of Data
Kausel and Assimaki, 2000	MIT profile	Based on Memphis data
Herrmann and Akinici, 2000		Based on travel time
Mueller, 1999		Based on travel time
Hashash, 1999	New Madrid Test Well	Based on borehole test data
Hashash, 1999	Haynes Well	Based on borehole test data
Hashash, 1999	Mini-Sosie	Based on mini-sosie test
Wen, 1999		Based partly on Hashash profiles
Hwang, 2000	Memphis Soil Profile I	Based on SPT data
Hwang, 2000	Memphis Soil Profile II	Based on compiled SPT data
Hwang, 2000	Memphis Soil Profile III	Based on compiled SPT data

Rosenblad et al. (2010) later measured shear wave velocities at eleven sites in the Mississippi Embayment using low frequency surface wave measurements, Figure 7. The  $V_s$  profiles each extend to a depth of approximately 650 feet. Rosenblad and Goetz (2010) combined those profiles with the averaged (not smoothed) full-depth reference  $V_s$  profile from Rix and Romero (2001) as shown in Figure 8. In this manner, full-depth profiles were developed using site-specific measurements for the top 200 meters and a generic reference profile for deeper layers. This approach has also been favored by other researchers (e.g., Zheng et al., 2010).

The shear wave velocity of the basement bedrock of the Mississippi Embayment is known to be relatively high, but there is no consensus on actual magnitude or variability. Cramer (2006) bounded the shear wave velocity of Paleozoic basement rock between 2.0 km/sec (6500 ft/sec) and 3.4 km/sec (11100 ft/sec) for a seismic hazard analysis. The higher bound was chosen to represent borehole observations, and the lower bound to match the assumption from an existing attenuation model commonly used in the region. Park and Hashash (2005) assumed a single bedrock shear wave velocity of 3.0 km/sec (9800 ft/sec) for a site response analysis based on reports from Ou and Herrmann (1990) and Nicholson et al. (1984). Rosenblad (2010) assumed a single bedrock shear wave velocity of 2.5 km/sec (8000 ft/sec) for the development of deep shear wave velocity profiles. Note that all of these estimates indicate that the Paleozoic bedrock classifies as AASHTO Site Class A because  $V_s$  exceeds 5000 ft/sec.

In contrast to using measured shear wave velocity profiles, engineers have also used SPT blowcounts to correlate near-surface  $V_s$ . Rogers et al. (2007) reported this approach for the site response analyses of three highway bridges in Missouri. However, NCHRP Synthesis 428 (NCHRP, 2012) warns against this technique because of significant uncertainty associated with the correlations. The Missouri study also used SPT blowcount to correlate density, which is less problematic. Density (specifically, total unit weight) can usually be assumed or correlated with

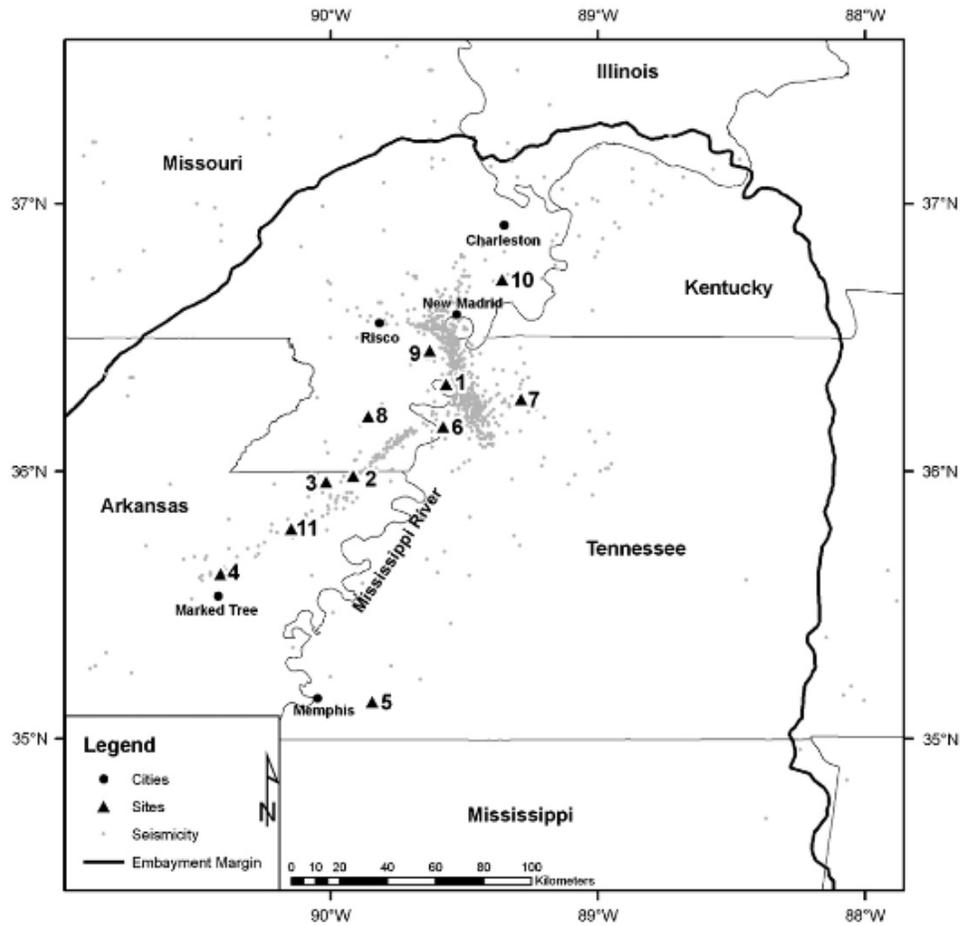


Figure 7. Locations of deep shear wave velocity profiles measured in Mississippi Embayment by Rosenblad et al. (2010).

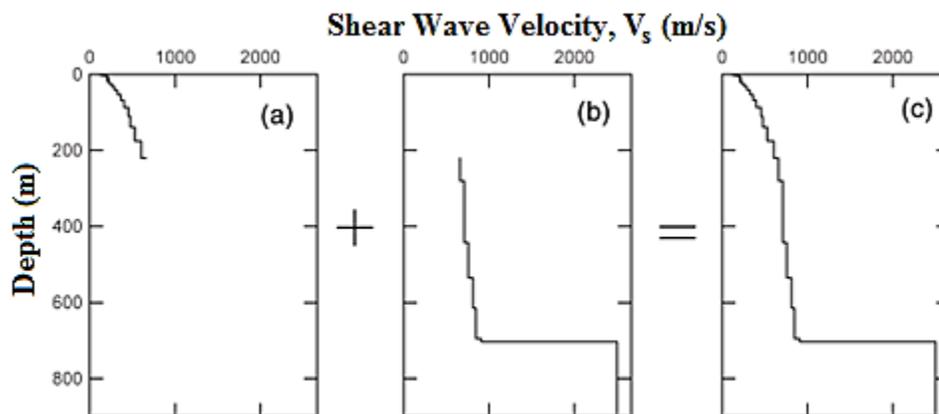


Figure 8. Combination of (a) deep  $V_s$  profiles from Rosenblad et al. (2010) and (b) full-depth reference  $V_s$  profile from Rix and Romero (2005) for (c) site-specific full-depth  $V_s$  profile (Rosenblad and Goetz, 2010).

reasonable confidence. Romero (2001) assumed a mass density of  $1.9 \text{ g/m}^3$  (119 pcf) for near-surface layers and  $2.3 \text{ g/m}^3$  (144 pcf) at the base of the embayment. Catchings (1999) provides a model for densities at greater depths, but those depths are not typically of concern for site response analyses.

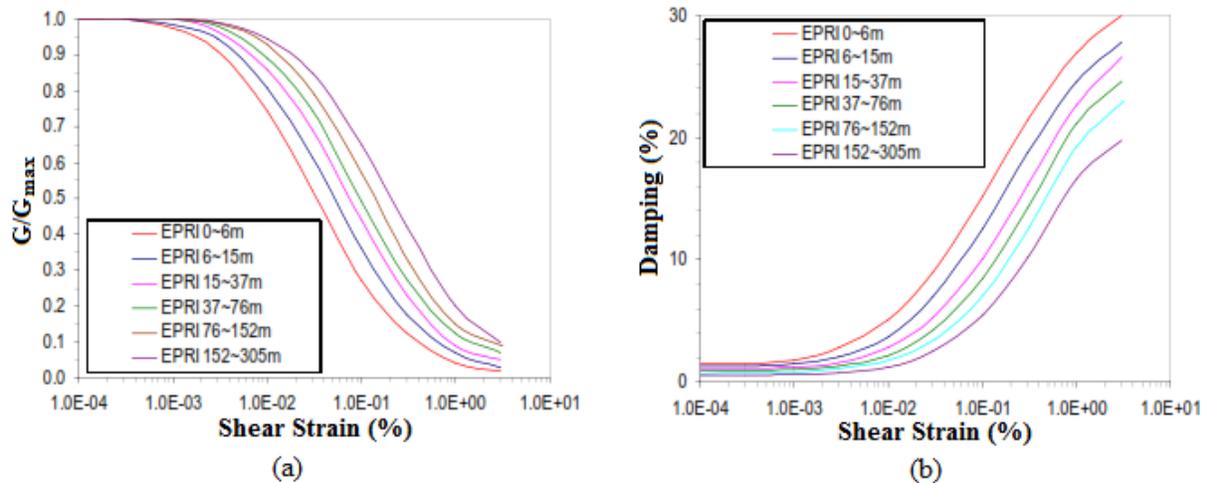
Site response analyses also require nonlinear modulus and damping parameters for each layer in the soil column. With increasing shear strain ( $\gamma$ ), shear modulus ( $G$ ) decreases while damping ( $D$ ) increases. These dynamic soil properties are typically reported in terms of  $G/G_{\max}$ - $\log[\gamma]$  and  $D-\log[\gamma]$ . This behavior may be measured directly in the laboratory in accordance with resonant column-torsional shear tests (ASTM D4015). However, undisturbed samples are very difficult to sample in the deep unconsolidated sediments of the Mississippi Embayment. Furthermore, these deep sediments would require very large confining stresses in the laboratory, which are challenging to simulate, as standard test equipment is not rated for such high stresses.

In the absence of experimental test data specific to the deep soils and high confining pressures of the Mississippi Embayment, engineers typically use empirical correlations based on other soils to develop generalized relationships between shear modulus and damping versus shear strain. These dynamic soil properties are referred to as modulus reduction and damping curves. Seed and Idriss (1970), Hardin and Drnevich (1972), Vucetic and Dobry (1991), Electric Power Research Institute (EPRI, 1993) and Darendeli (2001) developed a progression of dynamic soil property models over time as more and more empirical data became available for incorporation. Importantly, the EPRI (1993) and Darendeli (2001) models incorporate the effects of confining pressure. Additionally, the Darendeli (2001) model can be used to estimate dynamic soil properties for sands and clays with various plasticity indices. In general, the newer models are superior because of the larger databases from which they were developed. EPRI (1993) and Darendeli (2001) are likely the two most common models used today for site response analyses in the Mississippi Embayment. An example of depth-dependent EPRI (1993) curves is presented in Figure 9 (Zheng et al., 2010).

## 2.2. INPUT EARTHQUAKE GROUND MOTIONS

One of the most critical steps in site-specific ground motion response analyses is the selection of input earthquake ground motions for the analyses (NCHRP, 2012). Earthquake ground motions are most commonly represented by acceleration time histories that have been recorded by a seismometer during an actual earthquake event. Ground motion records are measured and archived as sets of three orthogonal motions (two horizontal components, one vertical component). Input ground motions may also be simulated, but this requires a high level of skill. The U.S. Geological Survey (USGS) provides a stochastic tool for simulating ground motions on its website (<http://eqint.cr.usgs.gov/deaggint/2002/index.php>). Rogers et al. (2007) noted that artificial time histories are quick and inexpensive to generate, but they may tend to overestimate earthquake motion.

Selection of appropriate ground motions for site response studies is not a trivial task because all of the following criteria should be satisfied: (1) should have been recorded from an



**Figure 9. EPRI material property curves for (a) shear modulus reduction, and (b) damping (Zheng et al. 2010)**

earthquake with a magnitude similar to what is expected for the design event, (2) should have been recorded at a distance from the earthquake epicenter or fault similar to the distance between the construction site and the expected design earthquake location, (3) should have been recorded in an area with similar geologic features as the construction site, and (4) the acceleration response spectrum should have similar spectral shape to a predetermined target response spectrum for the construction site.

### 2.2.1. TARGET SPECTRUM

Developing a target spectrum is typically the first step performed when selecting input ground motions for site-specific analyses. AASHTO Guide Specifications allow the target spectrum to be obtained from the “mapped” ground motions provided by the USGS (i.e., the standard USGS Seismic Hazard Maps) or from a site-specific hazard analysis. Regardless, AASHTO requires the use of ground motions with a 7% probability of being exceeded over an interval of 75 years for design of transportation infrastructure. These ground motions (GM’s) are often termed the 7% in 75 yr GM’s, which have an approximate return period of 1000 yrs. They are also approximately equal to the more commonly used 5% in 50 year GM’s. While the AASHTO Guide Specifications only provide maps for horizontal peak ground acceleration (PGA), and 5% damped horizontal response spectral acceleration at periods (T) of 0.2 and 1.0 seconds, various digital resources provide ground motions at seven different periods (PGA, 0.1, 0.2, 0.3, 0.5, 1 and 2 seconds). For example, the USGS provides a Java Ground Motion Parameter Tool on its website that calculates ground motions for specific latitude and longitude coordinates (<http://earthquake.usgs.gov/hazards/designmaps/javacalc.php>). Note that AASHTO specifications currently require the 2002 USGS data rather than the 2008 data.

All ground motions from the USGS/AASHTO maps, or the USGS Java Ground Motion Parameter Calculator, are for the reference soil conditions on the boundary between Site Class B/C. Because basement rock in the Mississippi Embayment is Site Class A, the ground motions corresponding to the AASHTO Site Class B/C boundary must be divided by period-dependent

scaling factors to obtain a Site Class A target spectrum. Frankel et al. (1996) provide these scaling factors in the commentary to the 1996 USGS Hazard Maps. Ivan Wong (personal communication) and Zheng et al. (2010) have reported using the same factors, which are provided in Table 2.

The target spectrum is a linear interpolation between the mapped ground motion amplitudes at the different periods. This type of spectrum is generally called a uniform hazard spectrum (UHS) (Kramer, 1996) since the GM hazard is identical (i.e., 7% in 75 yrs) at all periods. The shape of a UHS is different than a code-based design spectrum (refer to Figure 10) or an acceleration response spectrum from an actual earthquake record. In choosing input ground motions for site-specific ground motion response analyses, one would select acceleration time histories whose acceleration response spectra match the “rock” target spectrum (UHS) at the bottom of the soil column as closely as possible near the fundamental period of the structure being designed.

### 2.2.2. GROUND MOTION SELECTION AND SCALING

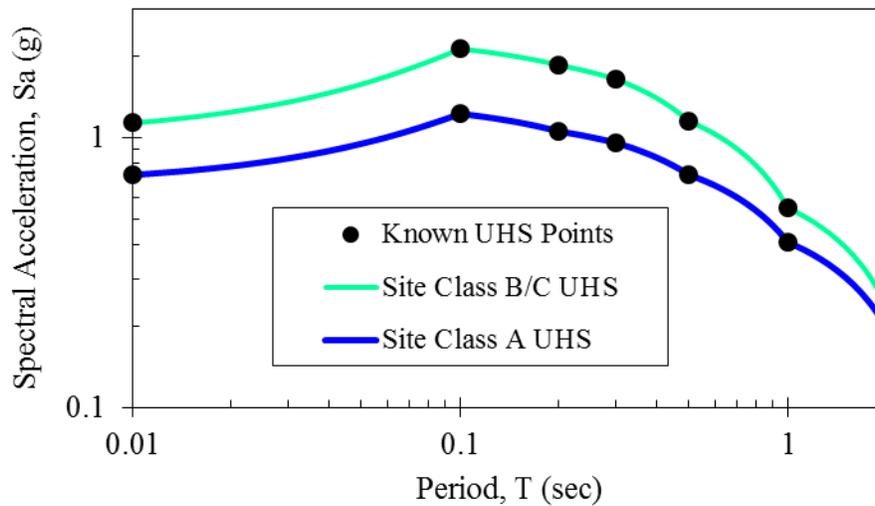
The controlling ground motion scenario(s) (i.e., magnitude and distance combinations contributing most to the expected ground motion hazard) for any location in the U.S. can be obtained from deaggregations of the USGS probabilistic seismic hazard analysis data. A sample deaggregation of the 5% in 50 yr GM’s for the Blytheville, AR bridge is provided in Figure 11. The governing magnitude and distance for this bridge are 7.6 and 12 km (7.5 mi), respectively. Therefore, input ground motions with a similar magnitude and distance should be sought for the site response analyses.

USGS provides user-friendly tools for generating deaggregations on its website (<https://geohazards.usgs.gov/deaggint/2002/>). Users supply latitude and longitude coordinates for the bridge location and then generate the deaggregation at different periods. Each deaggregation includes a figure similar to Figure 11 and a brief summary of the data. For Blytheville, AR, deaggregations are very similar for every period. This is because the seismic hazard in Northeast Arkansas is dominated by the potential for very large earthquakes in a specific region of the NMSZ. Note that deaggregation tools are available for the 1996, 2002 and 2008 data. Each tool provides different results because the corresponding probabilistic seismic hazard analyses are based on different assumptions. Again note that AASHTO specifications require use of the 2002 data rather than the 2008 data.

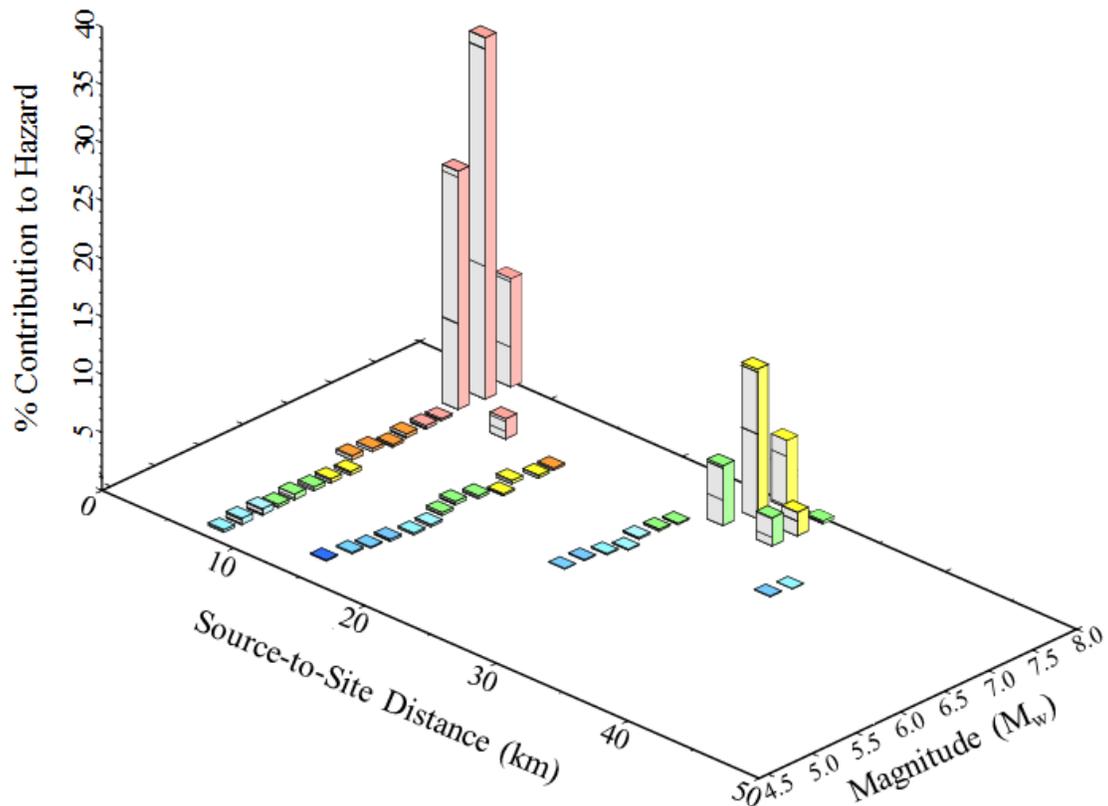
Typically, designers query free databases such as PEER ([www.peer.berkeley.edu](http://www.peer.berkeley.edu)) or COSMOS Virtual Data Center (<http://db.cosmos-eq.org>) to find ground motions that reasonably match the target deaggregation scenario (magnitude and distance) and tectonic setting of a site. However, ground motion time history selection is particularly difficult in the NMSZ because there are no recorded ground motions from large-magnitude earthquakes in or near the Mississippi Embayment. Due to the scarcity of appropriate recorded ground motions in the central and eastern U.S. (CEUS), McGuire et al. (2001) developed a suite of input ground motions for The Nuclear Regulatory Commission (NUREG/CR-6728). They selected a number of acceleration time histories from various earthquakes around the world in active tectonic

**Table 2. Scaling factors for converting AASHTO Site Class “B/C” ground motion amplitudes to AASHTO Site Class “A” ground motion amplitudes**

Period (seconds)	Scaling Factor
0 (PGA)	1.52
0.1	1.74
0.2	1.76
0.3	1.72
0.5	1.58
1.0	1.34
2.0	1.20



**Figure 10. UHS for generic reference conditions (Site Class B/C) and the Site Class A UHS (target spectrum for input ground motions, computed using scaling factors in Table 2) for stiff Paleozoic bedrock in Blytheville, AR.**



**Figure 11. Deaggregation of zero-second period (PGA) ground motions with a 5% probability of exceedance in 50 yrs for Blytheville, AR (2002 USGS Probabilistic Seismic Hazard Analysis).**

settings and organized them into magnitude and distance bins. These acceleration time histories were then synthetically altered to better resemble the frequency content expected in the stable continental setting of the CEUS. Essentially, the idea is that stiffer bedrock in areas such as the Mississippi Embayment would transmit more high-frequency energy. Thus, the selected ground motions were amplified in the high-frequency range. The ground motions are intended to be used in the reliability-based design of nuclear facilities in the CEUS where there are no recorded ground motions available for site-specific ground motion response analyses. The McGuire et al. (2001) NUREG database of time histories was used in this study.

Whatever method is used to collect input ground motions, they will inevitably need to be adjusted to match the target spectrum. AASHTO Guide Specifications suggest scaling the input ground motions so that the acceleration response spectra match the target spectrum at the fundamental period of the structure being designed. In practice, two methods are commonly used to adjust the selected ground motions to the target spectrum: scaling and spectral matching.

Scaling ground motions is the simpler approach. The entire acceleration time history is simply multiplied by a single factor (scaler) so that its pseudo acceleration response spectrum more closely matches the target spectrum at the fundamental period of the structure. Scaling may be performed in basically any software that allows computation of acceleration response

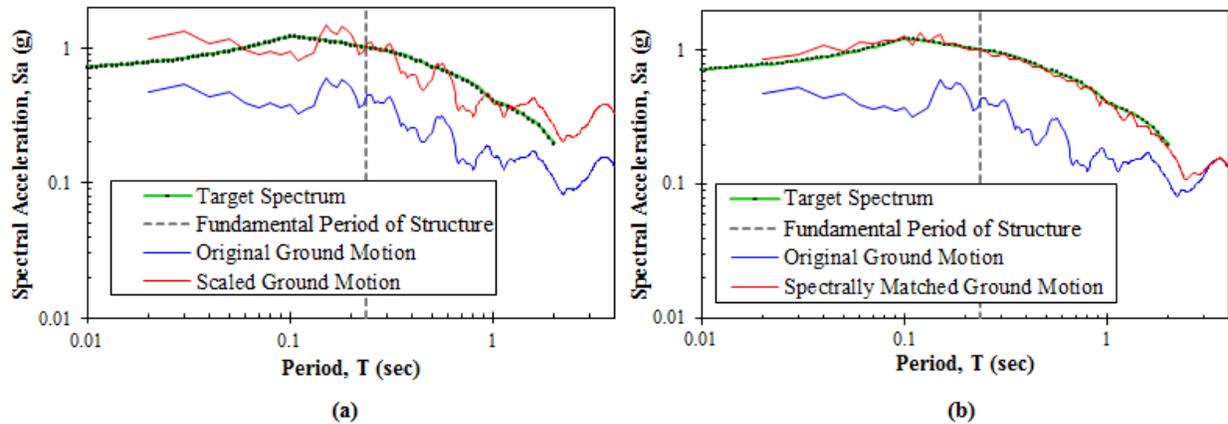
spectra from ground motion time histories. One such free program is called STRATA (Kotke and Rathje, 2010), freely available at <http://nees.org/resources/692>.

Spectral matching involves matching each selected acceleration response spectrum to the entire target spectrum. The approach for spectral matching has evolved over the past two decades. Currently, the preferred approach is to add wavelets to the acceleration time history until the corresponding pseudo acceleration response spectrum is reasonably close to the target spectrum (Al Atik and Abrahamson, 2010). RspMatch 2009 is a user-friendly computer program that can be used to perform spectral matching in this manner. It is available as a stand-alone program and marketed by GeoMotions, LLC ([www.geomotions.com](http://www.geomotions.com)). Heo et al. (2011) found that although many factors affect the quality of input ground motions, spectral matching tends to be a more reliable adjustment method than scaling. NCHRP Synthesis 428 (NCHRP, 2012) also reports that spectral matching is gaining popularity among designers. A visual comparison of the two methods is presented in Figure 12.

### 2.3. SITE RESPONSE ANALYSES

The actual site response analyses may be conducted in a number of ways. The essential task is modeling the propagation of the input “rock” horizontal ground motions from the bottom of the soil profile up to the ground surface. The analyses may be conducted in 1, 2 or 3 dimensions. Soil properties may be assumed linear, “equivalent-linear” (EQL) or fully nonlinear (NL), and pore pressure generation (effective stress) may or may not be considered in NL analyses. Most analyses by highway agencies are conducted in one dimension (1D), in terms of total stress, with nonlinear soil properties that are used in an equivalent-linear analysis (NCHRP, 2012). SHAKE (Schnabel et al., 1972; Ordonez, 2000) is the most commonly used computer program for 1D, total stress, EQL analyses. It is commercially available through GeoMotions, LLC. Other programs have also been developed independently to run the same type of analysis including DEEPSOIL (Hashash and Park, 2001, 2002; Hashash et al., 2011) and several others (NCHRP, 2012). DEEPSOIL is freely distributed at <http://deepsoil.cee.illinois.edu/>.

EQL analyses are popular because the input parameters are relatively easy to obtain and because it has been well studied and verified over the past four decades. However, EQL analyses become less reliable when the site response involves “large” shear strains, which are common for soft near-surface soils subjected to large input ground motions. The problem arises because, while dynamic soil properties are known to be nonlinear, EQL analyses require those properties to remain constant in each layer of the soil profile, only changing between iterations to converge on a target value of effective strain (usually 65% of the peak shear strain). NL analyses are more appropriate for large-strain situations because cyclic hysteretic soil behavior during unloading and reloading is represented with a constitutive model, allowing dynamic soil properties to change appropriately with time/induced strain. Nonlinear analyses are preferred for ground shaking greater than 0.4g (Ishihara, 1986) or when peak shear strains exceed 2% (FHWA, 1997). NCHRP Synthesis 428 (NCHRP, 2012) warns that the reliability of EQL analyses may diminish at shear strains as small as 0.5% (in any layer of the soil profile). However, Hashash et al. (2010) report that EQL analyses are robust in revealing the key



**Figure 12. Example of (a) scaling and (b) spectrum-matching of input ground motions to a target spectrum.**

characteristics of any site. In this respect, the EQL approach may be used at any site to flush out possible errors in more complicated analyses (NCHRP, 2012).

NL analyses may be conducted in terms of total or effective stress. Common computer programs for NL analyses include D-MOD (Matasovic and Ordonez, 2007), DEEPSOIL, FLAC (Itasca, 2005) and others (NCHRP, 2012). Effective stress analyses require a pore pressure generation model. These models may be semi-empirical or advanced models requiring specialized soil input parameters. NCHRP Synthesis 428 (NCHRP, 2012) reports that most designers in the U.S. simply use the pore pressure generation model that is incorporated in the software being used for the NL analysis. D-MOD and DEEPSOIL are the two most popular platforms for NL analyses, and both utilize the Matasovic (1993) pore pressure generation model. FLAC users employ the UBC sand model (Byrne et al., 1995; Beaty and Byrne, 1998), and may also choose to conduct 2D or 3D analyses.

Zheng et al. (2010) recently conducted site-specific hazard and ground motion response analyses for a coal-fired power plant in Osceola, Arkansas. They employed several different software programs and utilized both EQL and NL analyses. The governing building code for the project was the 2000 International Building Code (IBC). The research team developed a  $V_s$  profile by conducting geophysical tests for the top 36.5 meters (120 ft). That portion of the subsurface was parsed into a lower bound, average and upper bound velocity profile. This was done to study the effect of variable near-surface soil properties on the site response. These near-surface profiles were combined with the Romero and Rix (2001) full-depth reference profile, which was not varied or randomized in any way. Depth to bedrock was based on a nearby well log and assumed to be 880 m (2900 ft). EPRI (1993) curves were used to estimate modulus reduction and damping behavior for the cohesionless soils. Two shallow clay layers were characterized using the plasticity index (PI) dependent correlation from Vucetic and Dobry (1991).

Ground motions for the study were selected from the largest intra-plate earthquakes on record from around the world. That alone was not sufficient, however, because seismic design in the NMSZ is governed by a series of intra-plate earthquakes larger than any that have ever been

recorded. The intra-plate ground motions were thus complimented by records from inter-plate earthquakes with more appropriate magnitudes. In addition, the team considered a single simulated ground motion. The target spectrum was developed by constructing a UHS from the 1996 USGS Seismic Hazard Maps (USGS, 1996) as specified by the 2000 IBC. The selected input ground motions were first scaled and then spectrum-matched to the target spectrum using RspMatch 2005 (Abrahamson, 1992 and 1998). A total of 9 sets of GM's were used, each set comprising two horizontal and orthogonal components from the same recording station.

Three one-dimensional analysis methods were used for the site response: DEEPSOIL, SUMDES (Li et al., 1992) and SHAKE91+. EQL analyses in DEEPSOIL and SHAKE91+ produced essentially identical results. Two EQL analyses in DEEPSOIL, using discrete versus hysteretic modeling for modulus reduction and damping, also showed acceptable agreement. There was a slight difference, however, between NL models in DEEPSOIL and SUMDES. For strong input "rock" motions in the high-frequency range, DEEPSOIL calculated a larger surface response. This is attributed to the simplified Rayleigh damping scheme used in SUMDES, which is biased at high frequencies (Kwok et al., 2007).

The results were weighted as follows to construct an average site-specific design spectrum: 50% for nonlinear DEEPSOIL analyses, 17% for SUMDES nonlinear analyses and 33% for the equivalent linear analyses. The weighted mean was multiplied by 2/3 to calculate the design response spectrum in accordance with ASCE-7-05 (i.e., 2/3 of the 2% in 50 yr ground motions). Also in accordance with ASCE-7-05, the 2000 IBC requires that the final design spectrum from a site-specific site response be no less than 80% of the code-based design spectrum. The research team developed the site-specific response spectrum as shown in Figure 13, which indicated spectral accelerations less than the standard IBC 2000 Site Class E spectrum at all periods less than 1.5 seconds. This study serves as a relative benchmark for the site-specific response analyses performed and documented herein, as the Osceola site is located less than 20 miles from the Blytheville, AR bridge site.

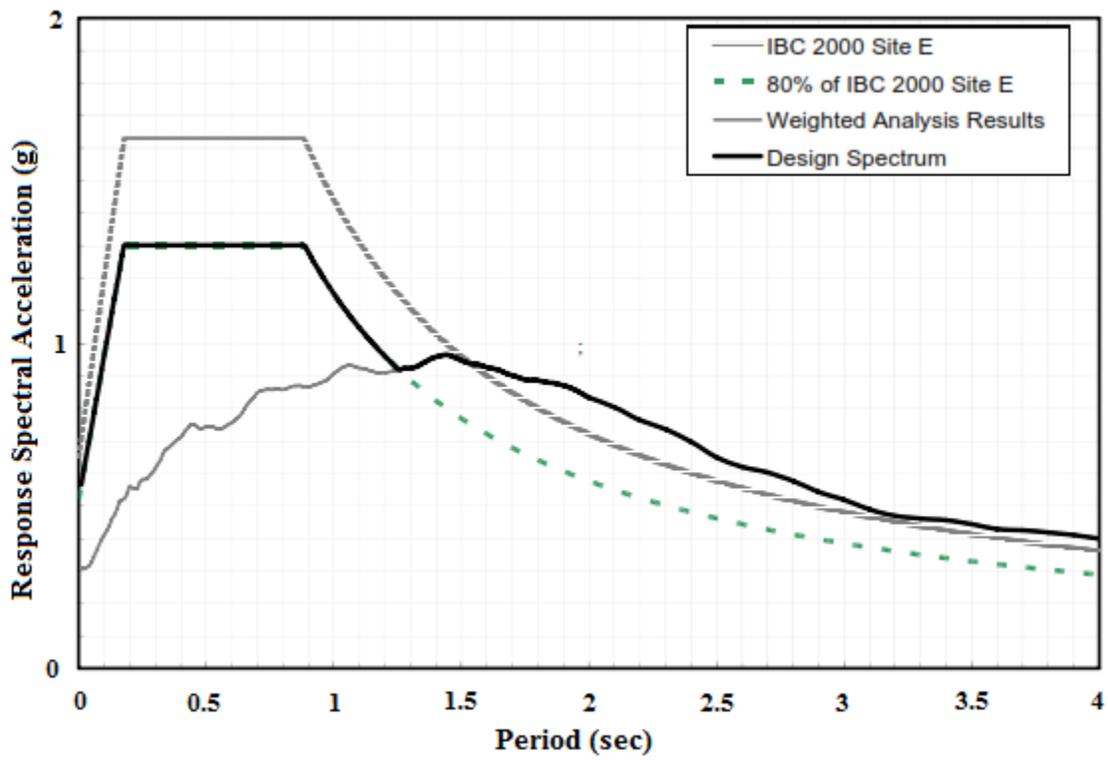


Figure 13. Design response spectrum from site-specific ground motion response analysis in Osceola, Arkansas (Zheng et al., 2010).

### 3. INFORMATION NEEDED FOR SITE-SPECIFIC GROUND MOTION RESPONSE ANALYSES

This study aims to demonstrate the feasibility of reducing seismic design ground motions in the short-period range by conducting site-specific ground motion response analyses in Northeast Arkansas. In that effort, all steps of the site response process have been conducted for a recently constructed bridge. The example site is AHTD Bridge No. 07204 from Job No. 100705, a railroad overpass bridge on Highway 18 in Blytheville, Arkansas. Site coordinates, mapped in Figure 14, are 35.92611 N, 89.90556 W.

#### 3.1. SHEAR WAVE VELOCITY PROFILES

Site response analyses in the Mississippi Embayment require shear wave velocity ( $V_s$ ) profiles for very deep soil deposits. Measured near-surface  $V_s$  profiles from a site are typically extended to bedrock using a generic full-depth reference profile for the region (Zheng et al., 2010; Rosenblad and Goetz, 2010; Park and Hashash, 2005; Romero and Rix, 2001). A similar approach is followed herein. However, no measured near-surface  $V_s$  profiles were available for the Blytheville bridge site, so the site  $V_s$  profile was constructed from four, 650-foot  $V_s$  profiles measured previously in Northeast Arkansas by Rosenblad et al. (2010). Locations of the four near-by  $V_s$  profiles are shown in Figure 15 relative to the Blytheville bridge. The measured  $V_s$  profiles are shown in Figure 16. The  $V_s$  profiles are remarkably similar over the top several hundred feet, indicating only moderate variability between near-surface soil profiles across the region. However, in practice it is always advisable to actually measure the site-specific  $V_s$  profile for each site response analysis.

Rosenblad et al. (2010) combined the measured 650-ft  $V_s$  profiles with the averaged full-depth reference  $V_s$  profiles by Romero and Rix (2001) to reach bedrock at each site. These extrapolated  $V_s$  profiles are provided in Figure 17. It should be noted that other researchers have used the averaged and smoothed reference profile (Park and Hashash, 2005) to represent the deep  $V_s$  structure. The depth to bedrock for each site was obtained based on work from Van Arsdale and TenBrink (2000). At the four Arkansas  $V_s$  sites, depth to bedrock ranges from 783 m (2570 ft) to 847 m (2780 ft). Rosenblad et al. (2010) assumed the bedrock  $V_s$  was 2500 m/s (8200 ft/s).

In order to account for measurement uncertainty and spatial variability, the four full-depth  $V_s$  profiles developed by Rosenblad et al. (2010) were synthesized into three design profiles for the site response analyses. The first site response  $V_s$  profile is the lognormal median of the four individual profiles. The second two profiles were developed by assuming a coefficient of variation (COV) of 20% to define upper and lower bound  $V_s$  profiles. The coefficient of variation is the ratio of the standard deviation to the mean. Figure 17 shows the four measured  $V_s$  profiles along with the lognormal median and +/- 20% COV profiles used in the site response analyses. Note that the upper and lower bounds encompass nearly all of the  $V_s$  profile variations and the range of bedrock  $V_s$  values (6500 – 11100 ft/sec) reported by Cramer (2006). Also note that the depth to bedrock is different for the median, plus 20% COV and minus 20% COV  $V_s$  profiles. The median profile has the average depth to bedrock from all four



**Figure 14. Location of example bridge site for site-specific ground motion response analysis: AHTD bridge No. 07204 from job No. 100705.**



**Figure 15. Locations of four  $V_s$  profiles (Site 2, Site 3, Site 4 and Site 11) measured by Rosenblad et al. (2010) relative to the example bridge site.**

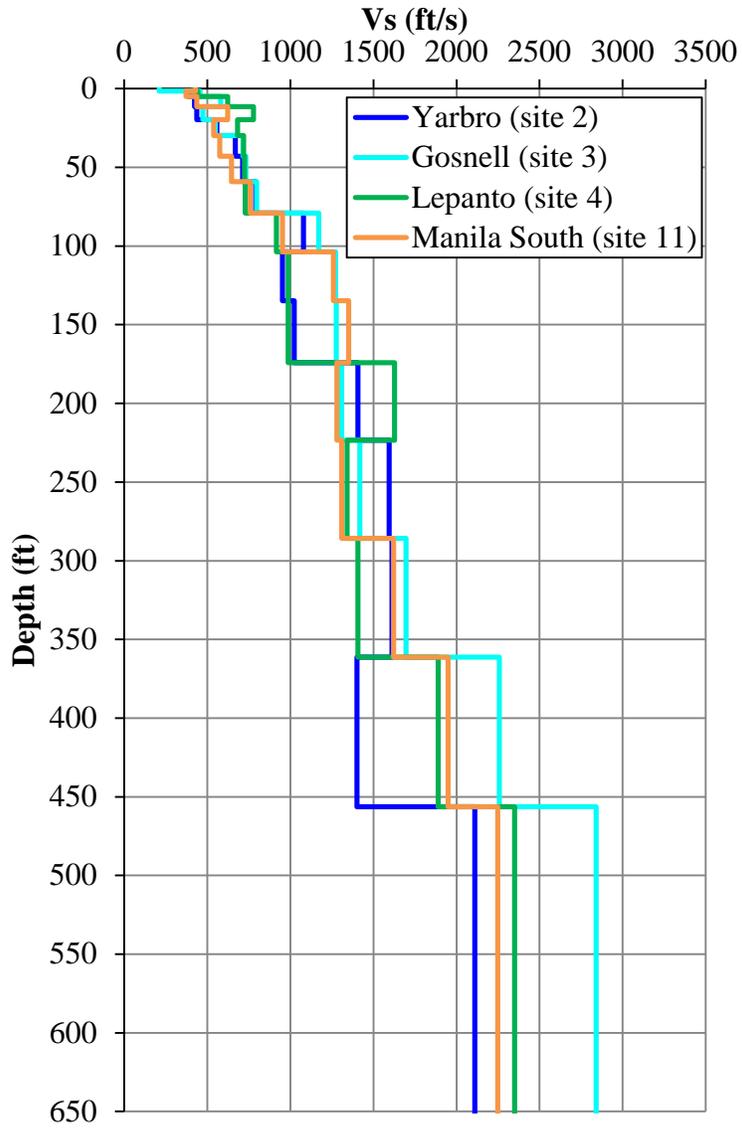
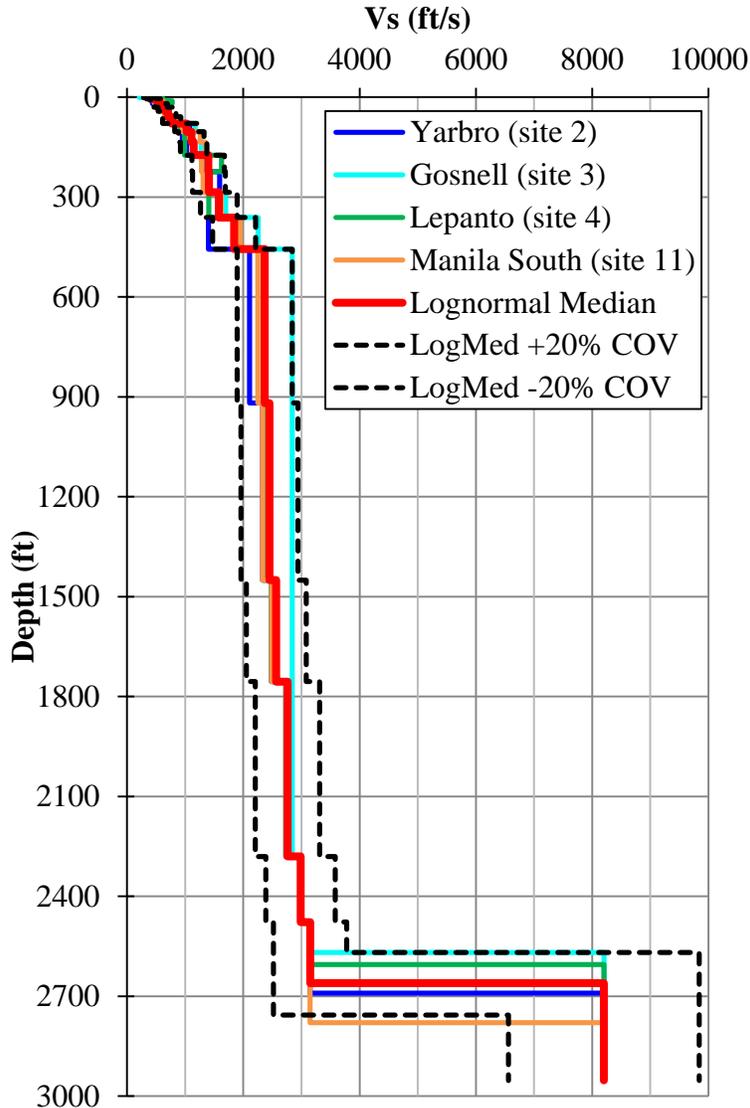


Figure 16. 650-foot  $V_s$  profiles in Northeast Arkansas measured by Rosenblad et al. (2010).



**Figure 17. Full-depth  $V_s$  profiles developed for the example bridge site in Blytheville, Arkansas: lognormal median and plus/minus 20% COV from four  $V_s$  profiles developed by Rosenblad et al. (2010).**

profiles. The +/- 20% COV profiles have depths to bedrock corresponding to one normal standard deviation above/below the average depth, respectively.

The Gosnell, AR and Yarbro, AR  $V_s$  profiles are particularly close to the example bridge site in Blytheville, AR (6.5 and 3.9 miles, respectively, refer to Figure 15). The Gosnell, AR  $V_s$  profile was chosen to “most closely” represent conditions at the example site and was utilized as a fourth  $V_s$  profile for site response analyses. The Gosnell  $V_s$  profile was chosen because, although Yarbro is closer to the example site, the Gosnell shear wave velocities are considerably larger at most depths. Earthquakes in this region are expected to contain a substantial amount of high-frequency energy and stiffer soils will transmit more of that energy to the ground surface than soft soils. By choosing the relatively stiffer Gosnell  $V_s$  profile, the more severe scenario for

short-period bridges is considered, in which more high-frequency energy reaches the ground surface. The Gosnell  $V_s$  profile was also used as a starting point for a sensitivity analysis to investigate the importance of determining the precise depth to bedrock for site response analyses in the region. In this sensitivity analysis the depth to bedrock was varied by 330 and 650 feet above and below the reported depth of 783 m (2570 ft).

### 3.2. UNIT WEIGHT (DENSITY) PROFILES

Soil density is a required parameter that is typically assumed for site response analyses. For this study, we have assumed total unit weights ranging from 120 to 140 pcf for sediments, and equal to 145 pcf for bedrock. Unit weights were assigned based on an assumed relationship with shear wave velocity, such that higher  $V_s$  values correspond to higher total unit weights. Table 3 describes the relationship that was used to assign values for total unit weight. Because: (1) unit weight can be assumed with relative accuracy and (2) errors in the assumed values are not likely to significantly impact results, we chose not to account for uncertainty in values for total unit weight.

### 3.3. DYNAMIC SOIL PROPERTIES: MODULUS REDUCTION AND DAMPING CURVES

Nonlinear dynamic soil properties may be measured directly in the laboratory, but very deep sediments with extreme in-situ confining stresses would require highly advanced sampling and testing procedures. For sites in the Mississippi Embayment where it is impractical to measure dynamic soil properties directly, it is particularly important to use a correlation that accounts for in-situ stress. Only recently (EPRI, 1993; Darendeli, 2001) have dynamic soil models accounted for the effects of confining pressure on modulus and damping. In addition, the Darendeli (2001) model accounts for the effects of plasticity on modulus reduction and damping. This is important at the Blytheville, AR bridge site because the top 30 feet of the soil profile comprise moderately plastic clay. Therefore, initial modulus reduction ( $G/G_{\max}-\log[\gamma]$ ) and damping ( $D-\log[\gamma]$ ) curves for this study were developed using correlations from Darendeli (2001).

The Darendeli (2001) correlations are based on a large set of empirical data from laboratory tests. The formulation includes three sets of curves to represent the scatter and covariance structure of the data. Mean  $G/G_{\max}-\log[\gamma]$  (denoted as  $\mu_G$  herein) and  $D-\log[\gamma]$  (denoted as  $\mu_D$  herein) curves provide the best estimates of modulus and damping from first-order-second-moment statistical analyses. The degree of variability about the mean curves varies with shear strain. Variability is smallest at small shear strains, but increases with increasing shear strains. Darendeli (2001) provides parameters for calculating the strain-dependent standard deviation ( $\sigma$ ) in order to develop statistical bounds on dynamic soil properties. Thus, three sets of modulus reduction and damping curves were developed for this study: (1) mean curves ( $\mu_G/\mu_D$ ), (2) upper bounds ( $+\sigma_G/-\sigma_D$ ) and (3) lower bounds ( $-\sigma_G/+\sigma_D$ ). Note that the plus one standard deviation modulus relationship ( $+\sigma_G$ ) is used with the minus one standard deviation damping relationship ( $-\sigma_D$ ), and vice-versa, because for a given soil and confining pressure a relatively higher modulus corresponds to relatively lower damping.

**Table 3. Assumed relationship used for assigning total unit weight values based on shear wave velocity**

$V_s$ Range (ft/sec)	Total Unit Weight (pcf)
$0 < V_s \leq 600$	120
$600 < V_s \leq 1200$	130
$1200 < V_s \leq 2500$	140
$V_s > 2500$	145

The three aforementioned sets of Darendeli (2001) curves were calculated for each soil layer and manually entered into DEEPSOIL for the site response analyses. The advanced user options in DEEPSOIL are different for equivalent linear (EQL) versus completely non-linear (NL) analyses. NL analyses include an option to fit the Darendeli (2001) curves with a modified hyperbolic model, which is intended to provide a better estimate of damping at large shear strains. The procedure utilizes a single modulus reduction and damping factor (MRDF) to simultaneously modify the modulus reduction and damping curves to match a slightly different hyperbolic model as closely as possible (Hashash et al., 2010). The MRDF-UIUC procedure (an option in DEEPSOIL) is preferred because, unlike the MRDF-Darendeli (2001) option, it prevents damping curves from decreasing at shear strains beyond approximately 1%. The MRDF procedure may be conducted with little effort using DEEPSOIL; however, the program utilizes an optimization procedure that is difficult to implement in a spreadsheet or elsewhere.

The MRDF-UIUC correction also addresses several issues associated with the Darendeli (2001) curves. For instance, at small shear strains, the upper bound Darendeli (2001) curves can include  $G/G_{\max}$  values greater than 1.0 and negative damping values, while the lower bound curves sometimes contain negative  $G/G_{\max}$  values at larger shear strains. Finally, as discussed below, all of the Darendeli (2001) curves predict a peak in damping around 1% shear strain followed by a marked decrease in damping at larger strains. This may indeed be appropriate behavior due to the highly irregular shape of collapsing hysteresis loops at large shear strains (Kenneth H. Stokoe, personal communication), however, a decrease in damping is not commonly used in site response analyses. The MRDF-UIUC procedure, when applied carefully, provides a remedy to each of these issues and was applied to all Darendeli (2001) curves used for NL analyses. The MRDF fitting option is not available for EQL analyses, so the Darendeli (2001) curves were manually adjusted for EQL analyses as follows:

1. Upper bound  $G/G_{\max}$  curves were bounded to  $\leq 1.0$  at small strains
2. Lower bound  $G/G_{\max}$  curves were bounded to  $\geq 0.01$  at larger strains
3. Lower bound Damping curves were bounded to  $\geq 0.2\%$  at small strains
4. All damping curves were held constant beyond the peak value at large strains

The degradation of large-strain damping associated with Darendeli (2001) curves is only a relic of the model's mathematical formulation. Dynamic soil behavior is not well understood or constrained by data at strains above 1%, and at this time there is no widely accepted method or model for predicting such behavior. The Darendeli (2001) correlations, like others, were developed from laboratory data measured at small shear strains ( $\gamma < 1\%$ ) and intended for use in

the shear strain range of  $0.0001\% < \gamma < 1\%$ . This range is sufficient for most applications in soil dynamics, but site response analyses in the heart of the NMSZ routinely predict “large” shear strains, sometimes on the order of 10%. This is a result of strong horizontal ground motions propagating upward through particularly soft soil profiles.

When large shear strains are expected, modifications to the modulus reduction curves may be needed in order to simulate realistic soil shear strengths. After all, shear failures typically occur at shear strains less than 10% in the laboratory and in the field. Modulus reduction curves are directly related to shear stress ( $\tau$ ) as defined in Equation 1. If a soil layer’s shear wave velocity ( $V_s$ ) and bulk density ( $\rho$ ) are known, Equation 1 may be used to calculate the shear strength ( $\tau_{max}$ ) implied by a given  $G/G_{max}$  relationship at large shear strains as follows:

$$\tau = \frac{G}{G_{max}}(G_{max}) \cdot \gamma = \frac{G}{G_{max}} \cdot (\rho \cdot V_s^2) \cdot \gamma \quad \text{Eq. 1}$$

Where:

- $G/G_{max}$  = Ratio of shear modulus to maximum shear modulus obtained from laboratory tests or relationships
- $\gamma$  = Shear strain (decimal, not percent)
- $\rho$  = Bulk density (i.e., total unit weight  $\div$  acceleration due to gravity)
- $V_s$  = In-situ shear wave velocity

The shear strengths implied by the shape of the  $G/G_{max}$  curve may not be realistic (either too high or too low) and should be checked in all soil layers where predicted shear strains exceed 1%. If shear strength corrections are required, target strength values ( $\tau_{max}$ ) must first be assigned to each soil layer of interest (for example, using Mohr-Coulomb theory and an assumed/measured friction angle for sands). Shear strength corrections involve adjustments to the entire stress-strain ( $\tau$ - $\gamma$ ) curve implied by the selected  $G/G_{max}$  curve, so an appropriate strain level must be chosen at which the implied  $\tau$ - $\gamma$  curve should approach the target strength. Hashash et al. (2010) describe an iterative procedure for adjusting the modulus reduction and damping curves to achieve reasonable implied shear strengths. This method provides a practical solution for defining the large-strain dynamic soil properties in the absence of empirical data:

1. Apply the MRDF-UIUC curve fitting procedure to the uncorrected Darendeli (2001) curves.
2. Paste the adjusted modulus reduction curve from [1] into a spreadsheet. Use the modulus reduction curve ( $G/G_{max}$ - $\log[\gamma]$ ) to calculate and plot the implied shear stress-strain curve ( $\tau$ - $\gamma$ ).
3. If the plots from [2] indicate that the implied  $\tau_{max}$  is less than the target shear strength (i.e., the implied shear strength is underestimated) in the strain range of interest (typically 3-4%), adjust the  $G/G_{max}$  data points at all shear strains greater than 0.1% until the implied  $\tau_{max}$  is slightly above the target strength. If the plots from [2] indicate that the implied  $\tau_{max}$  is greater than the target strength (i.e., the implied shear strength is overestimated) in the strain range of interest, adjust the  $G/G_{max}$  data points at all shear strains greater than 0.1% until  $\tau_{max}$  is slightly below the target strength.

4. Paste the adjusted modulus reduction curve from [3] back into DEEPSOIL along with the MRDF-adjusted damping curve from [1] and repeat the MRDF-UIUC curve fitting procedure.
5. Paste the fitted modulus reduction curve from [4] into a spreadsheet and plot the implied stress-strain curve. If the implied  $\tau_{\max}$  is reasonably close to the target shear strength in the strain range of interest, the procedure is finished. If not, repeat steps [2] – [5] until the implied shear strength is satisfactorily adjusted.

Generally, several manual iterations are required in order to achieve reasonable implied shear strengths. The number of necessary iterations varies according to soil type and user experience, but each iteration generally requires a significant amount of time and engineering judgment. In this report, the three possible adjustments to the Darendeli (2001) curves are denoted as follows:

1. *MA-EQL*: Manual adjustments for EQL analyses
2. *MRDF*: MRDF-UIUC adjustment within DEEPSOIL for NL analyses
3. *MRDF-ISSC*: Implied shear strength corrections for EQL and NL analyses

Figure 18 illustrates the iterative procedure used to generate the MRDF-ISSC curve for a cohesive soil layer. The same procedure is shown in Figure 19 for a cohesionless soil layer and illustrates the iterative procedure's minimal effects on both modulus reduction and damping.

Preliminary site response analyses were conducted after making MA-EQL and MRDF adjustments to the modulus reduction and damping curves for each layer of the soil profiles to identify depths where large strains occur, potentially requiring implied shear strength corrections. The preliminary study included 24 separate sets of site response analyses to account for uncertainties and potential variability, as described in Chapter 5. Figure 20 summarizes the lognormal median shear strain profiles from all of the preliminary site response analyses. Large shear strains were mostly limited to the top 100 feet, where  $\gamma$  consistently exceeded 1%. The corresponding shear stresses, as computed by DEEPSOIL, are shown in Figure 21 along with Mohr-Coulomb shear strengths corresponding to friction angles of 27°, 30° and 33°. Indeed, the computed shear stresses from several analyses clearly exceed the expected shear strength in the top 100 feet where large strains were observed.

Based on the preliminary results, implied shear strength corrections were used to adjust the dynamic properties of all soil layers in the top 100 feet of each trial soil profile. In order to expedite the MRDF-ISSC corrections, soil layers in the top 100 feet of the DEEPSOIL profiles were discretized into five aggregate layers. Darendeli (2001) curves were assigned based on properties at the middle of the aggregate layer. Target strengths for each aggregate layer were chosen based on laboratory and in-situ measurements by Coffman (2012). Separate target strengths were chosen for modifications to the mean ( $\mu_{NG}/\mu_D$ ), upper bound ( $+\sigma_G/-\sigma_D$ ) and lower bound ( $-\sigma_G/+\sigma_D$ ) curves for each aggregate layer. The strain range within which the target strength should be achieved was set at 3 – 5% for all curves. Properties of the five aggregate layers for MRDF-ISSC adjustments are summarized in Table 4.

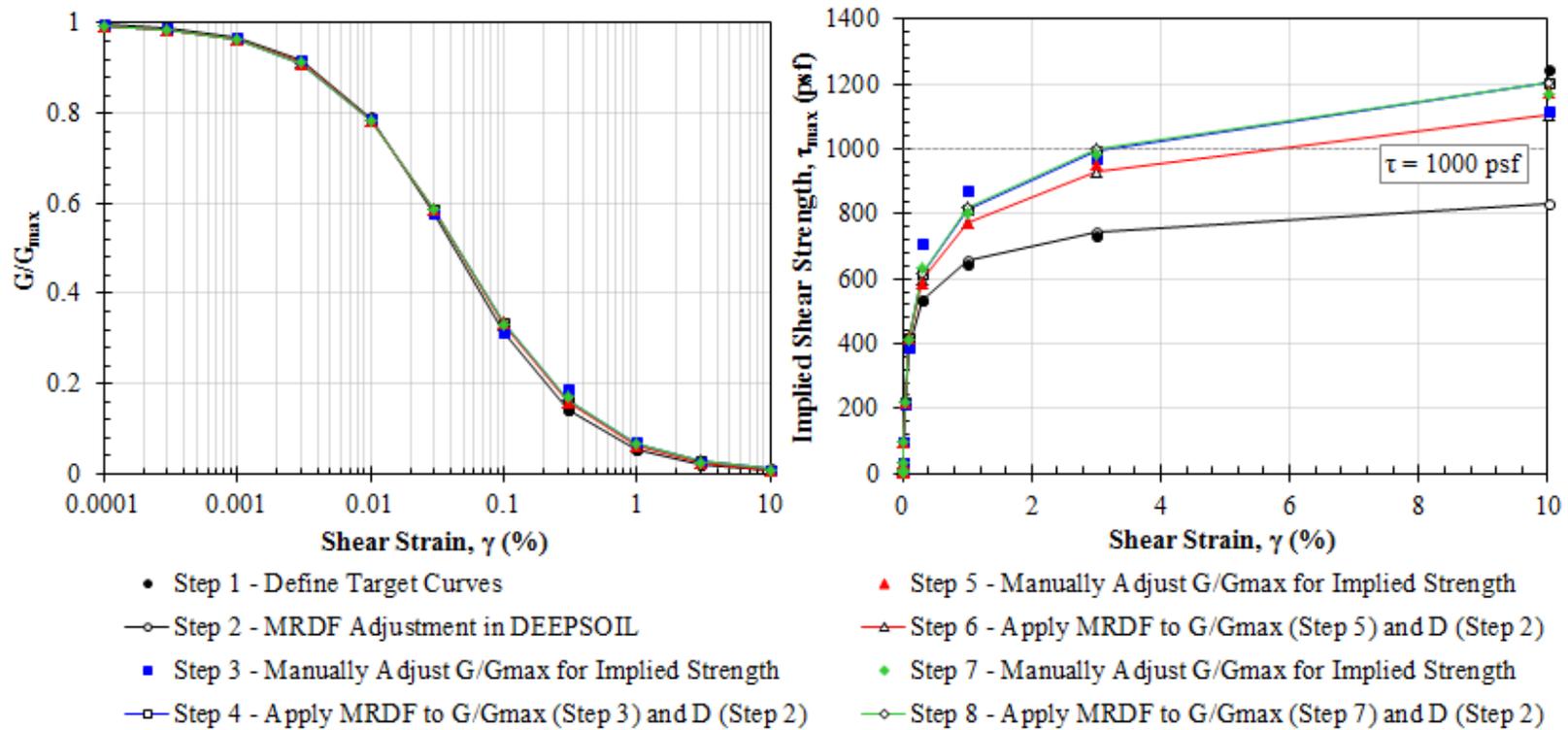
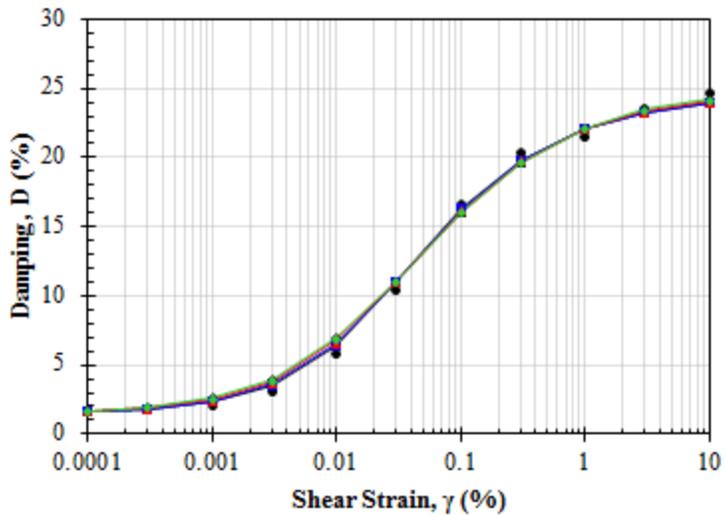
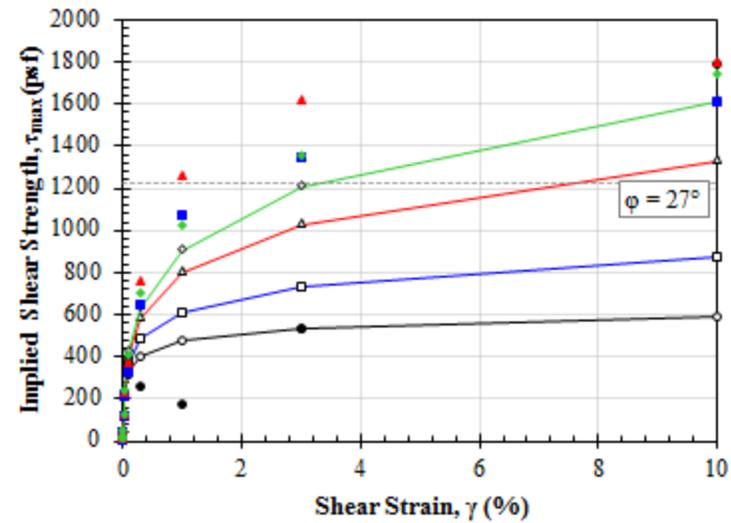
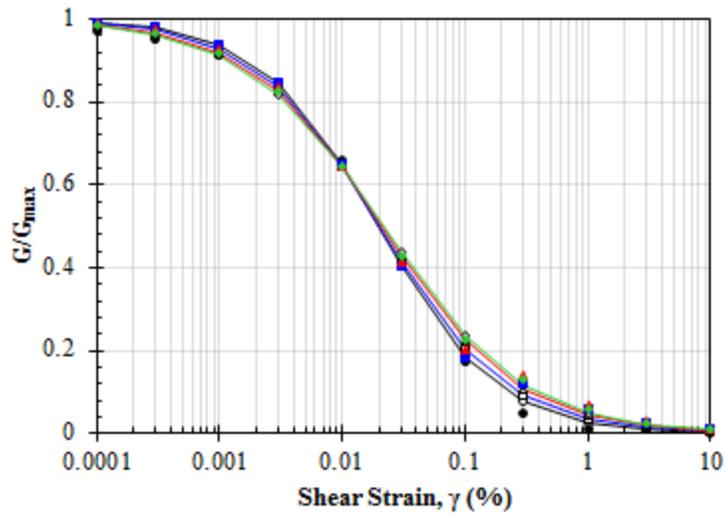
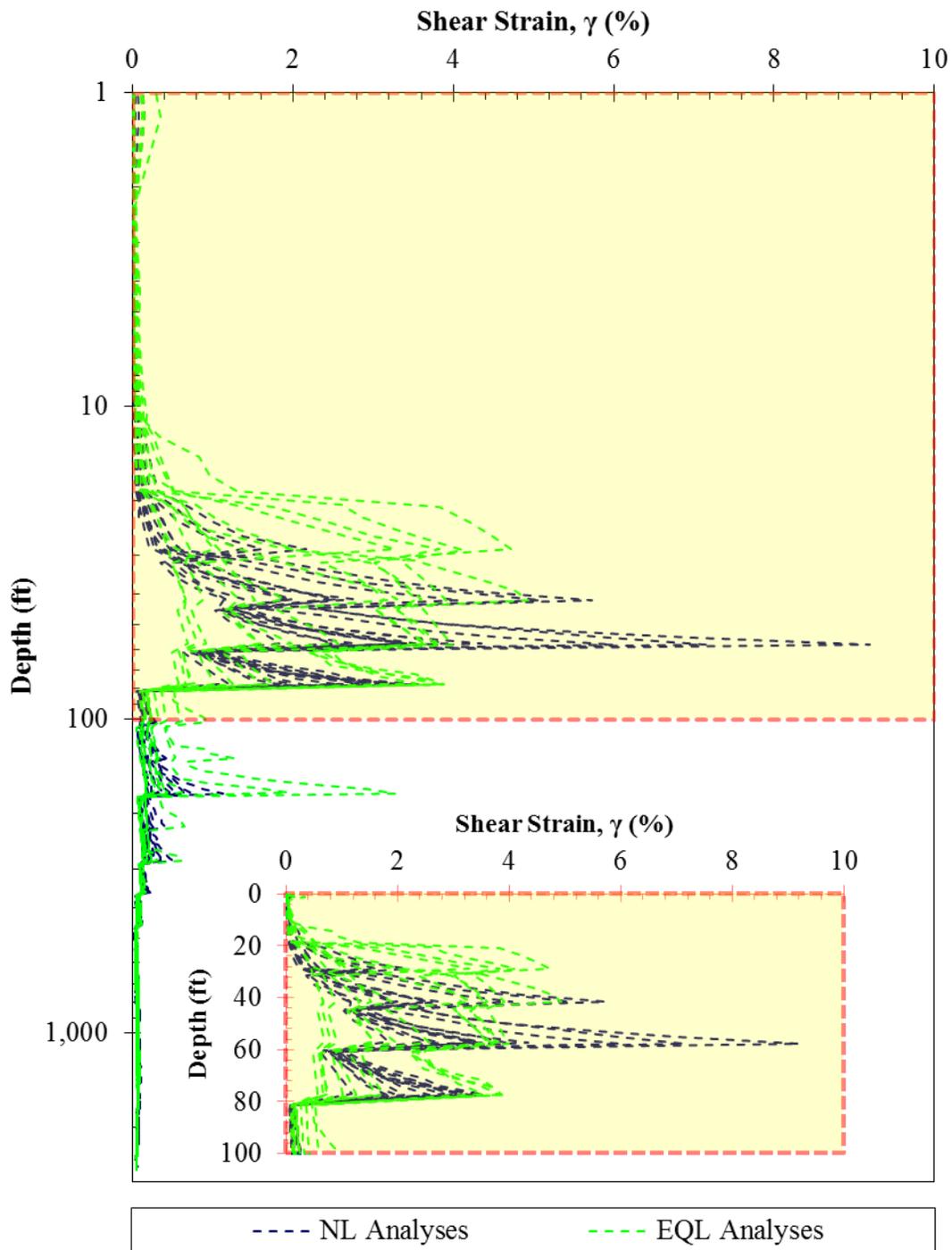


Figure 18. Example of procedure for modifying Darendeli (2001) modulus and damping curves to account for implied shear strength at large strains in a cohesive soil layer.

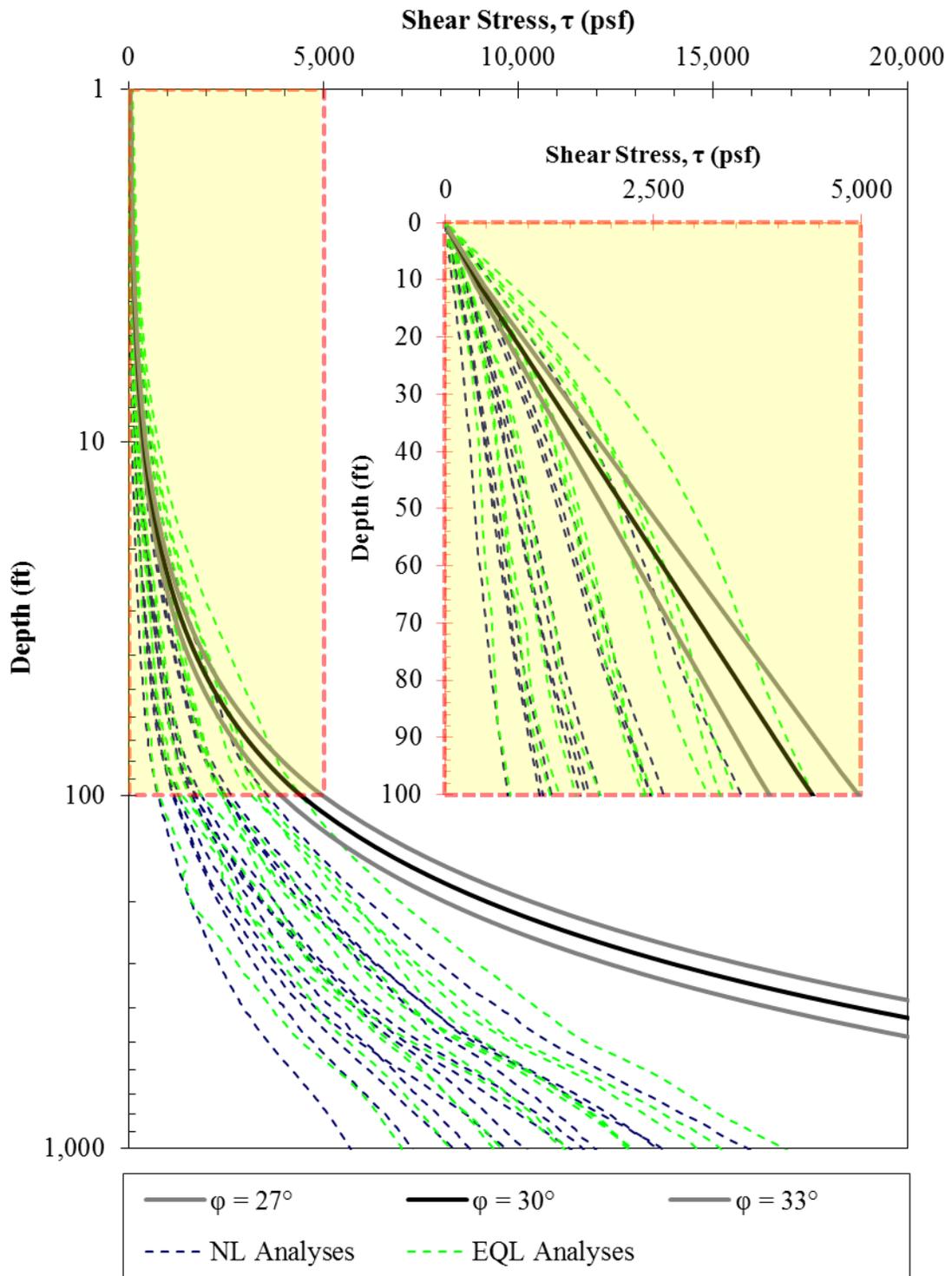


- Step 1 - Define Target Curves
- Step 2 - MRDF Adjustment in DEEPSOIL
- Step 3 - Manually Adjust G/Gmax for Implied Strength
- Step 4 - Apply MRDF to G/Gmax (Step 3) and D (Step 2)
- ▲ Step 5 - Manually Adjust G/Gmax for Implied Strength
- △ Step 6 - Apply MRDF to G/Gmax (Step 5) and D (Step 2)
- ◆ Step 7 - Manually Adjust G/Gmax for Implied Strength
- ◇ Step 8 - Apply MRDF to G/Gmax (Step 7) and D (Step 2)

Figure 19. Example of procedure for modifying Darendeli (2001) modulus and damping curves to account for implied shear strength at large strains in a cohesionless soil layer.



**Figure 20. Shear strain profiles from preliminary site response analyses, showing large strains in the top 100 feet of the soil column.**



**Figure 21. Shear stress profiles from preliminary site response analyses, indicating possible shear failure in top 100 feet of the soil column.**

**Table 4. Depths, soil properties and target strength values for five aggregate layers (curve sets 1 – 5) defining the top 100 feet of the soil profiles**

	Curve Set 1	Curve Set 2	Curve Set 3	Curve Set 4	Curve Set 5	
Depth (ft)	0 - 20	20 - 30	30 - 50	50 - 70	70 - 100	
Thickness (ft)	20	10	20	20	30	
Mid-Depth (ft)	10	25	40	60	85	
$\sigma_v'$ (psf)	576	1440	2405.4	3757.4	5447.4	
PI	60	20	0	0	0	
$\gamma_t$ (pcf)	120	120	130	130	130	
$V_s$ (ft/s)	506	578	666	768	1023	
$G_{max}$ (psf)	955238	1243413	1790400	2380518	4222444	
Target Strength	+ $\sigma_{NG}$ - $\sigma_D$	$\tau_{ult} = 2000$ psf	$\tau_{ult} = 1300$ psf	$\phi = 33^\circ$	$\phi = 33^\circ$	$\phi = 33^\circ$
	$\mu_{NG}$ $\mu_D$	$\tau_{ult} = 1500$ psf	$\tau_{ult} = 1000$ psf	$\phi = 30^\circ$	$\phi = 30^\circ$	$\phi = 30^\circ$
	- $\sigma_{NG}$ + $\sigma_D$	$\tau_{ult} = 1000$ psf	$\tau_{ult} = 700$ psf	$\phi = 27^\circ$	$\phi = 27^\circ$	$\phi = 27^\circ$

Each aggregate layer contains multiple sub-layers within the soil profiles used for analyses in DEEPSOIL. After MRDF-ISSC modulus reduction and damping curves were generated for any given aggregate layer, the corrected curves were assigned to all of the thinner DEEPSOIL sub-layers located within the depth of the aggregate layer. Each MRDF-ISSC adjustment utilized the lognormal median (LNM)  $V_s$  profile for calculating implied shear stresses from the modulus reduction curve according to Equation 1.

## 4. INPUT GROUND MOTIONS

### 4.1. SITE-SPECIFIC TARGET SPECTRUM

AASHTO Guide Specifications for Seismic Design (2009) specify seismic hazard based on the 2002 USGS Seismic Hazard Maps (USGS, 2002). AASHTO seismic design forces are based on ground motions (GM's) with a 7% probability of being exceeded over a period of 75 years. Technically however, the ground motions are from USGS maps based on a 5% in 50-year hazard. The two hazard levels are nearly equivalent, corresponding to a return period of approximately 1000 years.

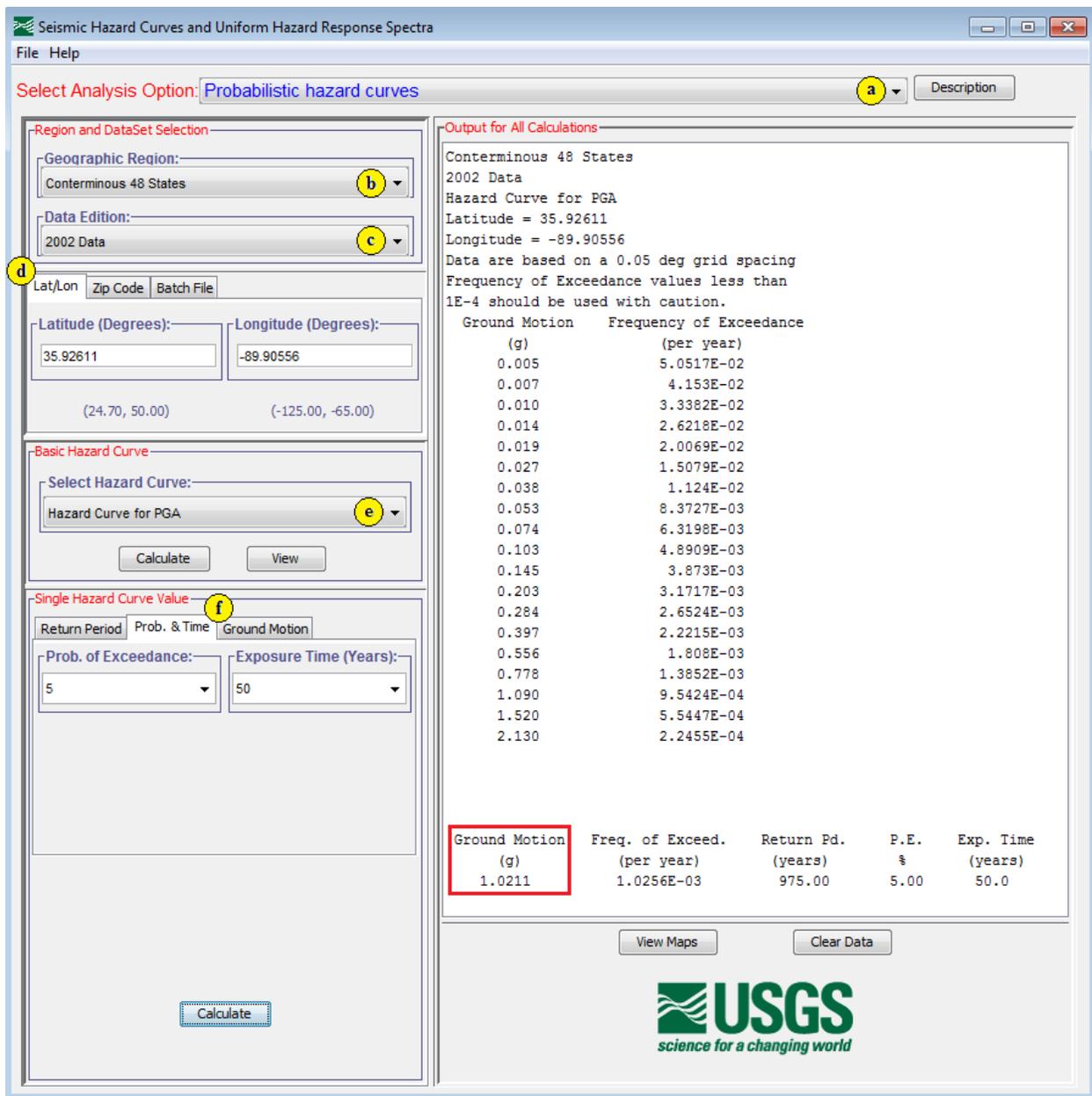
The Java Ground Motion Parameter Calculator (JGMPC) was used to construct a uniform hazard spectrum (UHS) based on a 5% in 50-year hazard for the example site. This UHS serves as the target spectrum for GM selection and scaling. Figure 22 illustrates the user interface and output for the JGMPC. The UHS is constructed by calculating the GM at each period available from the JGMPC (0, 0.1, 0.2, 0.3, 0.5, 1.0 and 2.0 seconds) and then plotting ground motion (in terms of spectral acceleration,  $S_a$ ) versus period (T). Straight line interpolation is assumed between the known points. These data are often plotted with period on a log scale to provide more detail in the short-period end of the spectrum, which governs the designs of most small to moderate-sized structures. In these cases, it is not sufficient to plot the PGA (T = 0) ground motion at T = 0.01 seconds or any other value other than zero; instead, the smallest value on the log scale (e.g., 0.01 sec) must be linearly interpolated.

The JGMPC calculates GM's corresponding to the boundary between Site Class B/C. In the current study, the input ground motions need to be appropriate for Paleozoic bedrock that is widely considered to be Site Class A. Therefore, the Site Class B/C UHS must be scaled to Class A conditions before constructing the appropriate target spectrum. Table 5 summarizes the data used to construct the Site Class A UHS target spectrum for the example bridge site in Blytheville, Arkansas. The reader may refer to Figure 10 (Section 2.2.1) for a comparison of the Site Class A UHS target spectrum with the Site Class B UHS.

After developing a target spectrum, deaggregations were conducted to determine which earthquake magnitudes and distances contribute most to the seismic hazard at the example site. The USGS application, 2002 Interactive Deaggregations, was used for this purpose. The user interface and instructions are illustrated in Figure 23. Note that deaggregations are different at each period, and should be examined period-by-period. As shown in Figure 24, the seismic hazard at the example site is governed by approximately the same earthquake characteristics at each period: magnitude 7.6 at a distance of 13 km (8 mi).

### 4.2. INPUT GROUND MOTION SELECTION

Artificial input GM's could be simulated with the USGS deaggregation tool, but the simulation involves a number of assumptions that add considerable uncertainty to the site



**Figure 22. Java ground motion parameter calculator for developing a uniform hazard spectrum: (a) select “Probabilistic hazard curves”; (b) select “Conterminous 48 States”; (c) select “2002 Data”; (d) select the “Lat/Lon” tab and enter the coordinates of the job site; (e) select the desired period and click “Calculate” in the “Basic Hazard Curve” section; (f) select the “Prob. & Time” tab, select “5” for Prob. Of Exceedance and “50” for Exposure Time, and click “Calculate” in the “Single Hazard Curve Value” section; record the calculated ground motion and associated period.**

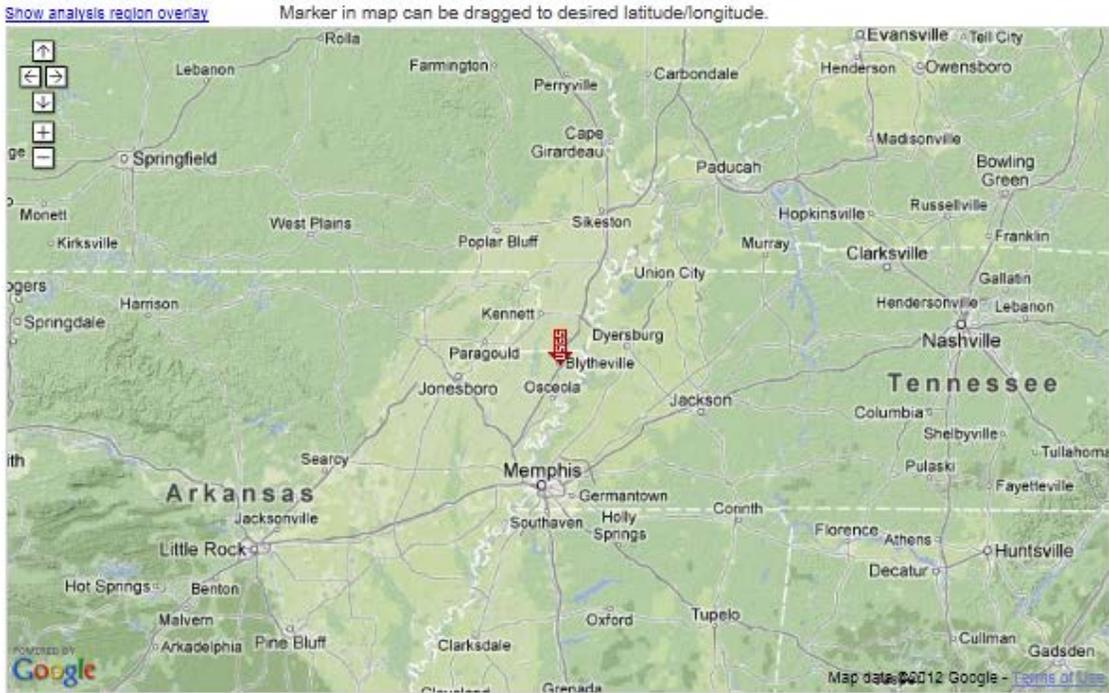
**Seven Input Fields Provided Below for Deaggregation Calculations**  
 (Click the input box labels for more information. Click the Generate Output button at the bottom of the form to perform calculations.)

**Site Name:**  (a) REQUIRED: Alphanumeric characters, commas, underscores, or blank spaces only.

[Switch to generic address input](#)

**Latitude:**  (b) REQUIRED: Specify in decimal degrees (e.g., 35.2).

**Longitude:**  (c) REQUIRED: Specify as negative decimal degrees (e.g., -100.2) indicating the Western Hemisphere.



**Return Period:**  (d) REQUIRED: General design maps consider the 2% in 50 years option which is the default selection.

**Spectral Acceleration:**  (e) REQUIRED: Frequency calculations done in groups (group 1 - PGA, 1, & 5 Hz / group 2 - 0.5, 2, 3.33, & 10 Hz).

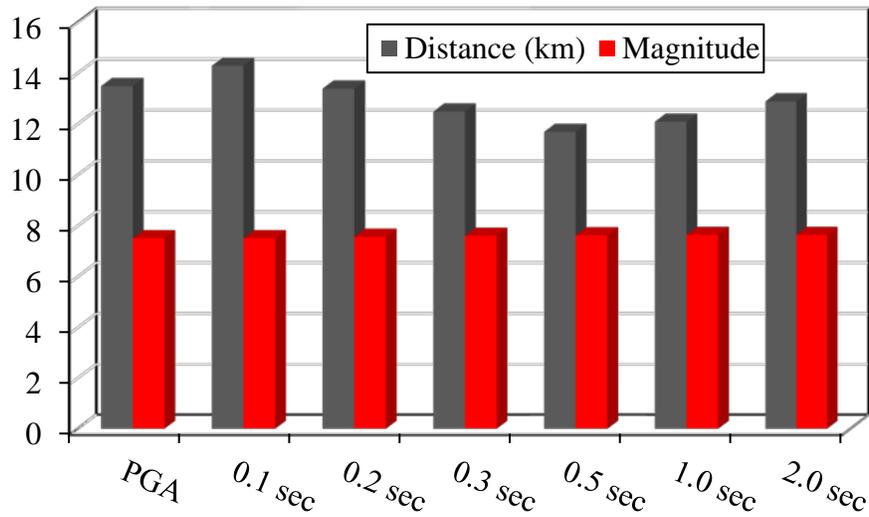
**Geographic Deaggregations:**  (f) OPTIONAL: Included to aid in visualizing relative hazards from various sources.

**Stochastic Seismograms:**  (g) OPTIONAL: Additional output resource.

**Generate Output** (h)

Please read the [README](#) file to understand limitations of this application and to ensure valid application results. The README file explains about the input parameters and also describes how the deaggregation is done. [Stochastic Seismograms](#) and [What is Epsilon?](#) are articles which discuss the theory behind the seismograms.

**Figure 23. Interactive deaggregation application for 2002 USGS Seismic Hazard Maps: (a) enter name of structure or site; (b), (c) enter latitude and longitude in decimal degrees; (d) select “5% in 50 years” as per AASHTO specifications; (e) select period for deaggregation; (f) option for geographic representation of results; (g) option to generate synthetic input ground motions; (h) click button to perform the deaggregation.**



**Figure 24. Mean deaggregation results for the Blytheville example bridge site obtained from USGS interactive deaggregation of the 2002 probabilistic seismic hazard analysis at each period available for calculation.**

**Table 5. Output from java ground motion parameter calculator and conversion to Site Class A to develop UHS target spectrum for example bridge site in Blytheville, AR**

Period (sec)	Calculated Ground Motion for Site Class B/C (g)	Scaling Factor	Site Class A Ground Motion (g)
0 (PGA)	1.0211	1.52	0.672
0.1	2.1308	1.74	1.225
0.2	1.8551	1.76	1.054
0.3	1.6417	1.72	0.954
0.5	1.1493	1.58	0.727
1.0	0.5482	1.34	0.409
2.0	0.2366	1.20	0.197

response analysis. As noted previously, however, large-earthquake and short-distance GM's have never been recorded in the central and eastern United States (CEUS). Such GM's have only been recorded in other parts of the world with different tectonic settings or where relatively soft rock formations attenuate high-frequency energy. Stiffer rock formations in the CEUS are expected to transmit more high-frequency energy. As discussed in Section 2.2.2, McGuire et al. (2001) modified a suite of recorded GM's from around the world to incorporate the expected high-frequency energy for seismic design in the CEUS. After screening the McGuire et al. (2001) GM's for distance and magnitude, and taking no more than two sets of records from any given earthquake event, 16 pairs of horizontal input GM components were selected for consideration. Each input GM chosen for consideration is described in Table 6. Note individual input GM's are hereafter discussed in terms of the ID numbers assigned in Table 6.

**Table 6. Summary of input ground motions selected after screening the suite of modified ground motions by McGuire et al. (2001)**

NUREG CEUS GMs								
ID	FileName	EQ	PGA (g)	M	R (km)	Geom	USGS	Dur (sec)
I	DAY-LN	Tabas, Iran	0.993	7.4	17	ABB	-	8.8
II	DAY-TR	Tabas, Iran	0.947	7.4	17	ABB	-	9.7
III	GBZ000	Kocaeli, Turkey	0.454	7.4	17	--A	A	7.3
IV	GBZ270	Kocaeli, Turkey	0.34	7.4	17	--A	A	6.4
V	TCU046-N	Chi-Chi, Taiwan	0.224	7.6	14.3	--1	A	25
VI	TCU046-W	Chi-Chi, Taiwan	0.336	7.6	14.3	--1	A	18.8
VII	BRN000	Loma Prieta	1.296	6.9	10.3	--A	-	7.2
VIII	BRN090	Loma Prieta	1.236	6.9	10.3	--A	-	8.6
IX	LCN260	Landers	1.844	7.3	1.1	A-A	A	10
X	LCN345	Landers	2.249	7.3	1.1	A-A	A	10.9
XI	IZT180	Kocaeli, Turkey	0.401	7.4	7.7	--A	A	10.1
XII	IZT090	Kocaeli, Turkey	0.349	7.4	7.7	--A	A	10.1
XIII	LGP000	Loma Prieta	1.642	6.9	6.1	--A	-	5.1
XIV	LGP090	Loma Prieta	1.645	6.9	6.1	--A	-	5.5
XV	CPM000	Cape Mendocino	4.721	7.1	8.5	IFA	A	3.1
XVI	CPM090	Cape Mendocino	3.242	7.1	8.5	IFA	A	0.6
XVII	CLS000	Loma Prieta	1.301	6.9	5.1	APB	B	3.7
XVIII	CLS090	Loma Prieta	1.207	6.9	5.1	APB	B	3.7
XIX	KJM000	Kobe	1.924	6.9	0.6	--B	B	6.1
XX	KJM090	Kobe	1.559	6.9	0.6	--B	B	5.7
XXI	TCU128-N	Chi-Chi, Taiwan	0.305	7.6	9.7	--1	B	29.9
XXII	TCU128-W	Chi-Chi, Taiwan	0.266	7.6	9.7	--1	B	35.7
XXIII	SHL000	Cape Mendocino	0.648	7.1	33.8	IFB	B	14.4
XXIV	SHL090	Cape Mendocino	0.585	7.1	33.8	IFB	B	14.6
XXV	ARC000	Kocaeli, Turkey	0.298	7.4	17	--B	B	10.4
XXVI	ARC090	Kocaeli, Turkey	0.298	7.4	17	--B	B	9.1
XXVII	TCU047-N	Chi-Chi, Taiwan	1.168	7.6	33	---	B	10.8
XXVIII	TCU047-W	Chi-Chi, Taiwan	0.7	7.6	33	---	B	12.9
XXIX	BOL000	Duzce, Turkey	1.645	7.1	16	--D	C	4.5
XXX	BOL090	Duzce, Turkey	1.765	7.1	16	--D	C	6.4
XXXI	IZN180	Kocaeli, Turkey	0.239	7.4	29.7	--D	C	14.5
XXXII	IZN090	Kocaeli, Turkey	0.175	7.4	29.7	--D	C	13.8

Response spectra (with 5% damping) for the 32 GM's considered for the site response analyses are plotted along with their lognormal median and the site-specific target spectrum in Figure 25. The lognormal median of all GM response spectra matches the spectral shape of the target spectrum quite well at periods above approximately 0.1 seconds. Recall also that the 32 GM's were recorded from locations around the world, each with unique geology, magnitude and distance characteristics. AASHTO only requires 3 to 7 input ground motions for site response analyses, but the larger number considered for this study was justified by the uncertainty associated with unknown ground motion characteristics in the Mississippi Embayment.

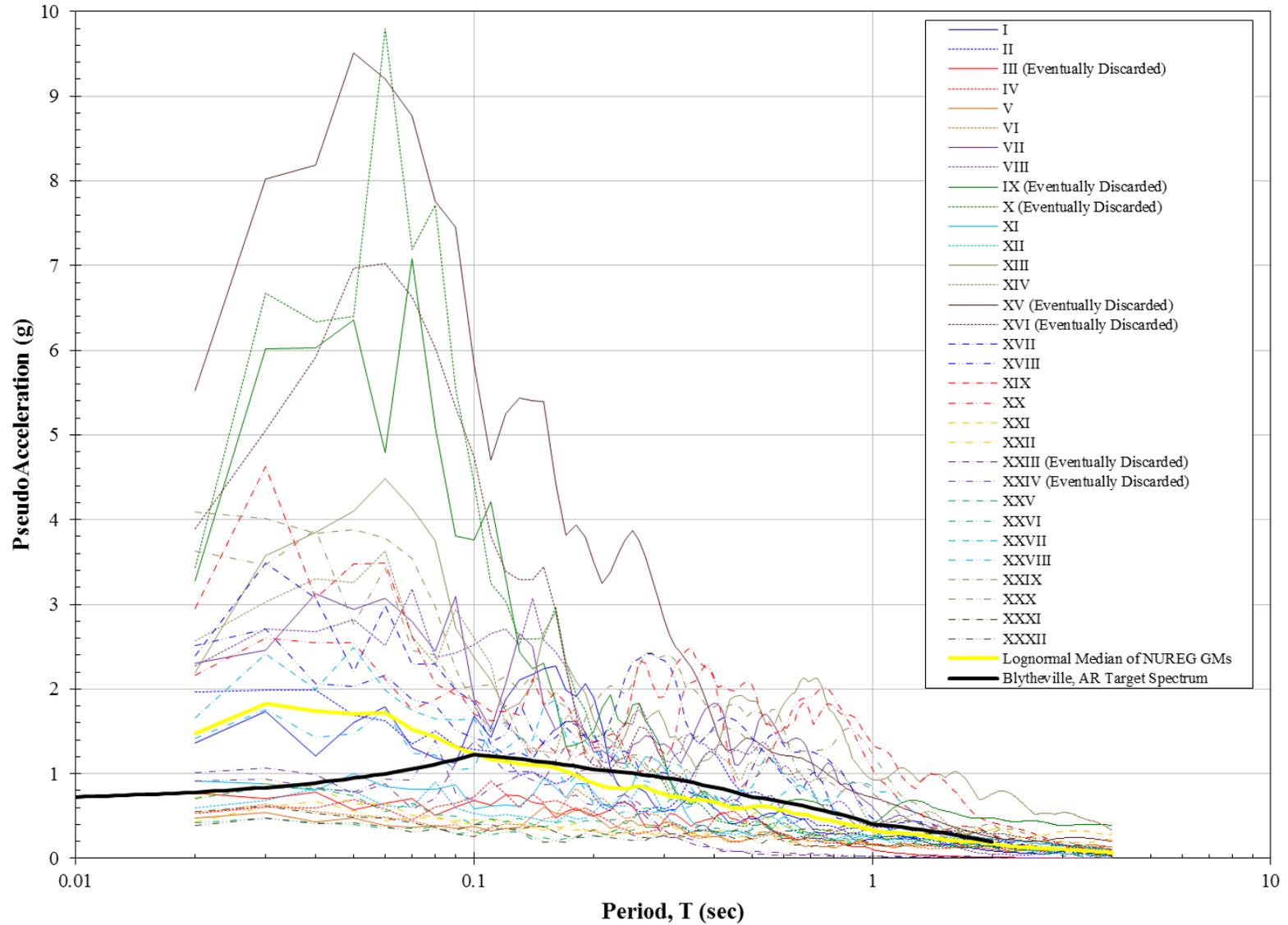
In working with ground motions (more specifically, acceleration time histories), file formatting is an important consideration. Computer programs for spectral matching and site response analyses typically load and output ground motions in a specific format. For example, the time step may be included as a header, and the recorded acceleration values listed in rows of four or five entries. The input/output may also be in terms of a Fourier amplitude spectrum (FAS) or response spectrum instead of a time history. SeismoSignal (available free from [www.seismosignal.com](http://www.seismosignal.com)) is a program that allows users to easily change the format of ground motion records and also generate other aspects of the ground motion such as FAS and response spectra.

#### 4.3. SPECTRAL MATCHING

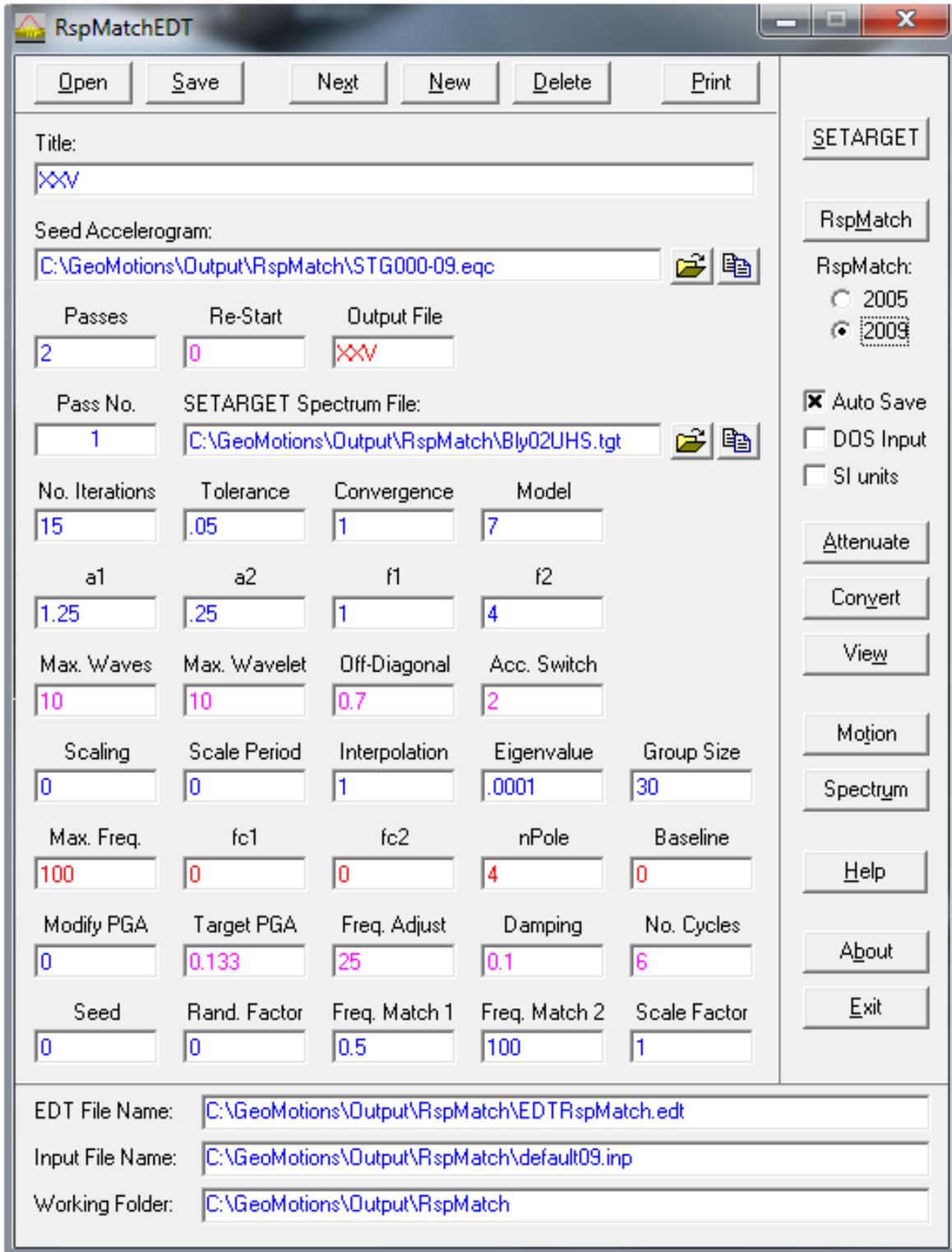
RSPMatch 2009 was used to spectrally match each of the 32 candidates for input GM's to the site-specific target spectrum. The "SETARGET" feature was used to generate a linear interpolation between known points on the target spectrum (i.e., the site-specific UHS). Spectral matching was conducted using two passes: (1) from 1 to 100 Hz, and (2) from 0.5 to 100 Hz. Settings for the first pass are shown in Figure 26; only the "Freq. Match 1" entry is changed for the second pass.

Spectral matching is most effectively explained by comparing the graphs presented in Figure 27. Observe the difference between the original and matched acceleration time histories. No portion of the original time history is removed; rather, various wavelets are added at specific times. Modifications to the acceleration time history are most visible, with the effect on the velocity and displacement time histories progressively less pronounced. Correspondingly, the pseudo acceleration response spectrum undergoes a marked change and more closely resembles the target spectrum. Other parameters such as arias intensity are affected, but not drastically.

The spectrum-matched input ground motions are shown in Figure 28, Ground motions III, IX, X, XV, XVI, XXIII and XXIV were not used in subsequent analyses because their spectral shapes did not match the target spectrum particularly well before and/or after spectral matching. The discarded ground motions are included in Figure 28 only to illustrate the extent to which they deviated from the target spectrum. For example, ground Motions IX and X maintained a considerable mismatch at low periods after spectral matching. Ground motions were mostly discarded for mismatches at the shortest or longest periods of the target spectrum. After discarding the seven motions noted above, the 25 remaining, spectrally-matched GM's were selected as input GM's for the site response analyses.



**Figure 25. Input ground motion response spectra (5% damping) selected for the example bridge site in Blytheville, AR with lognormal median and target spectrum.**



**Figure 26. Settings used in RspMatch 2009 to spectrally match ground motions to the target spectrum for input ground motions.**

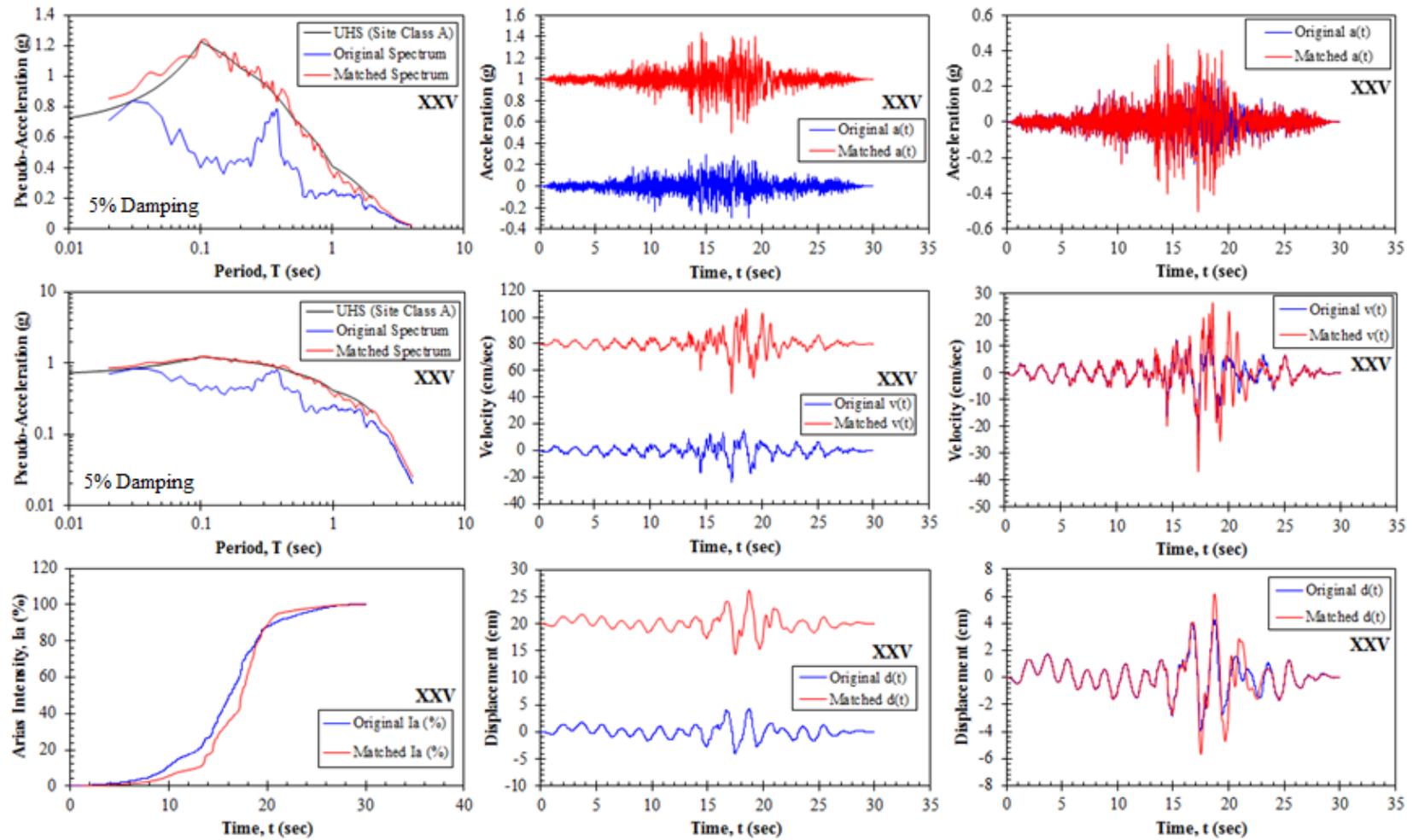
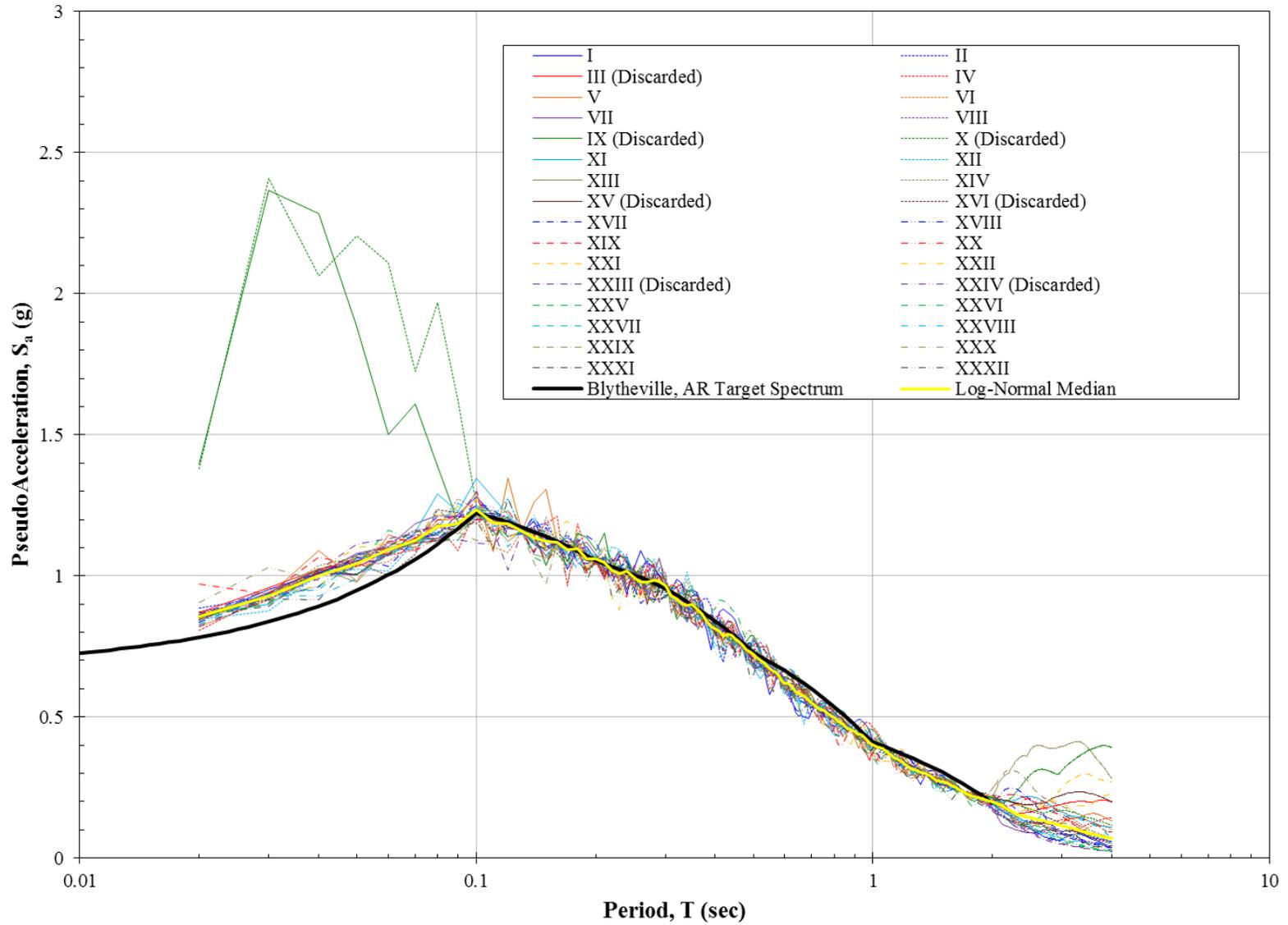


Figure 27. Effects of spectral matching on input ground motion XXV.



**Figure 28. Spectrally matched input ground motion response spectra (5% damping) and target spectrum for site response analyses at the example bridge site in Blytheville, AR**

## 5. FRAMEWORK FOR SITE RESPONSE ANALYSES

### 5.1. INPUT ALTERNATIVES AND ANALYSIS METHODS USED IN SITE RESPONSE ANALYSES

Section 3 describes multiple  $V_s$  profiles and dynamic soil properties that were chosen to represent a reasonable range of properties that may be exhibited by the soil profile at the example bridge site in Blytheville, AR. Because preliminary analyses revealed “large” shear strains, it was also necessary to modify the dynamic soil properties and conduct both equivalent linear (EQL) and fully non-linear (NL) site response analyses. Six hundred design-level site response analyses were required in order to account for the variety of input and analysis considerations.

Each analysis, or “run”, has been assigned a unique “run number”. Table 7 summarizes the 300 distinct sets of assumptions associated with each NL analysis. The EQL analyses are summarized in Table 8. Note that Tables 7 and 8 include the preliminary runs that were conducted prior to implied shear strength corrections in the top 100 feet of the soil profile. However, design-level analyses are limited to the runs with implied shear strength corrections (i.e., run numbers greater than 100).

Each “run” describes a batch analysis of 25 spectrally matched input bedrock ground motions (described in Section 4, shown in Figure 28 and summarized in Table 6), all analyzed with the same soil profile. The surface response from any given run is presented in this report as the lognormal median (LNM) of the individual surface response from each of the 25 input bedrock motions. In order to illustrate this, Figure 29 summarizes a single “run”, including the output (surface response spectra) from all 25 input “rock” motions, the computed LNM of the output response spectra and the code-based Site Class E and 2/3 Site Class E spectra. Surface responses from EQL and NL analyses are presented and discussed in Sections 6 and 7, respectively.

$V_s$  profiles and dynamic soil properties ( $G/G_{\max}$  and damping) were varied simultaneously to reveal any compounding and/or compensating effects on the surface response, particularly in the period range of interest (i.e., 0.1 – 0.5 seconds). The dynamic properties used include the mean modulus and mean damping curves ( $\mu_G/\mu_D$ ), the plus one standard deviation modulus and minus one standard deviation damping curves ( $+\sigma_G/-\sigma_D$ ), and the minus one standard deviation modulus and plus one standard deviation damping curves ( $-\sigma_G/+\sigma_D$ ). In other words, for each  $V_s$  profile, three separate “runs” were defined in order to consider all three sets of dynamic soil property curves with each  $V_s$  profile. The same 12 run scenarios were conducted using both NL and EQL techniques, resulting in a total of 24 different runs (600 distinct analyses).

Each different  $V_s$  profile represents not only a unique profile of soil stiffness, but also a different depth to bedrock. The effect of varying depth to bedrock independently is addressed with sensitivity analyses in Sections 6 and 7. Tables 9 and 10 summarize the input parameters for the sensitivity analyses, where all properties of the soil profile were held constant except for the depth to bedrock. Both NL (Table 9) and EQL (Table 10) methods of analysis were considered. The Gosnell  $V_s$  profile and mean  $G/G_{\max}$  ( $\mu_G$ ) and Damping ( $\mu_D$ ) curves define the

**Table 7. Summary of run numbers assigned to each fully non-linear (NL) site response analysis for the example bridge site in Blytheville, Arkansas**

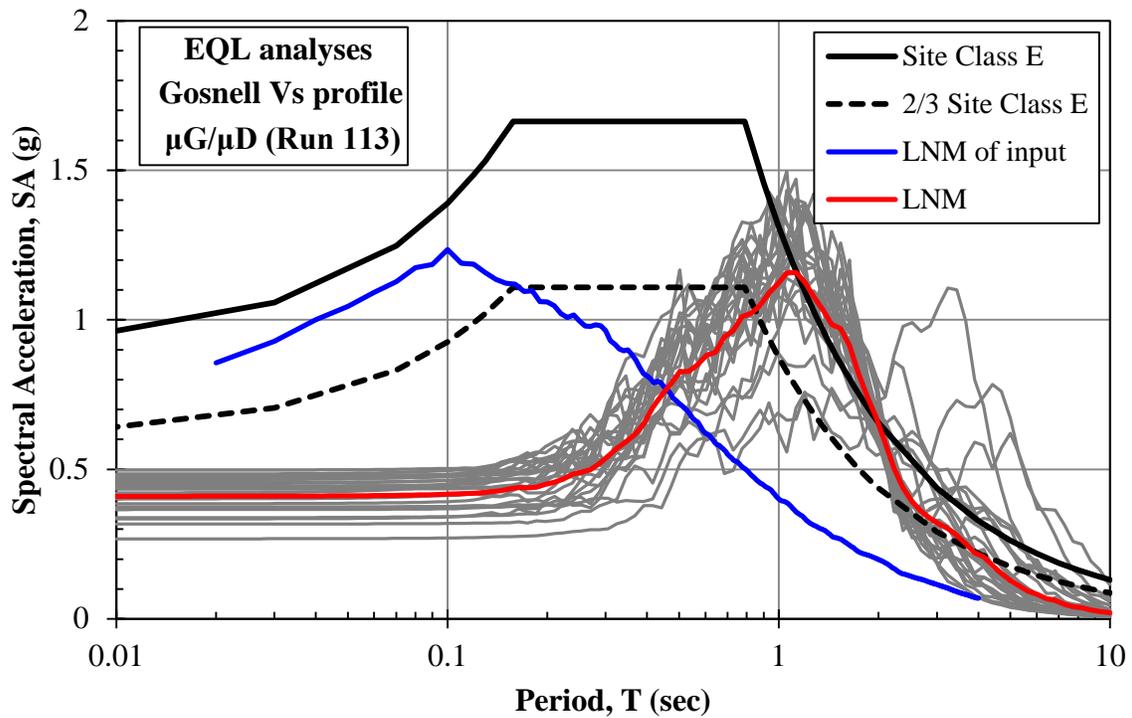
NL Analyses		Darendeli (2001) Dynamic Soil Properties					
		MRDF			MRDF-ISSC		
V <sub>s</sub> Profile	Depth to Rock (ft)	μ <sub>G</sub>	+σ <sub>G</sub>	-σ <sub>G</sub>	μ <sub>G</sub>	+σ <sub>G</sub>	-σ <sub>G</sub>
		μ <sub>D</sub>	-σ <sub>D</sub>	+σ <sub>D</sub>	μ <sub>D</sub>	-σ <sub>D</sub>	+σ <sub>D</sub>
Gosnell	2570	1	2	3	101	102	103
Log-Normal Median	2660	4	5	6	104	105	106
LNM + 20%	2570	7	8	9	107	108	109
LNM - 20%	2760	10	11	12	110	111	112

**Table 8. Summary of run numbers assigned to each equivalent linear (EQL) site response analysis for the example bridge site in Blytheville, Arkansas**

EQL Analyses		Darendeli (2001) Dynamic Soil Properties					
		MA-EQL			MRDF-ISSC		
V <sub>s</sub> Profile	Depth to Rock (ft)	μ <sub>G</sub>	+σ <sub>G</sub>	-σ <sub>G</sub>	μ <sub>G</sub>	+σ <sub>G</sub>	-σ <sub>G</sub>
		μ <sub>D</sub>	-σ <sub>D</sub>	+σ <sub>D</sub>	μ <sub>D</sub>	-σ <sub>D</sub>	+σ <sub>D</sub>
Gosnell	2570	13	14	15	113	114	115
Log-Normal Median	2660	16	17	18	116	117	118
LNM + 20%	2570	19	20	21	119	120	121
LNM - 20%	2760	22	23	24	122	123	124

**Table 9. Summary of run numbers assigned to each NL site response analysis conducted to study sensitivity of depth to bedrock for the example bridge site in Blytheville, AR**

NL Analyses		Darendeli (2001) Dynamic Soil Properties	
Depth to Rock (ft)		μ <sub>G</sub> /μ <sub>D</sub> (MRDF)	μ <sub>G</sub> /μ <sub>D</sub> (MRDF-ISSC)
Gosnell V <sub>s</sub> Profile	1910	31	131
	2240	32	132
	2470	33	133
	2670	34	134
	2900	35	135
	3220	36	136



**Figure 29. Equivalent linear analyses with all 25 output response spectra (gray lines) and the computed lognormal median, including lognormal median of the 25 input response spectra and the code-based Site Class E and 2/3 Site Class E Spectra.**

**Table 10. Summary of run numbers assigned to each NL site response analysis conducted to study sensitivity of depth to bedrock for the example bridge site in Blytheville, AR**

EQL Analyses		Dynamic Soil Properties	
Depth to Rock (ft)		$\mu_G/\mu_D$ (MA-EQL)	$\mu_G/\mu_D$ (MRDF-ISSC)
Gosnell $V_s$ Profile	1910	25	125
	2240	26	126
	2470	27	127
	2670	28	128
	2900	29	129
	3220	30	130

default profile for these analyses. In this manner, runs 101 and 113 from Tables 7 and 8, respectively, define the baseline conditions for the sensitivity analyses. Comparisons are based on the 5% damped surface response spectra and shear strain profiles.

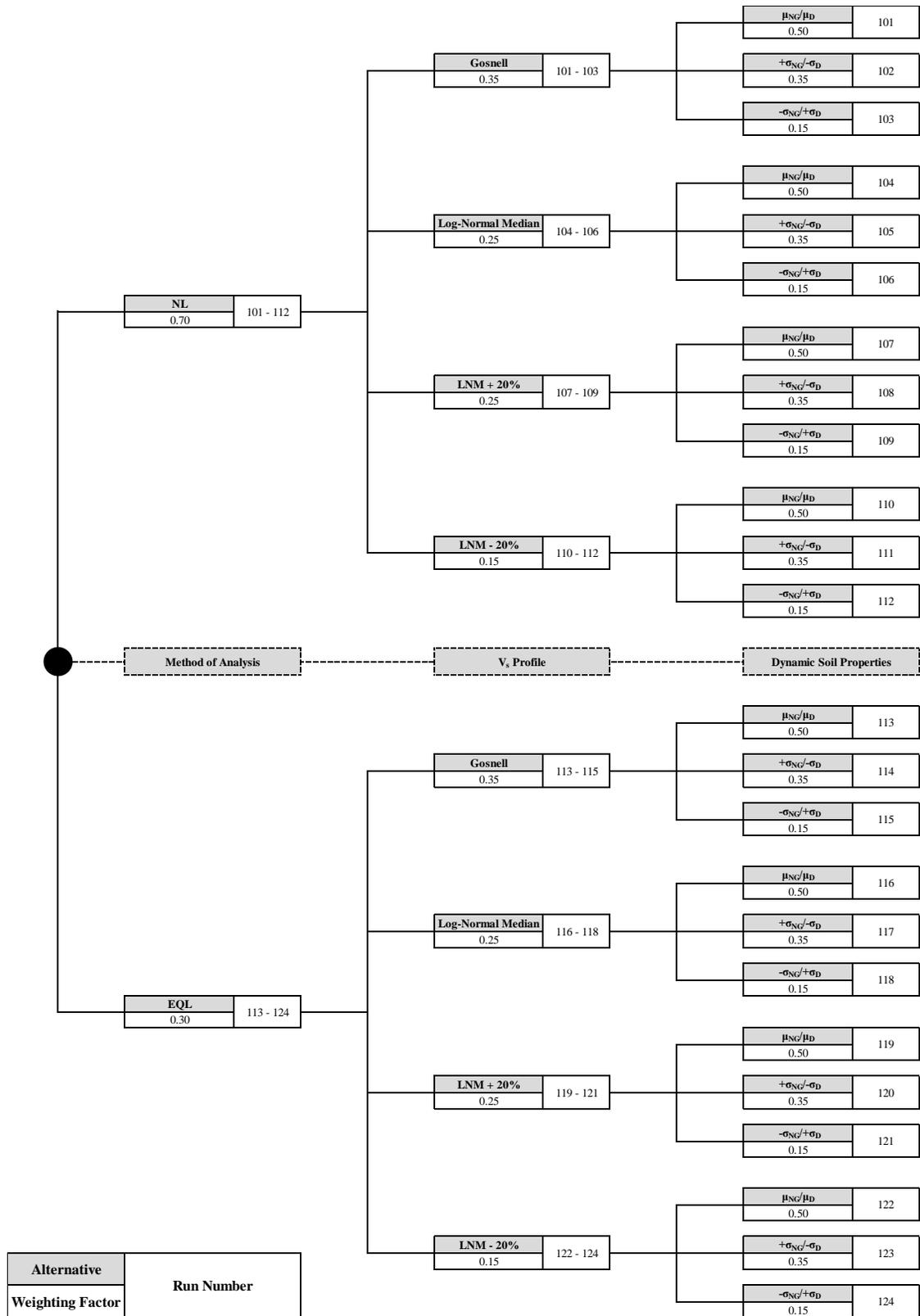
## 5.2. WEIGHTING FACTORS FOR RESULTS FROM DESIGN-LEVEL ANALYSES

Twenty-four unique site response “runs” were conducted in order to consider the effects of potential variations in the soil profile and differences between NL and EQL analyses. Working without direct measurements of  $V_s$  and dynamic soil properties in a region where “large” shear strains are expected, it is particularly important to account for these alternative subsurface scenarios and analysis methods. As depicted in Figure 30, weighting factors have been applied to each run in the form of a logic tree. The weighting factors in each branch of a logic tree must add to 1.0, the alternatives populating a branch must be mutually exclusive, and the magnitude of the weighting factor for any given alternative should reflect the engineer’s confidence in that alternative relative to the other alternatives in the same branch. For this study, weighting factors were chosen to impart conservatism with respect to the computed surface ground motions.

Three different sets of dynamic soil properties were considered in the analyses. All of the nonlinear curves were adjusted to correct for implied shear strengths in the top 100 feet. Mean curves ( $\mu_G, \mu_D$ ) represent the best fit through the empirical data from which the curves were developed (Darendeli, 2001). The upper bound on stiffness ( $+\sigma_G, -\sigma_D$ ) represents stiffer soil layers through which more high-frequency energy can propagate upward, resulting in larger short-period ground motions at the surface. For sites in the NMSZ, where considerable high-frequency energy is expected, the stiffest set of dynamic soil properties ( $+\sigma_G, -\sigma_D$ , upper bound) provides the most conservative estimate (in terms of surface acceleration) for short-period bridges. In contrast, the lower bound on stiffness ( $-\sigma_G, +\sigma_D$ ) causes soil layers to exhibit greater nonlinearity, allowing the greatest amount of high-frequency attenuation. Thus, the lower bound on dynamic soil properties corresponds to the smallest short-period design ground motions, making it the least conservative choice for short-period bridges. In light of the uncertainty associated with characterizing the deep sediments of the Mississippi Embayment, relatively conservative weighting factors were applied as follows:

- |  |      |   |                |
|--|------|---|----------------|
| • Mean Curves ( $\mu_G, \mu_D$ ):                | 0.5  | } | $\Sigma = 1.0$ |
| • Upper Bound Curves ( $+\sigma_G, -\sigma_D$ ): | 0.35 |   |                |
| • Lower Bound Curves ( $-\sigma_G, +\sigma_D$ ): | 0.15 |   |                |

The four  $V_s$  profiles were weighted in a similar manner, with the additional consideration of geometric proximity to the bridge site in Blytheville, AR. The Gosnell profile is closest to the example site, and represents the second-stiffest design profile through most depths. Recall that the Gosnell profile was chosen as a design profile specifically because of its proximity to the site and for the conservative implications of a stiff  $V_s$  profile. The LNM profile is based on four different  $V_s$  profiles in Northeast Arkansas, providing a seemingly reliable index for a region with relatively little variability in subsurface characteristics. The upper and lower bound (LNM +/- 20%)  $V_s$  profiles were considered solely to provide an appropriate range of potential soil stiffness. As with the dynamic soil properties, the upper bound  $V_s$  profile (LNM + 20%) is the most conservative option for short-period ground motions, and vice versa. The weighting factors are listed below:



**Figure 30. Logic tree for calculating a single 5% damped surface response spectrum from the 24 runs corresponding to 600 surface response spectra analyses for the example bridge site in Blytheville, Arkansas.**

- Gosnell 0.35
  - Log-Normal Median 0.25
  - LNM + 20% 0.25
  - LNM - 20% 0.15
- }  $\Sigma = 1.0$

Two methods of analysis were used in this study, NL and EQL. The EQL formulation is essentially a simplified way to efficiently approximate the NL formulation; however, the two types of analysis can diverge when “large” strains are generated. NL analyses are widely favored for analyses that involve large shear strains, which is the case for this study and virtually any other site response analysis that will be conducted in Northeast Arkansas. While NL analyses are preferred for large-strain analyses, the EQL method is well-proven and often used in conjunction with more advanced methods to flush out potential errors. Both methods of analysis are considered for this study, and weighted as follows:

- NL 0.7
  - EQL 0.3
- }  $\Sigma = 1.0$

## 6. EQUIVALENT LINEAR SITE RESPONSE ANALYSES

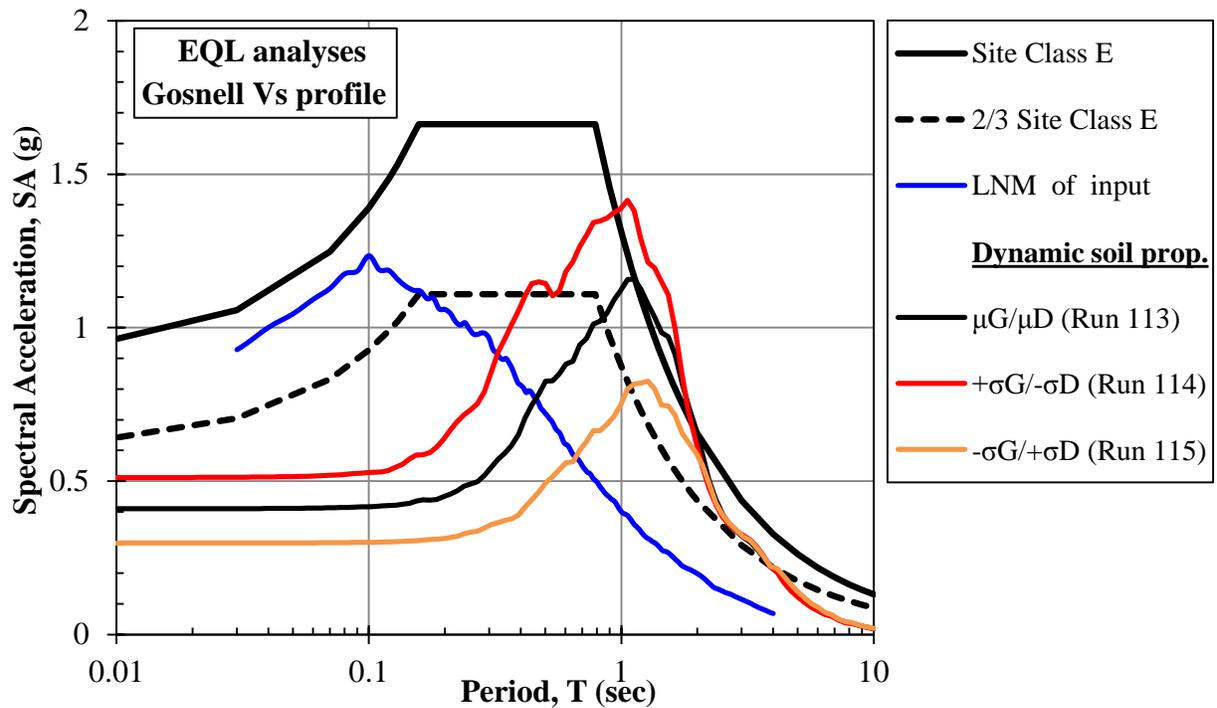
EQL total stress analysis is a simplified one-dimensional model used to predict earthquake motions at the ground surface. This type of analysis predicts the frequency-dependent extent to which input “rock” motions are amplified or attenuated as they propagate from bedrock up to the ground surface, based on local soil properties (layering, stiffness and damping). This section presents results from the 12 EQL runs and assesses the relative influence of each input parameter (dynamic soil properties, shear wave velocity and depth to bedrock) to the predicted surface response. Figures in this chapter include a text box to summarize the details of the runs being presented. In addition, each site response figure includes the Site Class E and 2/3 Site Class E AASHTO code-based spectra.

### 6.1. INFLUENCE OF DYNAMIC SOIL PROPERTIES

The effects of varying dynamic soil properties for a fixed  $V_s$  profile are presented in Figure 31. Each LNM output response spectrum represents a different set of dynamic soil properties used with the Gosnell  $V_s$  profile. The dynamic properties used include the mean modulus and mean damping curves ( $\mu_G/\mu_D$ ), the plus one standard deviation modulus and minus one standard deviation damping curves ( $+\sigma_G/-\sigma_D$ ), and the minus one standard deviation modulus and plus one standard deviation damping curves ( $-\sigma_G/+\sigma_D$ ). The dynamic soil properties have all been adjusted to account for implied shear strength over the top 100 feet of the profile, as discussed in Section 3.3.

The stiffest (most linear) modulus curves and lowest damping curves (i.e.,  $+\sigma_G/-\sigma_D$ , Run 114) result in the largest surface ground motions at all periods less than 2 seconds, while the softest/most nonlinear modulus curves and highest damping curves (i.e.,  $-\sigma_G/+\sigma_D$ , Run 115) result in the smallest surface ground motions. This trend is expected, as relatively stiffer soils transmit more short-period seismic energy to the surface. The surface response is unaffected by the choice of dynamic soil properties at periods greater than 2 seconds. The surface response spectrum resulting from the  $+\sigma_G/-\sigma_D$  (Run 114) properties begins to exhibit ground motion amplification relative to the input “rock” motion at a period of approximately 0.3 seconds. Amplification for the  $-\sigma_G/+\sigma_D$  (Run 115) properties does not occur until a period of approximately 0.7 seconds. Between the periods of 0.1 - 1.0 seconds, the differences in spectral acceleration between the LNM output response spectra of the  $+\sigma_G/-\sigma_D$  (Run 114) soil properties and the  $-\sigma_G/+\sigma_D$  (Run 115) properties range from 43% - 64%, being most different at 0.4 seconds.

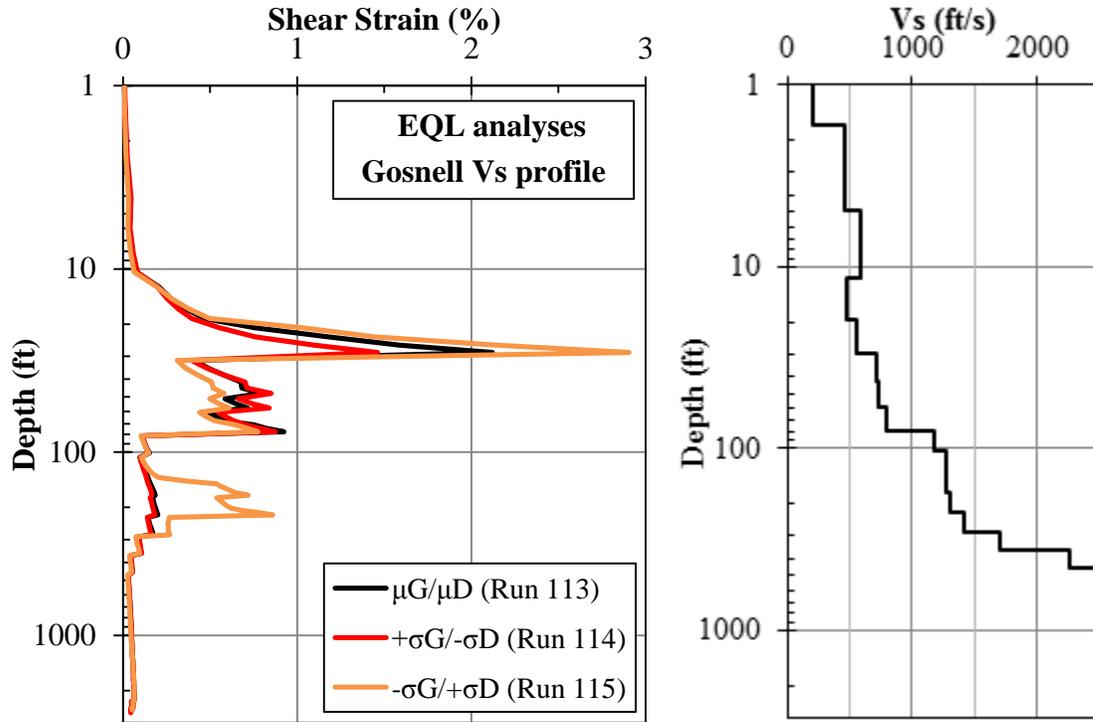
The LNM surface response spectrum for  $+\sigma_G/-\sigma_D$  (Run 114) exceeds the 2/3 AASHTO Site Class E spectrum at nearly all periods greater than 0.5 seconds, while the  $\mu_G/\mu_D$  (Run 113) and  $-\sigma_G/+\sigma_D$  (Run 115) LNM surface response spectra do not exceed the 2/3 Site Class E spectrum until periods of approximately 0.9 and 1.0 seconds, respectively. All of the output surface response spectra exhibit a shift towards greater predominant periods than those approximated by the Site Class E spectrum. This trend is expected as very deep soil sites tend to amplify long-period (low frequency) energy.



**Figure 31. Acceleration response spectra from equivalent linear site response analyses with fixed  $V_s$  profile and varying dynamic soil properties for the example bridge site in Blytheville, AR.**

The shear strain profiles predicted from different dynamic soil properties and a single  $V_s$  profile (Gosnell) are presented in Figure 32. The largest shear strain is computed at a depth of 28 feet for all three dynamic soil property assignments. This depth marks the boundary between soft near-surface clay and stiffer underlying sand. The corresponding impedance ratio at this boundary is a trademark of the profile, consistently resulting in large strains in the soft clay. The largest shear strains are predicted for the  $-\sigma_G/+ \sigma_D$  (Run 115), which also resulted in the lowest surface spectral accelerations, as discussed above (refer to Figure 31). This leads to an important, general observation: lower spectral accelerations at the ground surface are generally accompanied by higher shear strains below the surface.

The shear strain profiles all follow a similar trend, except at depths between 110 - 290 feet, where shear strains from the  $-\sigma_G/+ \sigma_D$  (Run 115) soil properties are 40% - 75% larger than those from the other soil properties. Preliminary analyses predicted high strains at these depths, but the dynamic soil properties were not adjusted to account for implied shear strengths. Rather, the modulus and damping curves were assigned based on the Darendeli (2001) curves, and manually adjusted as outlined in Section 3.3. If the dynamic soil properties had been adjusted to account for implied shear strengths, the shear strains from the  $-\sigma_G/+ \sigma_D$  (Run 115) soil properties would likely be more closely grouped with those from the other runs.



**Figure 32. Shear strain profiles from equivalent linear site response analyses with fixed  $V_s$  profile and varying dynamic soil properties for the example bridge site in Blytheville, AR.**

## 6.2. INFLUENCE OF $V_s$ PROFILE

Surface response spectra are presented in Figure 33 which correspond to the mean dynamic soil properties ( $\mu G/\mu D$ ) analyzed with the four different  $V_s$  profiles. These include the Gosnell  $V_s$  profile (Run 113), lognormal median (LNM)  $V_s$  profile (Run 116), lognormal median  $V_s$  profile plus twenty percent (Run 119), and lognormal median  $V_s$  profile minus twenty percent (Run 122). The largest spectral accelerations in the short-period range are predicted for the stiffest  $V_s$  profile (i.e., LNM +20%). Likewise, the smallest spectral accelerations are predicted for the softest  $V_s$  profile (i.e., LNM -20%). None of the output spectra exceed the 2/3 Site Class E spectrum at periods less than approximately 0.7 seconds. Between the periods of 0.1 - 1.0 seconds the differences in spectral acceleration between the output spectrum of the LNM + 20%  $V_s$  profile (Run 119) and LNM - 20%  $V_s$  profile (Run 122) range from 42% - 60%, with the largest difference occurring at 0.4 seconds. This is a very similar trend to the range in relative spectral accelerations obtained from fixing the  $V_s$  profile and varying the dynamic soil properties (refer to Figure 31). This indicates that accounting for uncertainty in the dynamic soil properties is just as important as accounting for uncertainty in the  $V_s$  profile.

The shear strain profiles corresponding to the varying  $V_s$  profiles are presented in Figure 34. The greatest predicted shear strain occurs at a depth of 42 feet for the LNM - 20%  $V_s$  profile (Run 122), which is the softest profile. In general, the shear strains at depths between 30 - 80 feet are 25% - 60% greater for the LNM - 20%  $V_s$  profile than the other  $V_s$  profiles. The  $V_s$

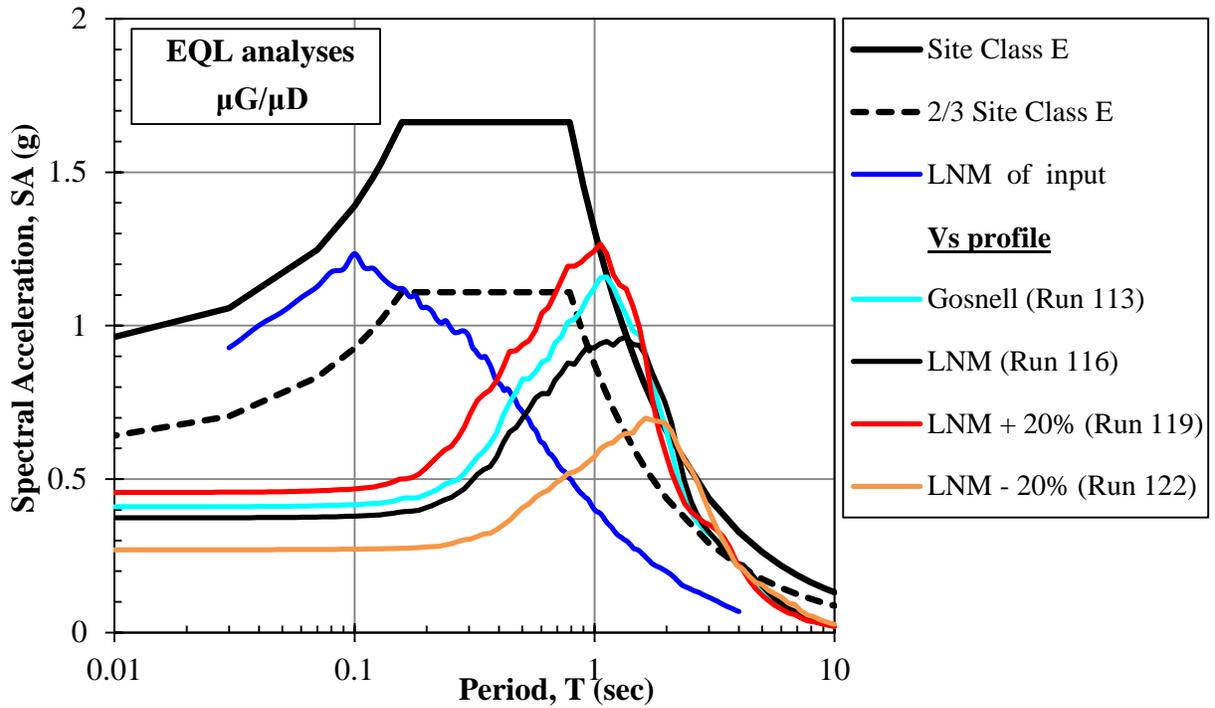


Figure 33. Acceleration response spectra from equivalent linear site response analyses with varying  $V_s$  profiles and fixed dynamic soil properties for the example bridge site in Blytheville, AR.

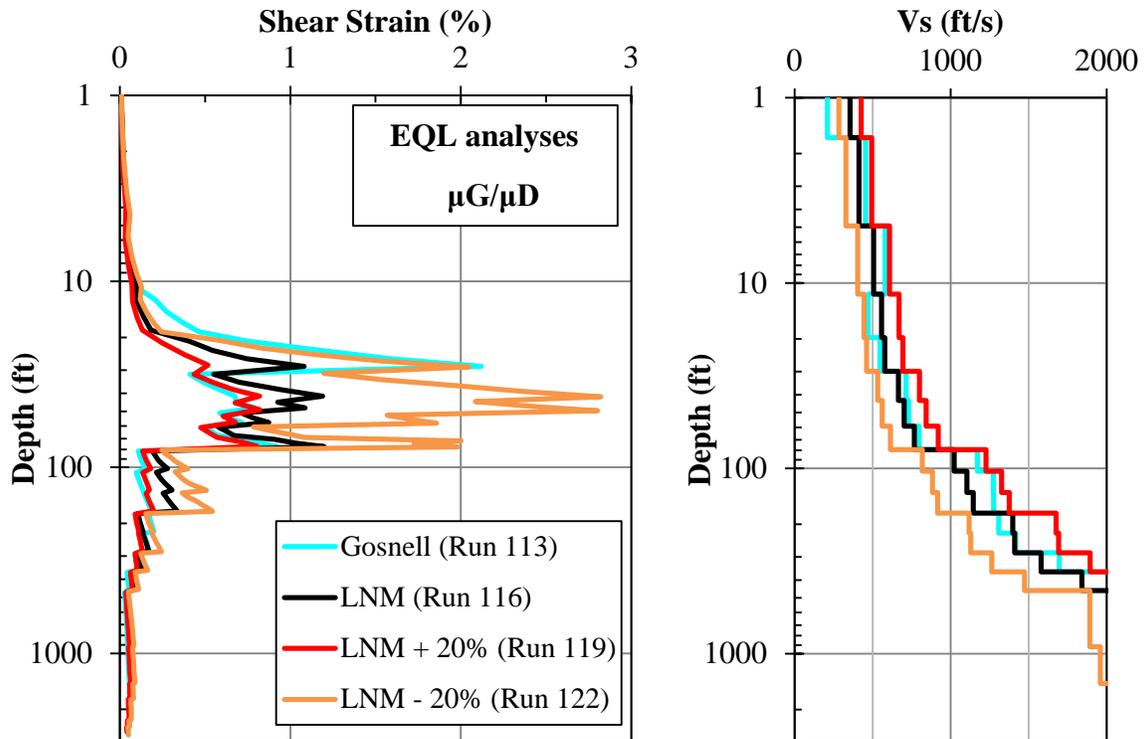


Figure 34. Shear strain profiles from equivalent linear site response analyses with varying  $V_s$  profiles and fixed dynamic soil properties for the example bridge site in Blytheville, AR.

profiles are included within Figure 34 on the same depth scale so the reader can see the direct relationship between the  $V_s$  profiles and the shear strain profiles. Note that the significant reduction in shear strains at a depth of approximately 80 feet is driven by a marked increase in  $V_s$ .

### 6.3. SENSITIVITY OF SURFACE RESPONSE SPECTRUM TO DEPTH TO BEDROCK

With soil profiles extending thousands of feet deep, it can be very difficult, if not impossible, to measure precise depths to bedrock for bridge sites in the Mississippi Embayment. Therefore, a sensitivity analysis is presented in this section to assess the contribution of absolute bedrock depth to the end results of site response analyses for deep soil sites. Figure 35 compares a set of LNM output surface response spectra, with each spectrum representing a different depth to bedrock. The dynamic soil properties and  $V_s$  profile were held constant for each of the analyses using the mean modulus and mean damping curves ( $\mu_G/\mu_D$ ) and the Gosnell  $V_s$  profile. The depth to bedrock was increased and decreased in increments of approximately 330 feet (100 m) for a total of four new soil profiles (two shallower and two deeper). The shallowest profile was assigned a depth to bedrock of 1910 feet (Run 125) and the deepest profile was assigned a depth of 3220 feet (Run 130), as shown on the right-hand side of Figure 36. Note that these changes represent a depth to bedrock of +/- 25% relative to the initial depth of 2570 feet.

The LNM output response spectra are tightly grouped, as shown in Figure 35, with amplification relative to the input “rock” motion beginning at a period of approximately 0.5 seconds. Each of the output spectra exceed the 2/3 Site Class E spectrum at a period of approximately 0.8 seconds and exceed the Site Class E spectrum at periods just beyond 1.0 second. Although the peak response from each output spectrum slightly exceeds the “flat top” of the 2/3 Site Class E spectrum, the output spectra all have predominant periods near 1.0 second. Over the periods of interest for typical bridges constructed by AHTD (i.e., 0.1 - 0.5 seconds), spectral accelerations are well below the 2/3 Site Class E code-based spectrum allowed by AASHTO, and the depth to bedrock does not impact the spectral accelerations over the depth range investigated (i.e., approximately 1900 – 3200 feet).

The shear strain profiles predicted from the depth to bedrock sensitivity analyses are presented in Figure 36. The shear strains are nearly identical for all bedrock depths except the shallowest  $V_s$  profile, which has a depth to bedrock of 1910 feet (Run 125). This profile has slightly larger strains than the other profiles at depths between 30 - 80 feet.

### 6.4. REMARKS ON EQL RESULTS

EQL total stress site response analyses have been proven to be a relatively simple means for predicting surface ground shaking (Kramer, 1996). The results presented in this section indicate that: (1) at periods greater than approximately 0.8 seconds surface response spectral accelerations may begin to exceed 2/3 of the AASHTO Site Class E design spectrum, (2) accounting for uncertainties (+/- one standard deviation) in the dynamic soil properties has as much of an effect on the predicted surface response spectra as varying the  $V_s$  profile by +/- 20%,

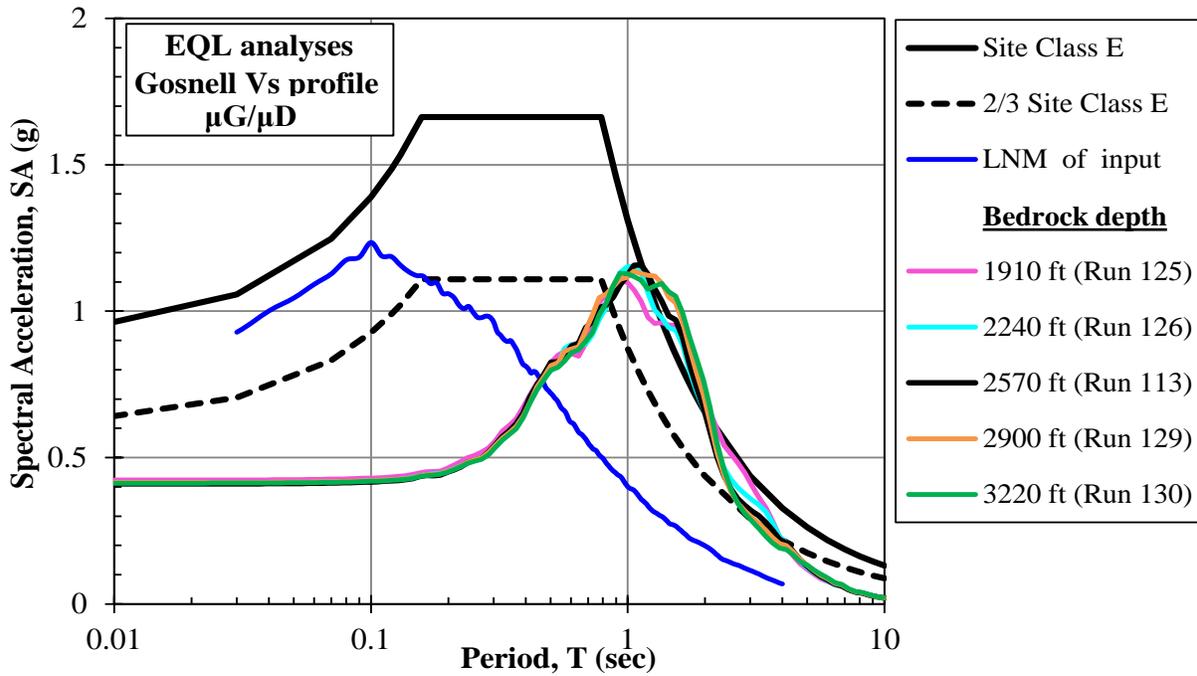


Figure 35. Acceleration response spectra from equivalent linear site response analyses with varying depths to bedrock and fixed dynamic soil properties for the example bridge site in Blytheville, AR.

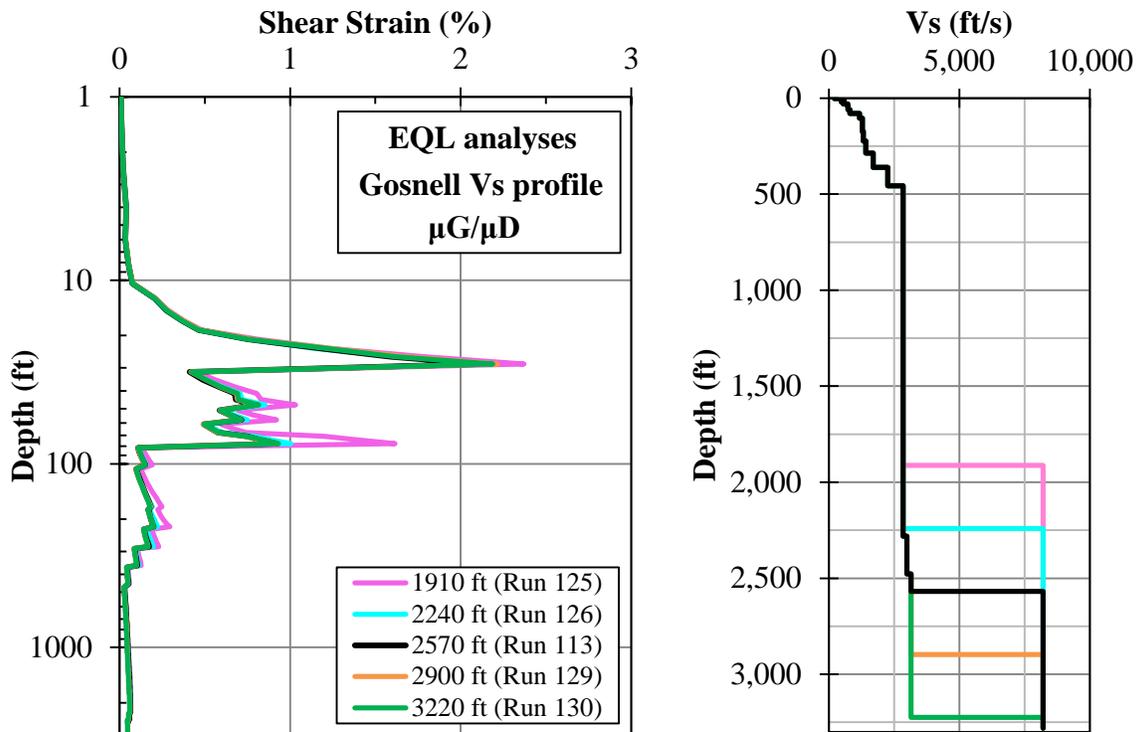


Figure 36. Shear strain profiles from equivalent linear site response analyses with varying depths to bedrock and fixed dynamic soil properties.  $V_s$  profile (not on a logarithmic scale) for the example bridge site in Blytheville, AR.

(3) the greatest surface spectral accelerations are expected from the combination of the stiffest  $V_s$  profile (i.e., +20% LNM) and the most linear modulus curves and lowest damping curves (i.e.,  $+\sigma_G/-\sigma_D$ ), while the greatest shear strains are expected from the combination of the softest  $V_s$  profile (i.e., -20% LNM) and the most nonlinear modulus curves and highest damping curves (i.e.,  $-\sigma_G/+\sigma_D$ ), and (4) for deep soil sites within the ME subjected to large input ground motions, the absolute depth to bedrock will have little impact on the site response results when varied within the range of approximately 1900 – 3200 feet below the surface. Regarding (3) above, the weighting factors discussed in Section 5.2 (refer to Figure 30) were chosen specifically to yield more “conservative” results in terms of estimated surface spectral accelerations than in terms of predicted soil shear strains. Regarding (4) above, the selection of the correct depth to bedrock will have virtually no effect on the estimated surface response spectra at periods less than 1.0 seconds, and only minor effects at greater periods, provided the depth to bedrock falls within the range noted.

## 7. FULLY NON-LINEAR SITE RESPONSE ANALYSES

Fully NL total stress analysis results are presented in this section. Hashash et al. (2010) concluded that NL time domain analyses may be necessary in order to model the dynamic soil properties for cases of high seismic intensity at the rock base or high strain levels within the soil column where EQL analyses cannot represent the dynamic soil properties over the entire duration of the seismic event. A total of 300 surface acceleration response spectra have been computed using the NL formulation. NL analyses were necessitated by a large input “rock” motions associated with extreme seismic hazard in the NMSZ. As a result, large shear strains were observed in all of the preliminary analyses, and also in the design-level EQL analyses.

### 7.1. INFLUENCE OF DYNAMIC SOIL PROPERTIES

The effects of varying dynamic soil properties with a fixed  $V_s$  profile are presented in Figure 37. Each LNM surface response spectrum represents the output from a different set of dynamic soil properties used with the Gosnell  $V_s$  profile. The most linear modulus curves and lowest damping curves (i.e.,  $+\sigma_G/-\sigma_D$ , Run 102) result in the highest surface ground motions at all periods less than 2 seconds, while the most nonlinear modulus curves and highest damping curves (i.e.,  $-\sigma_G/+\sigma_D$ , Run 103) result in the lowest ground motions. As noted in Section 6, this is expected because relatively stiff (linear) soils transmit more high-frequency seismic energy up to the ground surface. The surface response is unaffected by the choice of dynamic soil properties at periods greater than 2 seconds. The LNM surface response spectrum computed using the  $+\sigma_G/-\sigma_D$  (Run 102) properties begins to exhibit ground motion amplification relative to the input “rock” motion at a period of approximately 0.4 seconds, while amplification for the  $-\sigma_G/+\sigma_D$  (Run 103) properties does not occur until a period of approximately 0.8 seconds.

Between the periods of 0.1 - 1.0 seconds, the differences in spectral acceleration between the LNM surface response spectra computed using the  $+\sigma_G/-\sigma_D$  (Run 102) and  $-\sigma_G/+\sigma_D$  (Run 103) soil properties range from 47% - 67%, with maximum difference at 0.1 seconds. The  $+\sigma_G/-\sigma_D$  (Run 102) surface response spectrum exceeds the 2/3 AASHTO Site Class E spectrum at periods between 0.03 and 0.06 seconds and nearly all periods greater than 0.9 seconds. The  $\mu_G/\mu_D$  (Run 101) surface response spectrum does not exceed the 2/3 Site Class E spectrum at periods below 1.0 second, and the  $-\sigma_G/+\sigma_D$  (Run 103) spectrum does not exceed the 2/3 Site Class E spectrum at any of the periods investigated in the analyses. All of the output response spectra exhibit a shift towards greater predominant periods than those approximated by the Site Class E spectrum. This is expected as very deep soil sites tend to amplify long-period (low frequency) energy.

The shear strain profiles resulting from a fixed  $V_s$  profile (i.e., the Gosnell profile) and different dynamic soil properties are presented in Figure 38. A large shear strain occurs at a depth of 28 feet for all properties. As discussed in Section 6.1, this depth marks the transition between soft clay and relatively stiff sand.

The largest shear strain is predicted at a depth of 77 feet for the profile corresponding to the most linear dynamic properties,  $+\sigma_G/-\sigma_D$  (Run 102). These large shear strains are a result of

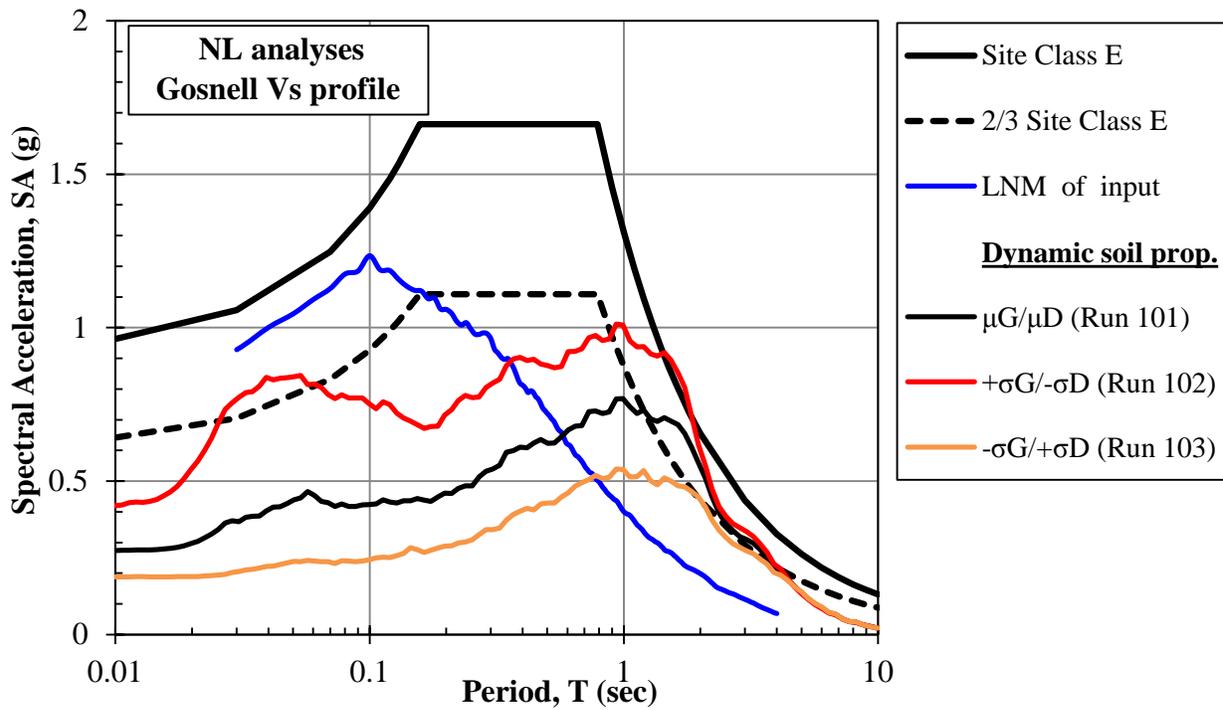


Figure 37. Acceleration response spectra from non-linear site response analyses with fixed  $V_s$  profile and varying dynamic soil properties for the example bridge site in Blytheville, AR.

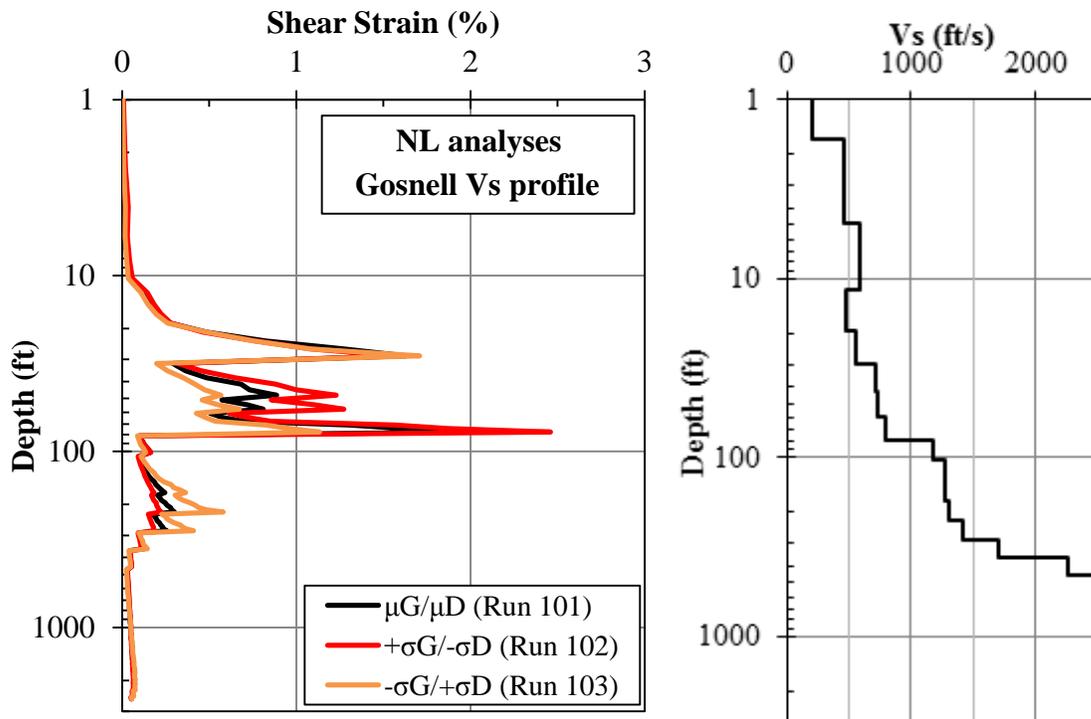


Figure 38. Shear strain profiles from non-linear site response analyses with fixed  $V_s$  profile and varying dynamic soil properties for the example bridge site in Blytheville, AR.

a large  $V_s$  contrast in two adjacent sand layers (800 fps vs. 1170 fps, see Figure 38). While all three profiles exhibit a peak in shear strains at this depth, the shear strain magnitude is largest for the profile corresponding to most linear dynamic soil properties,  $+\sigma_G/-\sigma_D$  (Run 102). The smallest peak shear strain at 77 feet is predicted for the most nonlinear soil profile,  $-\sigma_G/+\sigma_D$  (Run 103). This indicates that nonlinearity facilitates more attenuation of seismic energy in the soil below 77 feet, which is observed by the larger strains at depths between 110 – 290 feet in Figure 38. Thus, unlike the EQL analyses, the largest ground surface spectral accelerations are not always associated with the smallest shear strains below the ground surface. It is important to note that dynamic soil properties were adjusted to account for implied shear strengths in the top 100 feet, but not from 110 – 290 feet (see Section 3.3). If the dynamic soil properties had been adjusted at depths both depth ranges, the peaks in shear strain at 77 feet would likely exhibit somewhat different behavior.

## 7.2. INFLUENCE OF $V_s$ PROFILE

Surface response spectra are presented in Figure 39 which correspond to the mean dynamic soil properties ( $\mu_G/\mu_D$ ) analyzed with the four different  $V_s$  profiles. These include the Gosnell  $V_s$  profile (Run 113), LNM  $V_s$  profile (Run 116), LNM  $V_s$  profile plus twenty percent (Run 119), and LNM  $V_s$  profile minus twenty percent (Run 122). The largest spectral accelerations in the short-period range are predicted for the stiffest  $V_s$  profile (i.e., LNM +20%). Likewise, the smallest spectral accelerations are predicted for the softest  $V_s$  profile (i.e., LNM - 20%). None of the output spectra exceed the 2/3 Site Class E spectrum until periods greater than approximately 1.0 seconds. Between the periods of 0.1 - 1.0 seconds the differences in spectral acceleration between the output surface spectra corresponding to the LNM + 20%  $V_s$  profile (Run 107) and LNM - 20%  $V_s$  profile (Run 110) range from 50% - 57%, with a maximum difference at a period near 0.4 seconds. This range in relative spectral accelerations is less than that obtained from fixing the  $V_s$  profile and varying the dynamic soil properties (refer to Figure 37). This indicates that accounting for uncertainty in the dynamic soil properties is just as important as accounting for uncertainty in the  $V_s$  profile.

Shear strain profiles corresponding to the varying  $V_s$  profiles are presented in Figure 40. The largest predicted shear strain of 3% occurs at a depth of 77 feet in the LNM - 20% (Run 110), which is the softest profile. The impedance ratio at this depth has already been discussed in Section 7.1 with respect to the effect of variations in dynamic soil properties. This same impedance ratio, now with respect to variations in  $V_s$  profiles, yields the largest shear strains in the softest soil profile (Run 110). Likewise, the smallest shear strains are predicted for the stiffest soil profile (Run 107). The shear strains at depths between 30 - 80 feet range from very similar to 700% different from one another, with the largest shear strains predicted in the softest soil profile (Run 110). At depths between 100 – 200 feet, the largest strains again occur in the softest profile (Run 110), but unlike Figure 38 this profile also has the largest strains at a depth of 77 feet.

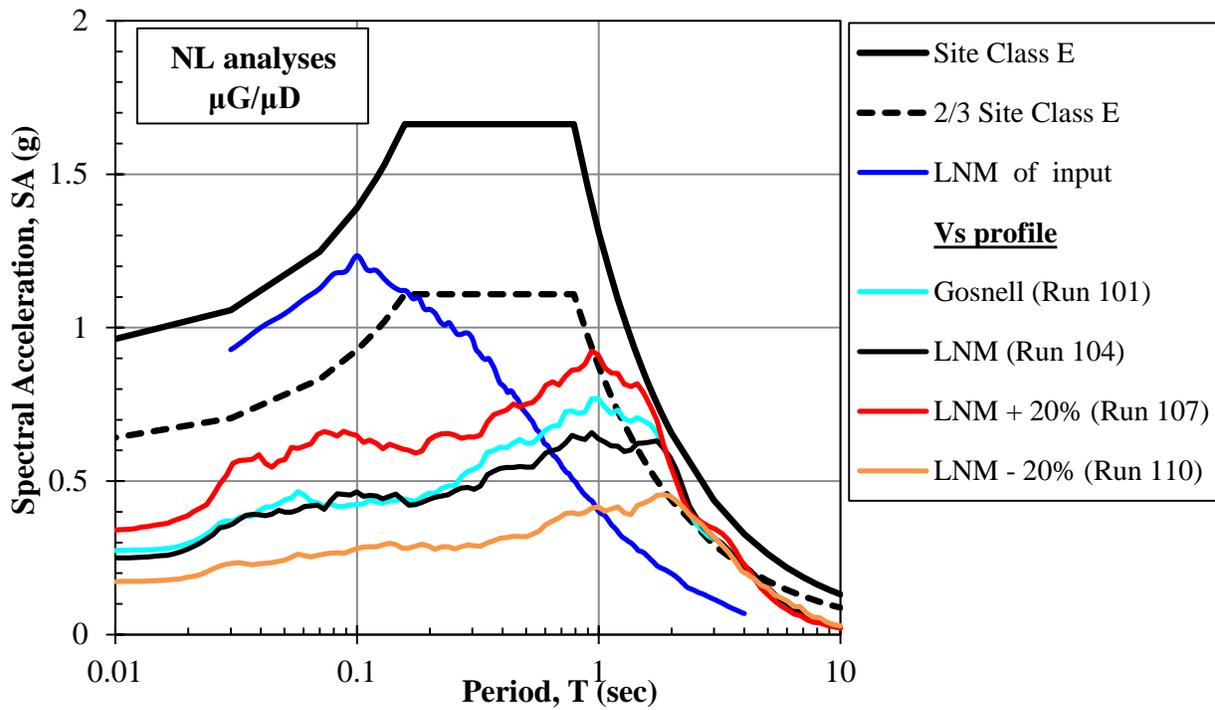


Figure 39. Acceleration response spectra from non-linear site response analyses with varying  $V_s$  profiles and fixed dynamic soil properties for the example bridge site in Blytheville, AR.

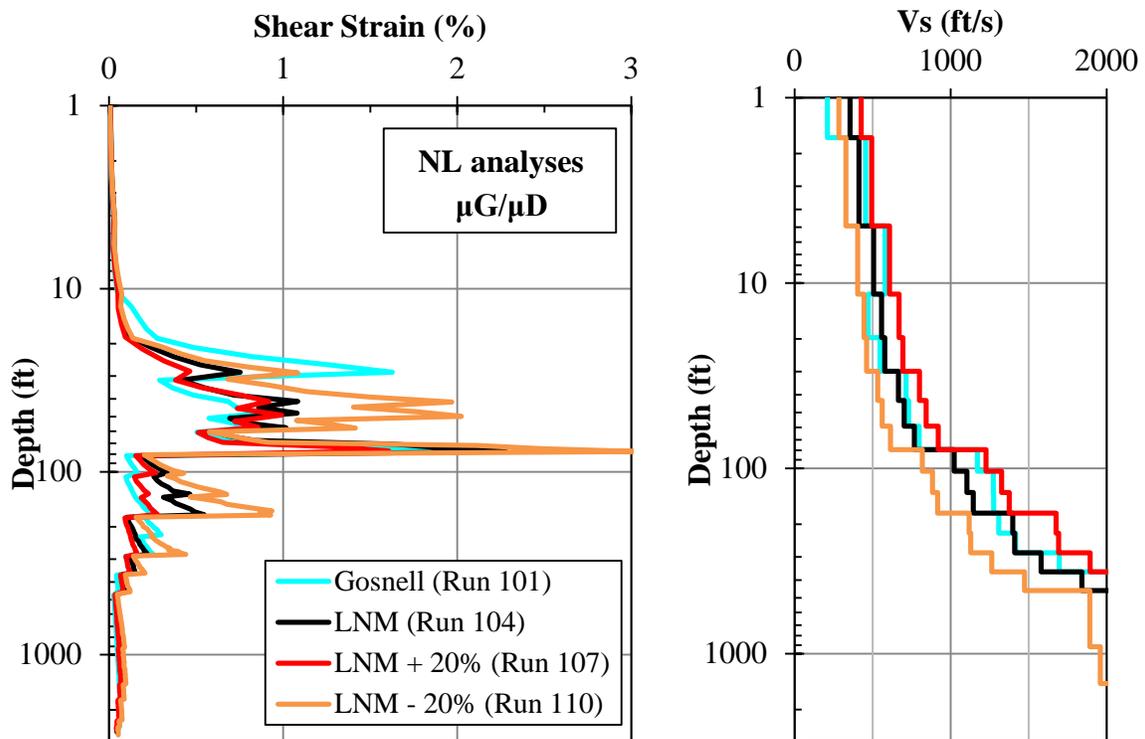


Figure 40. Shear strain profiles from non-linear site response analyses with varying  $V_s$  profiles and fixed dynamic soil properties for the example bridge site in Blytheville, AR.

### 7.3. SENSITIVITY OF SURFACE RESPONSE SPECTRUM TO DEPTH TO BEDROCK

Similar to Section 6.3, a sensitivity analysis is now presented to assess the contribution of absolute bedrock depth to the surface acceleration predicted by NL site response analyses for deep soil sites. The dynamic soil properties and  $V_s$  profile were held constant for each of the analyses using the mean modulus and mean damping curves ( $\mu_G/\mu_D$ ) and the Gosnell  $V_s$  profile. The depth to bedrock was increased and decreased in increments of approximately 330 feet (100 m) for a total of four new soil profiles (two shallower and two deeper). The shallowest profile was assigned a depth to bedrock of 1910 feet (Run 125) and the deepest profile was assigned a depth of 3220 feet (Run 130), as shown on the right-hand side of Figure 41.

The LNM output response spectra are tightly grouped, as shown in Figure 41, with amplification relative to the input “rock” motion beginning at a period of approximately 0.6 seconds. Each of the output spectra exceed the 2/3 Site Class E spectrum at a period greater than 1.0 seconds and do not exceed the Site Class E spectrum at any of the periods investigated in this study. The output spectra all have predominant periods near 1.0 second and do not exceed the “flat top” of the 2/3 Site Class E spectrum. Over the periods of interest for typical bridges constructed by AHTD (i.e., 0.1 - 0.5 seconds) spectral accelerations are well below the 2/3 Site Class E code-based spectrum allowed by AASHTO and the depth to bedrock does not impact the spectral accelerations over the depth range investigated (i.e., approximately 1900 – 3200 feet).

The shear strain profiles predicted from the depth to bedrock sensitivity analyses are presented in Figure 42. The shear strains are nearly identical for all bedrock depths, except at depths where shear strains peak, in which cases the shallower profiles exhibit slightly larger shear strains.

### 7.4. REMARKS ON NL RESULTS

The results from NL analyses indicate that: (1) at periods between 0.1 and 0.9 seconds the surface response spectral accelerations do not exceed 2/3 of the AASHTO Site Class E design spectrum; (2) accounting for uncertainties (+/- one standard deviation) in the dynamic soil properties has a similar effect on the predicted surface response spectra as compared to varying the  $V_s$  profile by +/- 20%; (3) the greatest surface spectral accelerations are expected from the combination of the stiffest  $V_s$  profile (i.e., +20% LNM) and the most linear modulus curves and lowest damping curves (i.e.,  $+\sigma_G/-\sigma_D$ ), but do not necessarily correspond to the smallest shear strains throughout the soil profile; and (4) for deep soil sites within the ME subjected to large input ground motions, the absolute depth to bedrock will have little impact on the site response results when varied within the range of approximately 1900 – 3200 feet below the surface. Regarding (3) above, the weighting factors discussed in Section 5.2 (refer to Figure 30) were chosen specifically to yield more “conservative” results in terms of estimated surface spectral accelerations than in terms of predicted soil shear strains. Regarding (4) above, the selection of the correct depth to bedrock will have virtually no effect on the estimated surface response spectra at periods less than 1.0 seconds, and only minor effects at greater periods, provided the depth to bedrock falls within the range noted.

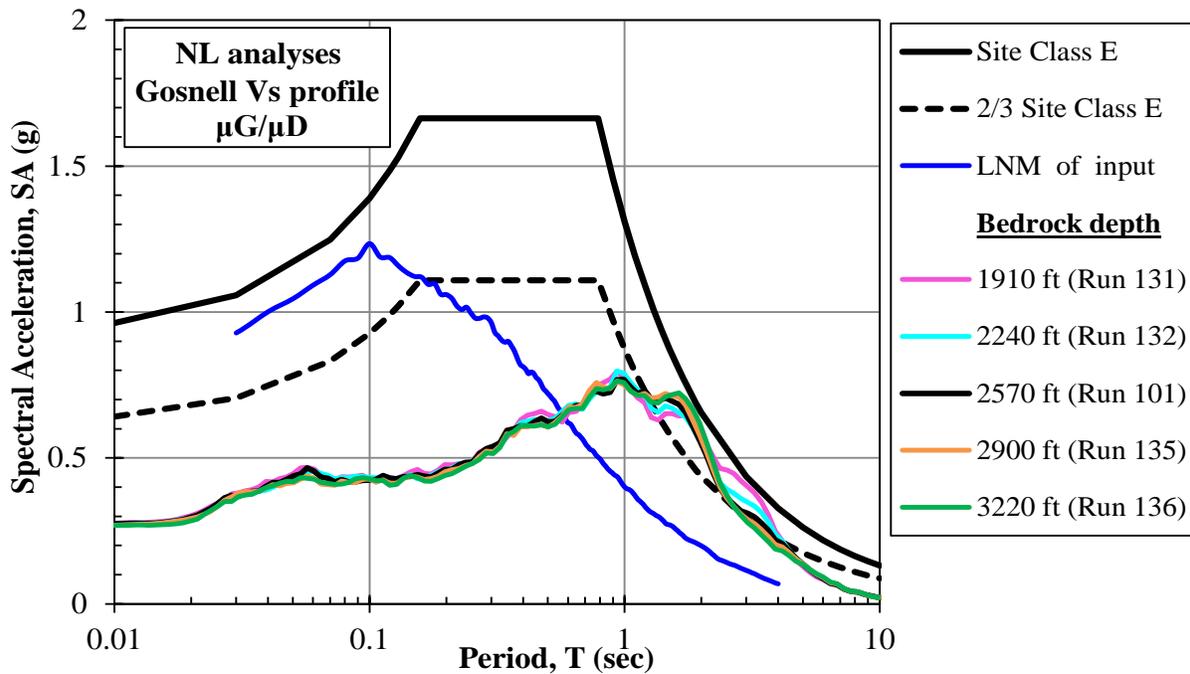


Figure 41. Acceleration response spectra from non-linear site response analyses with different depths to bedrock and fixed dynamic soil properties for the example bridge site in Blytheville, AR.

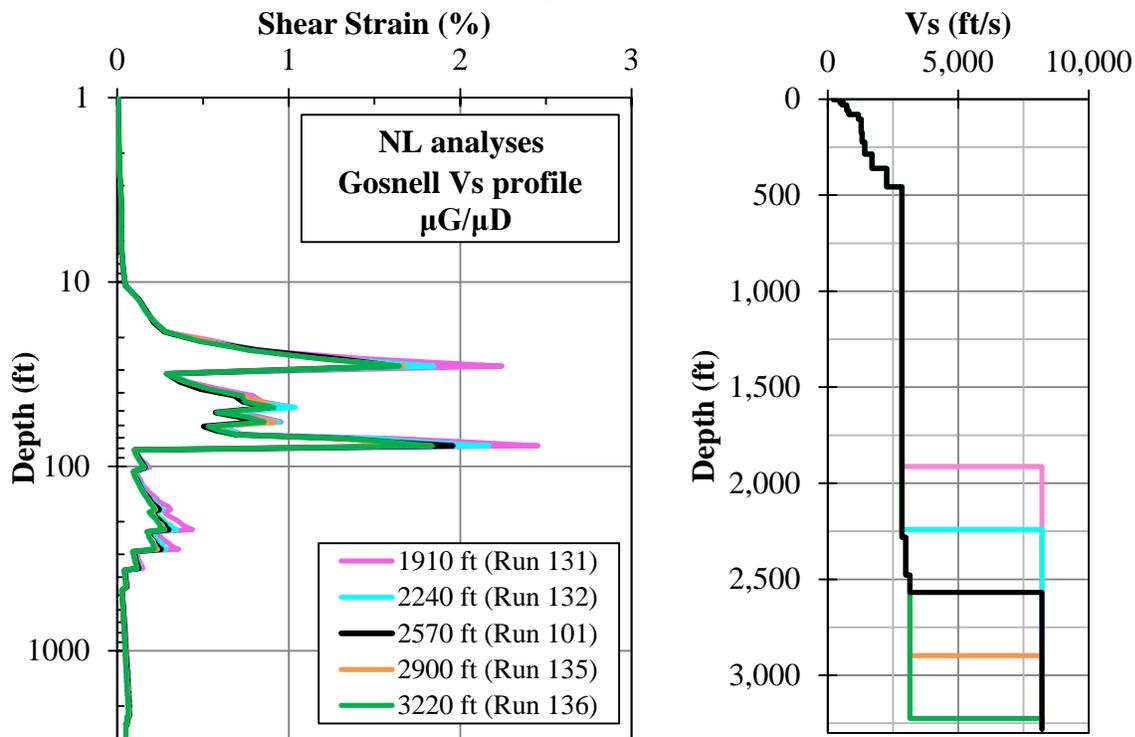


Figure 42. Shear strain profiles from non-linear site response analyses with varying depths to bedrock and fixed dynamic soil properties (note,  $V_s$  profile is not on a logarithmic scale) for the example bridge site in Blytheville, AR.

## 8. DEVELOPMENT OF DESIGN RESPONSE SPECTRUM AT GROUND SURFACE

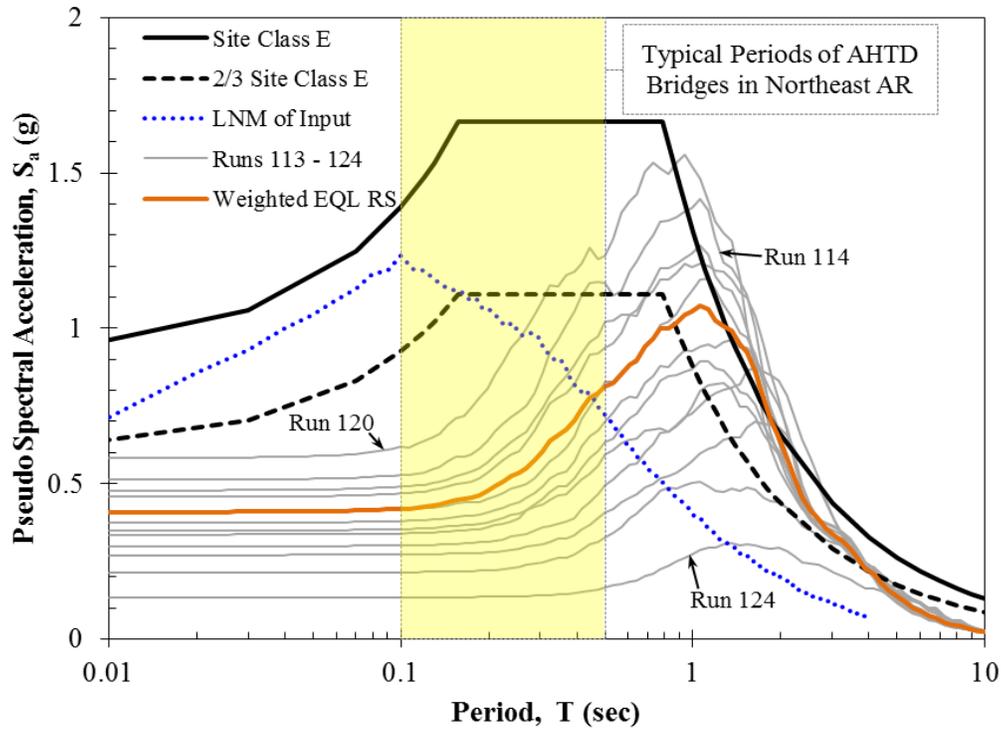
### 8.1. COMPUTATION OF THE DESIGN RESPONSE SPECTRUM

Sections 5 – 7 describe a total of 1,300 site response analyses that were conducted throughout this study. These include preliminary analyses, sensitivity analyses and a total of 600 design-level analyses. Each analysis comprises a distinct combination of input “rock” ground motion, dynamic soil properties,  $V_s$  profile and analysis technique (i.e., EQL or NL). Section 5.2 describes the method by which results from the 600 design-level analyses were categorized and weighted to construct a single, site-specific surface response spectrum. The merging process implements conservatism on the basis of engineering judgment (refer to Section 1), consistent with the Level 1 site response approach described in McGuire (2001) and NUREG (2002).

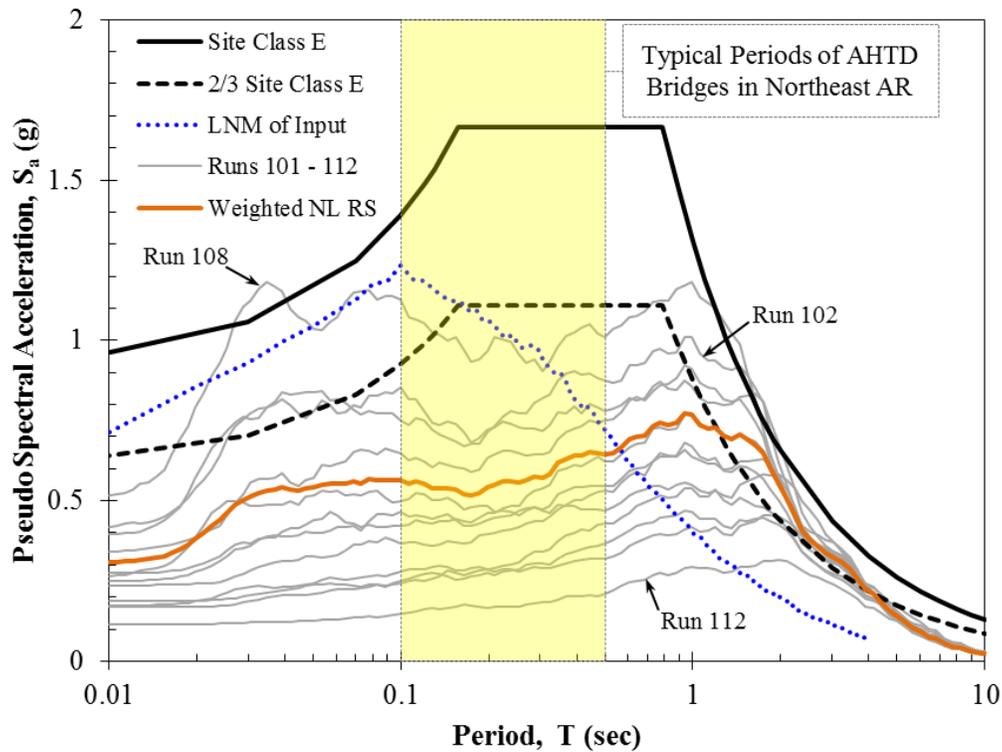
Results from the design-level EQL and NL analyses are presented in Figures 43 and 44, respectively. Each figure includes 12 gray lines to summarize 300 distinct site response analyses. The gray lines each represent the LNM surface response spectrum from the 25 different input “rock” motions, each analyzed with the same combination of dynamic soil properties,  $V_s$  profile and analysis technique. The dotted blue line represents the LNM of the 25 input “rock” motions, and the bolded orange line is the weighted average surface response spectrum from the 300 distinct analyses. The fully weighted site-specific surface response spectrum, representing all 600 design-level analyses (300 EQL, 300 NL), is shown in Figure 45.

AASHTO Guide Specifications for Seismic Bridge Design describe a procedure for developing a site-specific response spectrum for design, based on the results from site-specific ground motion response analyses. First, values of the site-specific surface response spectrum (Figure 45) are divided, period-by-period, by the input “rock” motion. In this study, the divisor is represented by the lognormal median of the 25 input “rock” motions (i.e., the blue dotted line in Figures 43 – 45). The result is a series of period-dependent spectral ratios. Spectral ratios represent the amount by which the site-specific soil column is expected to amplify or attenuate horizontal ground motions as they propagate from bedrock up to the ground surface. Simply put, the period-dependent spectral ratios from a site-specific ground motion response analysis replace the generic, code based amplification factors  $F_{PGA}$ ,  $F_a$  and  $F_s$ . The site-specific spectral ratios from this study are presented in Figure 46.

In order to maintain some level of governance on the results from site-specific analyses, AASHTO requires that the site-specific response spectrum for design be calculated using a generic, code-based response spectrum to represent the input “rock” motion. In Northeast Arkansas, the deep Paleozoic bedrock is Site Class A. Therefore, the site-specific response spectrum for design (hereafter referred to as the “design spectrum”) is calculated by multiplying the AASHTO Site Class A response spectrum (shown in Figure 2), period-by-period, by the spectral ratios from the site-specific analyses. Furthermore, AASHTO strongly recommends a lower bound on the design spectrum equal to 2/3 of the generic, code-based response spectrum for the simplified site class. For the example site in Blytheville, AR, which is classified as Site Class E, the lower bound is 2/3 of the Site Class E response spectrum. The delineated design spectrum, which incorporates the recommended lower bound of 2/3 AASHTO Site Class E, is presented in Figure 47.



**Figure 43. Surface response spectra and weighted average from EQL site response analyses for the example bridge site in Blytheville, AR.**



**Figure 44. Surface response spectra and weighted average from NL site response analyses for the example bridge site in Blytheville, AR.**

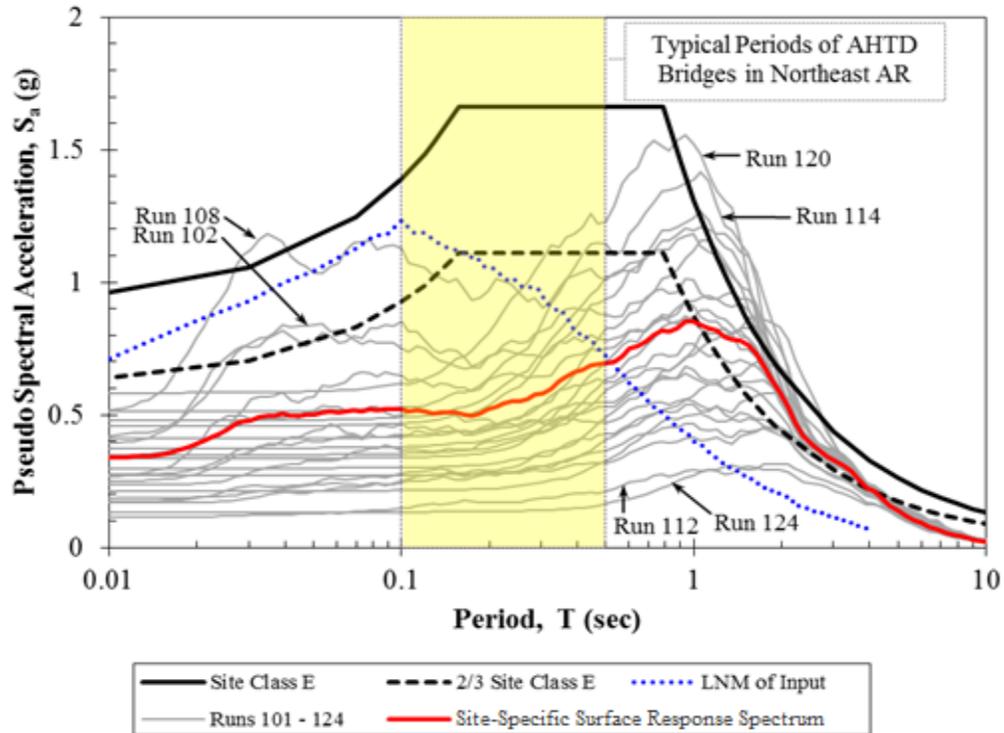


Figure 45. Site-specific surface response spectrum developed from EQL and NL analyses for the example bridge site in Blytheville, AR.

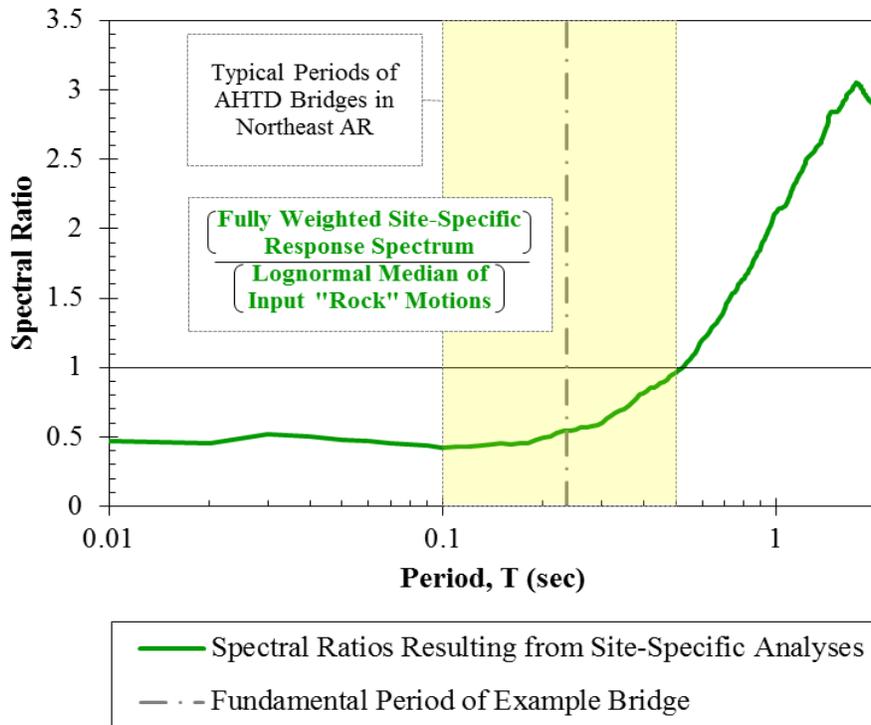
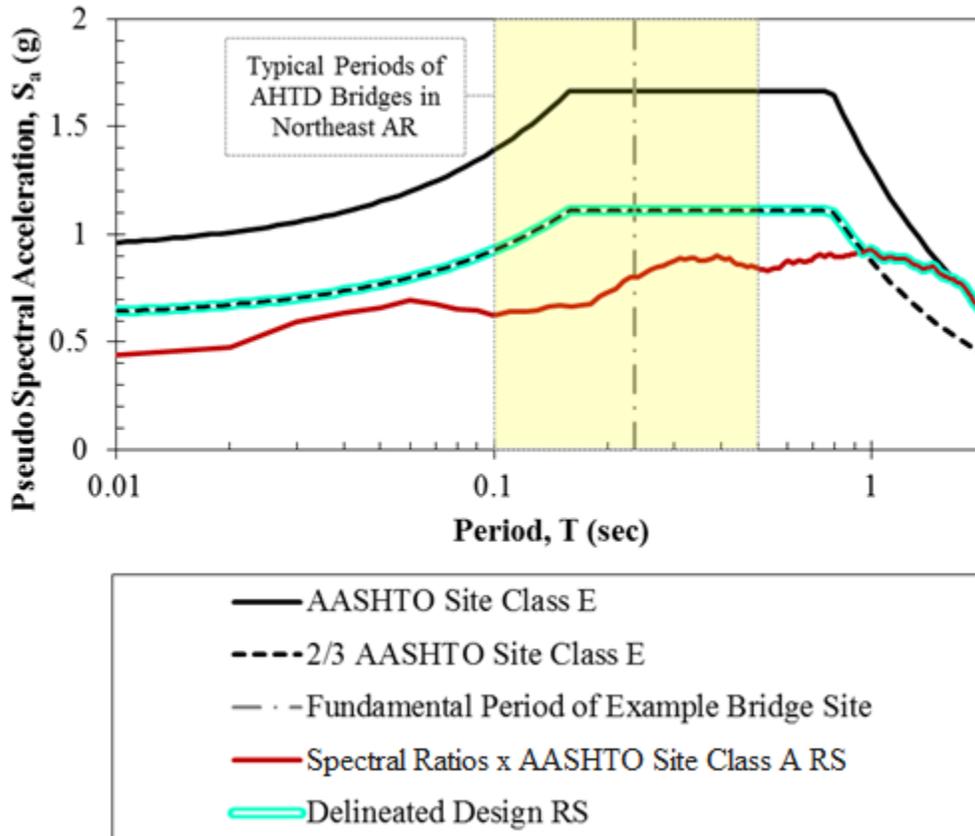


Figure 46. Spectral ratios of surface response to input “rock” motions for the example bridge site in Blytheville, AR.



**Figure 47. Design surface response spectrum from site-specific ground motion response analysis for the example bridge site in Blytheville, AR, delineated along the AASHTO suggested minimum reduction.**

## 8.2. DISCUSSION OF THE DESIGN RESPONSE SPECTRUM

Figures 46 and 47 show that significant attenuation of input “rock” motions is predicted in the short-period range (i.e., < 0.5 seconds), including the fundamental period of the example bridge site in Blytheville, AR. Beyond 1.0 second, however, amplification from the rock to soil surface approaches 300%. At periods beyond approximately 1.0 second, the delineated design spectrum exceeds the 2/3 AASHTO Site Class E spectrum, but never exceeds the original AASHTO Site Class E spectrum. If the present study had been conducted for the example bridge in Blytheville prior to design and construction, the spectral acceleration (i.e., seismic force) used in the design could have been reduced from 1.7 g (the code-based Site Class E spectrum) to 1.1 g (2/3 of the code-based Site Class E spectrum). This 33% reduction would have resulted in considerable cost savings for the bridge superstructure, the bridge foundations and the approach embankments. Although site response analyses must be carefully conducted on a site-by-site basis, results from the current study indicate a very real and feasible potential for significant cost savings on design and construction of transportation infrastructure in Northeast Arkansas.

## 9. CONCLUSIONS

### 9.1. DISCUSSION

#### *9.1.1. COLLECTION AND MODIFICATION OF INPUT “ROCK” GROUND MOTIONS*

The governing earthquake scenario in the New Madrid Seismic Zone is based on earthquakes for which no physical measurements of ground motion are available. Site response analyses require input “rock” ground motions that correspond to the governing earthquake scenario(s). In the absence of physical records from the region of interest, input motions may be collected by various methods, which are described in Chapter 4.

No clear consensus exists regarding the “best” method of obtaining input ground motions for site response studies in the New Madrid Seismic Zone. In the current study, 25 input “rock” motions were chosen from the ground motion records modified by McGuire, et al. (2001). Each of the ground motions was then spectrally matched to a target spectrum using the computer program RspMatch. Particularly important with the Level 1 approach to seismic hazard preservation, spectral matching provides a reliable method for generating hazard-consistent input “rock” motions. This study describes an example of the Level 1 approach (McGuire, 2001; NUREG, 2002) to site response, wherein the target spectrum must be defined using the bedrock uniform hazard spectrum (UHS) in order to establish the seismic hazard. The UHS for bedrock at the example bridge site was developed using the USGS Java Ground Motion Parameter Calculator, and used thereafter as the target spectrum. The same approach is recommended for subsequent site response analyses performed by AHTD.

#### *9.1.2. DYNAMIC SOIL PROPERTIES*

Modulus reduction ( $G/G_{\max}-\log[\gamma]$ ) and damping ( $D-\log[\gamma]$ ) curves are required input parameters for site response analyses. As discussed in Sections 2 and 3, the direct measurement of these properties in the NMSZ would require highly advanced sampling and testing techniques. Instead, these properties may be estimated using empirical correlations. The Darendeli (2001) family of modulus reduction and damping curves is deemed most appropriate for Northeast Arkansas because it is formulated to: (1) account for the effects of confining pressure, (2) consider plasticity for clayey soils, and (3) calculate the empirical, strain-dependent standard deviation.

Designers should account for the potential variability of dynamic soil properties for site response analyses in the Mississippi Embayment. This is true for the general case when physical measurements are not included in the design, but particularly so in the ME where the site-specific behavior of deep sediments must be extrapolated from generalized correlations. In the present study, Darendeli (2001) modulus reduction and damping curves were varied one standard deviation above and below the mean. These upper and lower bounds cause differences of over 60% in the output surface response spectrum at certain periods. A remarkably similar sensitivity was observed for a +/- 20% variation in the  $V_s$  profile.

Modulus reduction curves may be used to calculate the implied strength as a function of shear strain. The implied  $\tau$ - $\gamma$  curve often deviates significantly from the anticipated shear strength, as well as the shear strain at which strength is achieved. For layers of a soil profile where site response analyses predict “large” shear strains, the dynamic soil properties must be adjusted to reflect reasonable implied  $\tau$ - $\gamma$  curves. Adjustments to account for implied shear strengths will likely be required for any site response analysis performed for designs in Northeast Arkansas due to large input ground motions and deep soft soil associated with the ME.

### *9.1.3. $V_s$ PROFILES*

In practice, the near-surface  $V_s$  profile should always be measured directly in the field, with upper and lower bounds assigned to account for a reasonable range of potential shear wave velocities. Note that the current study relied on four 200-meter deep  $V_s$  profiles from Northeast Arkansas that were measured previously for an unrelated project (Rosenblad et al., 2010). Section 3 describes how the four measured  $V_s$  profiles were represented in this study by employing the lognormal median (LNM) of the four measured profiles, the LNM +/- 20%, and the measured Gosnell  $V_s$  profile. Although this facilitated a  $V_s$  profile estimate accurate enough for the feasibility study, it is not recommended for routine engineering practice. In general; the stiffer  $V_s$  profiles (i.e., LNM +20%) resulted in the largest surface response spectra, while the softer  $V_s$  profiles (i.e., LNM -20%) resulted in the smallest surface response spectra.

In parts of Northeast Arkansas, it will be impossible to measure the  $V_s$  profile down to bedrock. In such cases, the measured site-specific near-surface profiles should be merged with the full-depth reference profile developed by Romero and Rix (2001). Eventually, testing techniques may be refined to facilitate the direct measurement of deeper  $V_s$  profiles.

### *9.1.4. DEPTH TO BEDROCK*

Soil deposits in Northeast Arkansas are usually thousands of feet deep, making the direct determination of the depth to bedrock quite difficult. The most common method of identifying depths to bedrock in this region is by use of structural contour maps developed from well logs, deep boreholes and geophysical tests. The precision of estimates made in this manner is certainly not perfect. However, sensitivity analyses in the current study show that, as long as the depth to bedrock is approximately between 1900 and 3200 ft, the site response results are almost identical at periods less than two seconds. In fact, when all other factors are equal, any depth to bedrock within this range will produce essentially the same short-period surface response spectrum.

### *9.1.5. SITE RESPONSE ANALYSES: NL VS. EQL*

In regions like Northeast Arkansas where site-specific ground motion response analyses predict “large” shear strains, it is prudent to consider the results from both equivalent linear (EQL) and non-linear (NL) analyses. As described in the literature review, NL analyses are

more appropriate for large shear strains, but the results of EQL analyses also provide valuable information about the ground motion response. In general, at periods less than 0.5 seconds the NL analyses resulted in larger spectral accelerations than the EQL analyses, while at periods greater than 0.5 seconds the opposite trend is manifest (i.e., EQL analyses resulted in larger spectral accelerations than the NL analyses). Both NL and EQL analyses were conducted for this study with greater weight assigned to the results from NL analyses.

In Eastern Arkansas, thick sandy deposits underlay deposits of surface clay. These sandy soils have a high likelihood of liquefying during a seismic event. In order to properly predict the surface response, these soils should be modeled using effective stress non-linear analyses. Future work in this area should include effective stress non-linear soil models.

## 9.2. CONCLUSIONS

This study aims to demonstrate the feasibility of conducting site-specific ground motion response analyses for the seismic design of transportation infrastructure in Northeast Arkansas as a means to reduce short-period design ground motions. Located in the New Madrid Seismic Zone, this region is underlain by incredibly thick layers of soft sediments that are subject to significant seismic hazards. Generic, code-based designs are not capable of accounting for this unique geologic setting. In particular, code-based designs cannot account for the anticipated short-period attenuation and long-period amplification of earthquake ground motions. As a result, short-period structures may be over-designed at a significant cost, and long-period structures may be under-designed at a significant risk.

Site-specific ground motion response analyses have been conducted for an example site in Blytheville, AR, where a railroad overpass bridge, previously designed using the generic, code-based procedures documented in AASHTO, was recently constructed. Sections 3 and 4 describe the collection and modification of input parameters to describe the soil profile and seismic hazard, respectively. The parameters have been characterized with upper and lower bounds in addition to best estimates. In all, 1300 distinct site response analyses were conducted to account for the multiple uncertainties, although only 600 analyses contributed to the final delineated design spectrum. Section 5 describes why the different analyses were conducted, and how the results were characterized and weighted to construct a single site-specific surface response spectrum. Chapters 6 and 7 discuss the results from equivalent linear (EQL) and non-linear (NL) site response analyses, respectively. Chapter 8 describes how the weighted site-specific surface response spectrum was used to develop the delineated design spectrum.

Results from this study show that, had the site-specific ground motion response analyses been conducted prior to design, seismic design loads for the example bridge in Blytheville, AR could have been reduced by 33%. The example bridge cost approximately \$11 million. Lowering the design loads by 33% would have led to significant cost savings. Similar results can be expected if site-specific analyses are conducted for new bridges throughout Northeast Arkansas, because (1) the probabilistic seismic hazard is dominated by a single earthquake scenario, and (2) the subsurface characteristics are relatively homogeneous throughout the region.

Most AHTD bridges have fundamental periods in the range of 0.1 – 0.5 seconds. In Northeast Arkansas, this will usually fall into the “short period” range where site-specific analyses can reduce the predicted seismic design loads relative to the generic code-based procedures. For long-period structures, site-specific analyses may predict amplification exceeding the provisions in the AASHTO specifications. These alternative cases, while not cost-saving, are perhaps the more compelling argument for site-specific ground motion response analyses in Northeast Arkansas.

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