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Site Characterization and Site-Specific Seismic Ground Motions Analyses for a Gravelly Site in Wyoming

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16 Abstract

Surface wave testing was performed as part of the sub-surface soil investigation for a bridge replacement project to be constructed within the next five years by the Wyoming Department of Transportation (WYDOT), over the Snake River near Jackson Wyoming. This investigation was performed in order to determine the shear wave velocity structure and seismic site classification for the site. The surface wave testing is part of a larger study to perform a site-specific seismic site response analysis for the site and determine if reductions in design ground motions can be justified. The site has sub-surface conditions consisting of gravels and cobbles (2.5 in. to 10 in. diameter material) that can present challenges for surface wave testing, and other investigation techniques. For both sides of the river active surface wave data was collected and analyzed using the Multi-channel analyses of surface wave (MASW) method. This data was combined with passive data collected using micro-tremor array measurement (MAM) techniques in both an L-array and nested equilateral triangular arrays. The triangular arrays included total side lengths between 20 and 200 meters. Horizontal to Vertical ratio data was also obtained, but was not used due to poor signal quality. The analyses of the collected surface wave data employed a joint inversion procedure using the layering ratio method. The inversion yielded site-specific shear wave velocity profiles. Both abutments classified as seismic soil Site Class D. In total over 448 analyses were performed in order to quantify the uncertainty in the analyses from the ground motions and shear wave velocity profiles. The site-specific analyses justified a decrease in the design spectral accelerations at most frequency/periods when compared with generic code based procedures. The site-specific seismic site response analysis provides more realistic seismic loading scenarios for the site under investigation. The reductions in spectral accelerations could result in significant cost savings for WYDOT.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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LIST OF ABBREVIATIONS AND SYMBOLS

SRA:	Site response analyses
1D:	One dimensional
USGS :	United States Geological Survey
WYDOT:	Wyoming Department of Transportation
AASHTO:	The American Association of State Highway and Transportation Officials
ASCE:	American Society of Civil Engineers
IBC:	International Building Code
Vs30:	Average shear wave velocity over the top 30 meters
Vs	Shear wave velocity
EQL:	Equivalent linear
NL:	Nonlinear
G:	Shear modulus
D:	Damping ratio
SASW:	Spectral analysis of surface waves
MASW:	Multichannel analysis of surface waves
ReMi:	Refraction micro-tremor
MAM:	Micro-tremor array method
FDBF:	Frequency domain beamformer
FK:	Frequency-wave number transformation
MSPAC:	Modified spatial auto-correlation
HRFK:	High-resolution frequency-wave number transformation
JWE:	Jackson Wilson East
JWW3:	Jackson Wilson West 3
DC:	Dispersion curve
GPS:	Global position system
UHS:	Uniform hazard spectra
LRFD	Load and Resistance Factor Design
G/G_{max}	Modulus Reduction Curve

CHAPTER 1 INTRODUCTION

1.1 Research Goals and Objectives

The Jackson Wilson Bridge over the Snake River between Jackson and Wilson, WY is planned to be replaced by WYDOT within the next three to four years. The site is about 6 km from the Teton fault, which is capable of producing a magnitude 6.6 earthquake, according to the United States Geological Survey (USGS). The seismic forces used for design of the bridge can be determined using a number of methods. The method presented in this project, one-dimensional (1D) site response analyses, involves modeling the soil using measured soil stiffness parameters and propagating earthquake motions from the bedrock layers through the modeled soil layers and predicting the surface accelerations. The predicted surface accelerations using the 1D site response analysis yielded a lower design response spectra than the generic site class code based procedure at most periods. Surface wave testing was performed as part of this project to measure the shear wave velocity profile at both bridge abutments yielding a Site Class D (AASHTO 2014) designation for both sides of the river.

1.2 Report Organization and Research Goals

In order to properly perform a site response analyses, the soil properties must be measured or correlated to similar soil types, appropriate input ground motions must be obtained, and an analyses must be performed. In order to explain the complexities involved with each of these parameters and steps, this report is organized into six chapters and is organized as follows. The interested reader is referred to a more in-depth and complete report written by Frazier (2019)

Chapter 1 includes the introduction, project goals, and project motivation. Chapter 2 includes a brief literature review. Chapter 3 provides the reader with the project site details including, previously recorded borehole data, the surface wave data gathering locations and analyses procedures, and other necessary information to model the site and determine the shear wave velocity for each abutment.

Chapter 4 includes the procedures used in the 1D site response analysis. These procedures include the scaling and determination of the input ground motions, the analysis types used in the procedure and the dynamic soil properties used for the models for the east and west side of the river. Additionally, chapter 4 presents the 1D site response results including uncertainty analyses and modeling for near fault effects. Chapter 5 presents the final design response spectra and the procedures used for its determination as well as the conclusions and future research recommendations.

The goal of this project is to perform a 1D site specific seismic site response analysis at the bridge location, and to provide design earthquake accelerations for the structural bridge design team. The project is motivated by a desire to provide a more accurate estimate of the design ground motions when compared with the standard code based procedure. The desirable possibility of reduced design acceleration was realized in this project and the bridge design team may elect to use these reduced design accelerations which may result in significant cost savings.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

In Wyoming, the Jackson Wilson Bridge over the Snake River is due to be replaced by the Wyoming Department of Transportation (WYDOT) in the next 3-4 years. The goal of this project is to perform a 1D site-specific seismic site response analysis at the bridge location, and to provide more realistic seismic design spectral accelerations for the WYDOT structural bridge design team. In order to perform these analyses, surface wave data has been measured at the site, input ground motions have been obtained, and the soil properties have been modeled. This chapter provides the reader's background information to familiarize themselves with 1D site-specific seismic site response analysis, surface wave testing, and the models used to perform the site-specific seismic site response analysis.

2.2 Code versus Advanced Procedure

The American Association of State Highway and Transportation Officials (AASHTO), American Society of Civil Engineers (ASCE), and International Building Code (IBC) have all specified similar seismic design regulations to prevent the loss of life and prevent structure collapse. The current codes; ASCE 7, 2016, IBC, 2018, and AASHTO, 2014, help guide the design of most structures in the U.S. with each focusing on a unique structural type (i.e. buildings, bridges and roadways). These codes also encourage the use of site-specific analyses for sites with high seismic demands and thick soil deposits, in areas with very soft or liquefiable soils, wherein the code mandates a site-specific seismic site response analysis be performed, or in locations that lack recorded data. These codes each use the same seismic site classification procedures. The site classes, A-F, are based on the weighted average shear wave velocity over the top 30 meters of soil (Vs30). This project utilized the design guidelines in the Load Resistance Factor Design (LRFD) Seismic Bridge Design Guide (AASHTO, 2014). Current AASHTO code (AASHTO, 2014) requires site-specific seismic site response analysis for sites that classify as site class E or F.

A site-specific procedure may consist of a site-specific hazard analysis, a site-specific ground motion analysis, or both. The site-specific hazard analysis should be considered, regardless of the site classification, if one of the following conditions are met: 1) the structure is deemed critical and a desire to meet the seismic performance objectives is desired, or 2) information about one or more of the active seismic sources in the project's location has become available since the development of the 2006 USGS/AASHTO Seismic Hazard Maps, and the new information will result in a significant change to the seismic design of the site (AASHTO, 2014).

2.3 Site-Specific Seismic Site Response Analysis

Site response analyses are a powerful tool that can be used to predict the design forces used in seismic design for geotechnical and structural engineering applications. These include retaining walls, liquefaction susceptibility, bridges, buildings, tunnels and others. In order to perform 1D site response analysis, one must determine appropriate 1) type of analysis (equivalent linear, nonlinear, etc.), 2) input ground motions, 3) dynamic soil properties, and 4) soil stiffness and layering. The proper determination of each of these variables can have an effect on the predicted ground motions at the site and may influence the overall cost of the project or remediation needed to withstand the design forces. The remaining four sections of this chapter provide the reader with a brief review of previous literature and important information regarding each of the four important inputs used for 1D site response analyses.

2.3.1 Analysis Procedure

When performing advanced site response analyses there are many model types (including 1D, 2D and 3D models), as well as analyses types (linear, equivalent linear and fully nonlinear). One dimensional seismic site response analysis employing either equivalent linear or nonlinear analyses are the most common. These analyses are much less complicated than the 2D and 3D analyses, while still accounting for soil nonlinearity. One dimensional site response analyses are valid at sites that lack of topographic or basin effects (Kramer, 1996), and usually result in a more accurate estimate of the design ground motions at a site than those produced by the standard code based site class procedures. Due to its common use in practice and research, this study implemented a total stress 1D site response analysis using both the equivalent linear (EQL) and nonlinear (NL) analyses procedures. Figure 1 presents a schematic of a 1D site-specific seismic site response analysis (Nikolaou, 2009).

Many software packages have been developed to model 1D site response analysis, including Shake, DEEPSOIL and Strata (GeoMotions, 2009; Hashash et al., 2016; Rathje and Kotke, 2018, respectively). DEEPSOIL was used in this analysis because of the user-friendly format, the software's ability to perform the EQL and NL analyses simultaneously, and DEEPSOIL is a freely available software that WYDOT personal can download and use in the future.

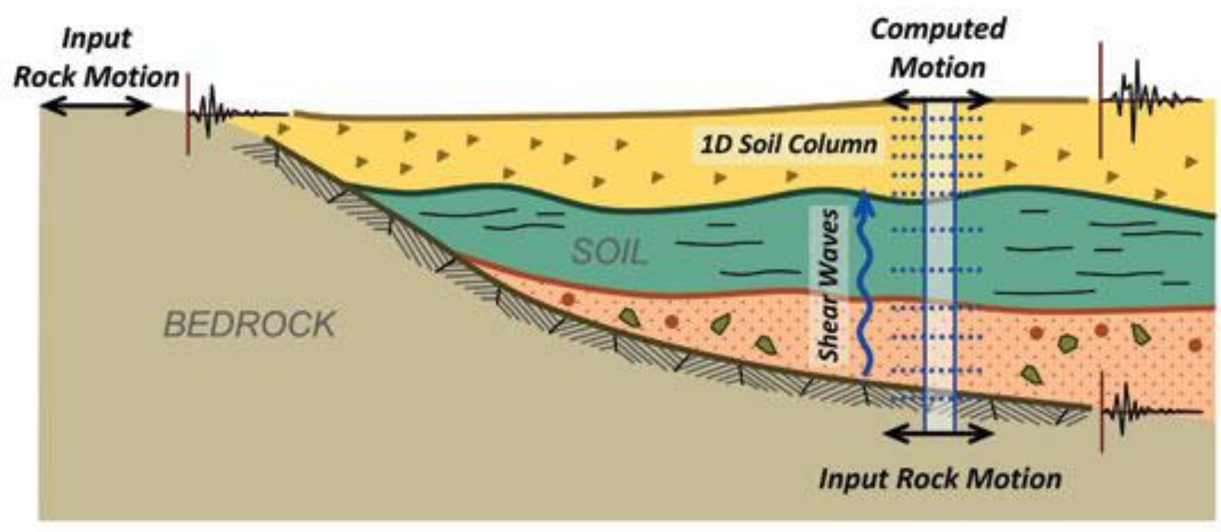

Figure 1. Site Specific Seismic Site Response Analysis (from Nikolaou, 2009).

2.3.2 Input Ground Motion and Scaling

Input ground motions can be determined synthetically, from recorded ground motions or a combination of the two. Synthetic ground motions require special consideration, which have not been used in this study, and therefore, will not be discussed further. Databases of recorded motions, including the Pacific Earthquake Engineering Research Center (PEER) or Kik-net databases (Okada, 2004; PEER, 2018; NIED, 2018); which includes recorded data from past earthquakes for many countries, which are typically used in ground motion selection. These data have the advantage of coming from real recorded earthquakes at surface locations. To help determine appropriate motions for a specific site the data can be sorted by distance from the fault, magnitude, and fault type among other variables. When using scaled ground motions and the user desires to preserve uncertainty Rathje and Kottke (2013) recommend using at least 10 to 15 motions.

2.3.3 Dynamic Soil Properties

The dynamic soil properties used in 1D analyses are the shear modulus (G) and damping ratio (D). Many relationships have been developed to model these dynamic soil properties of various soil types (Kondner and Zelasko, 1963; Hardin and Drnevich, 1972; Hasash and Park, 2001). Each of these models build upon, and modify the modulus and damping relationship of previous researchers, however, each are based on the shear stress shear strain cyclic behavior outlined by Masing (1926). It is important to note that the accuracy of both the EQL and NL solutions is heavily dependent on the assumed G and D curves for each layer. This project utilized the G and D curves developed by Stokoe and Menq (2003), which modified G and D from Stokoe and Darendeli (2001) to include course grained material data, such as gravels.

2.3.4 Surface Wave Testing

The most common surface waves result from body wave interactions with the surface of the earth and can be categorized as either: Rayleigh or Love waves. Rayleigh waves are a result of an interaction between primary waves and secondary vertical waves creating both a vertical and horizontal motion resulting in a retrograde elliptical particle motion. Love waves are a result of secondary horizontal waves and have no vertical motion, but can be oriented in either a vertical or horizontal direction with the particle motion transverse to the direction of propagation.

In practice, measurement of surface waves can be performed using active and passive techniques. The active technique is used when a seismic energy source is generated at a set location, or shot location, relative to the seismic sensor array. Typically the data recording is triggered by the trigger switch attached to the hammer source, and collection occurs over a set time period (Eker et al., 2012). Within active surface wave testing there are two main types of tests: spectral analysis of surface waves (SASW) and multichannel analysis of surface waves (MASW). Nazarian, et al. (1983) pioneered the SASW method in the 1980's. As computation and recording ability increased, Park et al. (1999) developed MASW. The MASW test utilized more sensors and allowed a site to be mapped without the need to reconfiguring the sensors as often as the SASW method.

In contrast to active testing, passive testing uses the ambient earth motions as a signal, instead of an introduced source. Passive surface wave testing data is continuously collected as a continuous time series and motions from ambient field sources are recorded. Passive surface wave testing can be categorized as either the refraction micro-tremor (ReMi; Louie, 2001) or micro-tremor array method (MAM; Okada, 2003; Hayashi, 2008). In general, passive surface wave testing results in much lower frequency content than active testing. By combining the two methods, it is possible to sample both the near surface as well as deeper velocity information as proposed by Foti et al. (2009). The surface wave testing process includes the 1) collection of data, 2) dispersion curve construction and 3) the back-calculation (inversion of the dispersion curves) of the shear wave (Vs) profile for the site-specific seismic site response analysis. The dispersive nature of surface waves, wherein different frequencies travel at different velocities, allows this signal to be utilized to infer near-surface elastic properties of the soil (Nazarian et al., 1983; Stokoe et al., 1994; Park et al., 1998; Cox et al., 2014).

Once data has been collected, using any of the above methods a dispersion curve can be determined. Engineers are interested in both the near surface Vs, as well as depth to bedrock and the Vs of the deep soil layers. In order to determine both the near surface and deeper soil Vs both high and low frequency data must be obtained. Generally the higher frequency data comes from the MASW data collection, while the lower frequency from the ambient MAM data.

There are many methods that can be used to estimate the dispersion curve from the recorded data, these include, frequency domain beamformer (FDBF) for MASW data, frequency-wave number transformation (FK), the modified spatial auto-correlation (MSPAC), and the high-resolution frequency-wave number transformation (HRFK) for MAM data (Capon, 1969; Zywicki, 1999; Bettig et al., 2001). Once the passive and active data have been analyzed, they compose the combined dispersion curve at all measured frequencies. This measured data can then be used to constrain the forward problem and search for Vs profiles that produce theoretical solutions that match the experimental data. Ideally, the collected dispersion data will have an overlap between the MASW and MAM data, helping guide the trend selection and create a useable measured dispersion curve for the inversion process.

Because of the nonlinear, ill-posed, and mixed determined nature of the inverse problem (Foti et al., 2009) it is not possible to directly solve for the Vs profile from the experimental dispersion data. A number of inversion schemes are available to determine the Vs profile from an experimental dispersion curve including linear regression models, Tikhonov regularizations and iterative least-squares models (Santamarina and Fratta, 2005; Aster et al., 2013).

Iterative least squares solutions allow for automation of the inversion (Xia et al., 1999) and use a trial layered earth model with assumed or assigned P-wave velocity (or Poisson's ratio), Vs, mass density and layer thickness, from which an associated theoretical dispersion curve is calculated. The model then compares the measured experimental dispersion curve with the calculated theoretical dispersion curve associated with the trial Vs profile, if a satisfactory fit between the two curves is found, the Vs profile is assumed an appropriate solution for the experimental data. Wathelet et al. (2004) developed a computation of the misfit between the theoretical and experimental data, wherein lower misfit values correspond to a better fit between experimental and theoretical dispersion curves.

As outlined by Griffiths et al. (2016b), the *Geopsy* (2005) software package, used in the inversion process, uses a forward 1D seismic wave propagation model developed originally by Thomson (1950) and Haskell (1953) and later modified by Dunkin (1965) and Knopoff (1964) to calculate theoretical dispersion curves for each trial layered earth model. The *Geopsy* (2005) software allows the user to define an initial trial model with variable stiffness and thickness constraints. *Geopsy* (2005) then uses a Neighborhood algorithm (Wathelet et al. 2004; Wathelet, 2008) to search for all layered earth models within the model bounds that fit within the uncertainty of the experimental data.

Cox and Teague (2016) outlined an approach to determine the initial Vs parameters without any prior information based on a layering ratio selected for the site. Borehole data can be used to establish layer thickness if desired. Using the layering ratio approach with the maximum depth of resolution outlined by Comina et al. (2011), the depth of the inversion models can be established for the calculated Vs profile. The layering thickness constraints are also defined in the layering ratio method by the maximum depth of resolution and the layering ratio defined. Vs profiles should have enough layers to model the soil column under investigation.

2.4 Conclusion

In order to perform a site-specific seismic site response analyses determination of: 1) analysis procedures (equivalent linear, nonlinear, etc.), 2) input ground motions, 3) dynamic soil properties, and 4) soil stiffness and layering are necessary. This chapter has provided the reader with a brief review of relevant information concerning analysis type, input ground motions, and dynamic soil properties. The interested reader is referred to Frazier (2019) for a more detailed review. The basics of surface wave testing which are used to determine soil layering and stiffness has been reviewed and can be broken into three main steps: 1) data collection, 2) data analysis and 3) inversion. There are many methods available for each of these three steps, and some of these where covered in this chapter. The end result of surface wave testing is a measure of the stiffness, shear wave velocity, of the site that is necessary for the soil model used in the site response analysis. These analyses can be performed for any site, but are required for sites that are classified as E and F (AASHTO, 2014).

CHAPTER 3 SUB-SURFACE INVESTIGATION AND SURFACE WAVE DATA ANALYSIS

3.1 Introduction

This chapter presents information concerning the bridge and fault location, borehole and subsurface data collected as well as the data analyses used to determine the Vs profile. Data for this project was collected on both sides of the river at four locations during two separate trips, one in September 2017 and the other in September 2018. The measured data resulted in an experimental dispersion curve that was used in the inversion analyses to determine appropriate Vs profiles. Because uncertainty analyses are an important part of any site response analyses, multiple Vs profiles (in this case 1000) where determined for each abutment. This chapter presents how these Vs profiles where determined and identifies site will be used for further site response analyses.

3.2 Project Site

The Jackson Wilson Bridge crosses the Snake River on Wyoming Highway 22 between the cities of Jackson and Wilson in northwest Wyoming. The current five span, two-lane bridge is approximately 265 m (880 ft.) long and has been in service since the early 1960’s. It is due to be replaced in fiscal year 2022. The bridge site is 3.8 km (2.4 miles) away from the Teton Fault at the closest point. Figure 2 shows the project site’s relation to the Teton Fault.

Based on boring data from 1958, a boring performed upstream of the bridge in 2012 and deep boring performed in coordination with this project, gravelly soils are confirmed to a depth of approximately 27.5 m (90 ft.). Figure 3 presents the locations of the borings performed in 1958 for the current bridge, 2012 for the pedestrian path upstream from the vehicle bridge, and borings performed in 2018 as part of this subsurface investigation and research project.

The 1958 borings performed by a third party hired by WYDOT, the borings were all terminated at varying depths, but all four borings indicated gravelly soils at the site. The borings performed by Nelson Engineering (Figure 3) terminated at a depth of 12.6 m (41.5 ft.) The WYDOT borings (performed by Authentic Drilling) were completed at three locations with two of the borings terminated around a depth of 22.9 m (75 ft.) and one of the borings being completed to a depth of 27.5 m (90 ft.). This deeper boring (ST18-01) includes standard penetration test (SPT) data to a depth of 24.4 m (80 ft.) but at greater depths, due to difficult drilling, no SPT tests were performed and only cuttings where logged. The soil boring data logs are included in Appendix A.

This soil boring data often provides useful information concerning soil type and thicknesses to help constrain surface wave data analyses. At this site, based on SPT data, no definitive soil layering information was discernable; however, the gravelly material appears to become denser as depth increases.

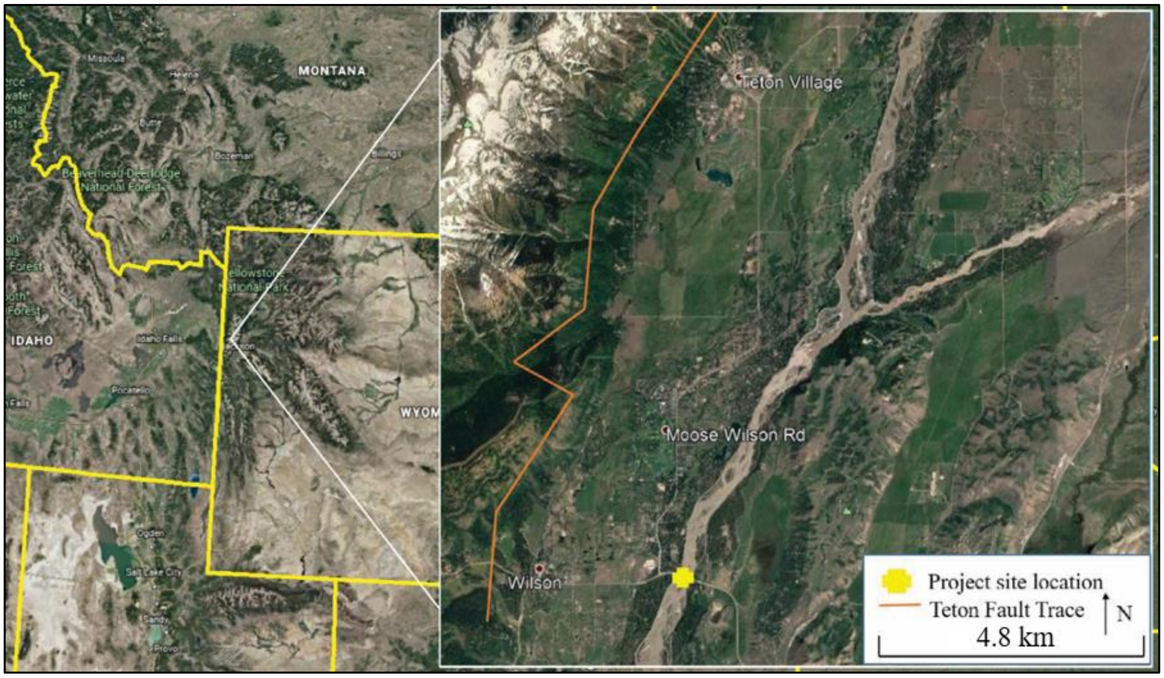

Figure 2. Approximate location of the Jackson Wilson Bridge in northwest Wyoming. Inset presents approximate location of Teton fault relative to the Jackson Wilson Bridge which is 3.8 km (2.4 miles) away (Google, Inc., 2018).

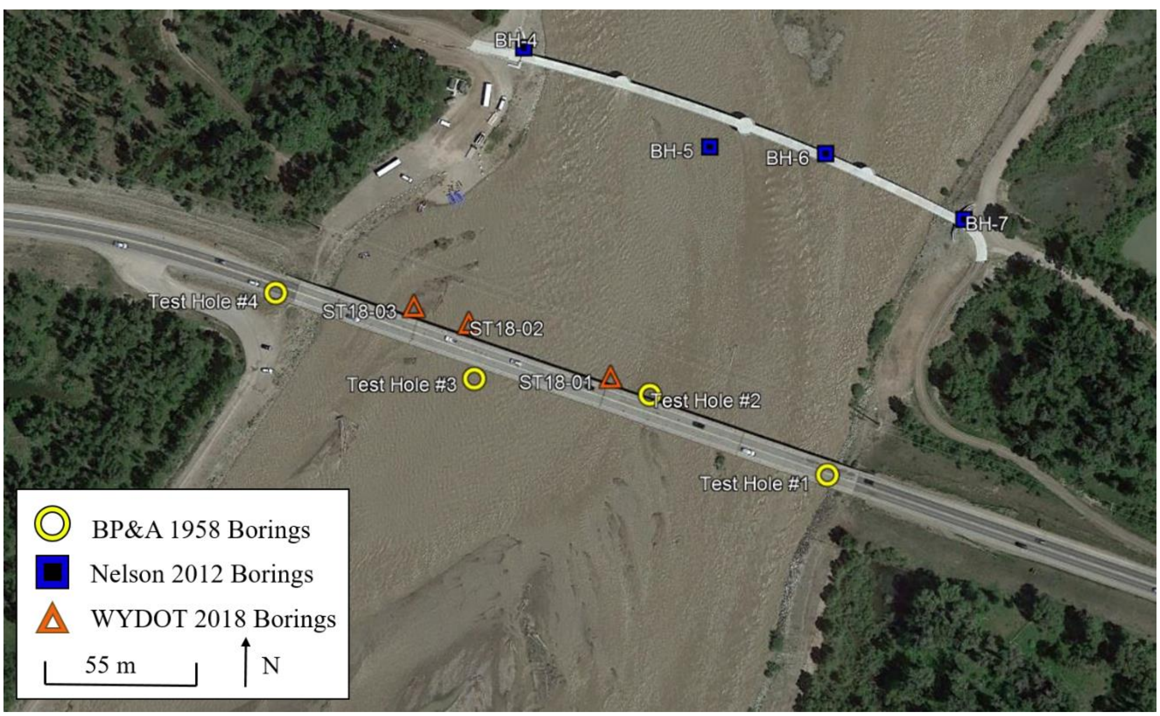

Figure 3. All boring locations used on the Jackson Wilson project site (Google, Inc., 2018).

To accurately classify the site and model the soil profile, surface wave data were collected on the east and west sides of the river. Figure 4 presents the testing locations at this site. Surface wave data was collected in September 2017 and again September 2018. Due to access limitations from the water and vegetation on the site, two data collection sites on the west side of the bridge were used. These are labeled as Jackson Wilson West (JWW1) and Jackson Wilson West 2 (JWW2), and were collected during the same data collection trip as the Jackson Wilson East (JWE) data in September 2017. Due to a poor depth of resolution following the data analyses, a return trip to the west side of the river in September 2018 was performed. This data is termed the Jackson Wilson West 3 (JWW3) data. For this report, for brevity, only data from the Jackson Wilson East (JWE) site and JWW3 site will be discussed

Prior to field data collection, geologic maps, well logs (water and natural resource), and other available information were used to try to estimate the depth to bedrock. One of the goals of this project was to determine the depth of the soil layers above competent rock at the bridge site, which can be an important factor in site response analysis. While the depth to bedrock was not determined prior to field data collection, due to lack of information, it was determined that the bedrock depth was at least greater than 30 m (100 ft.).

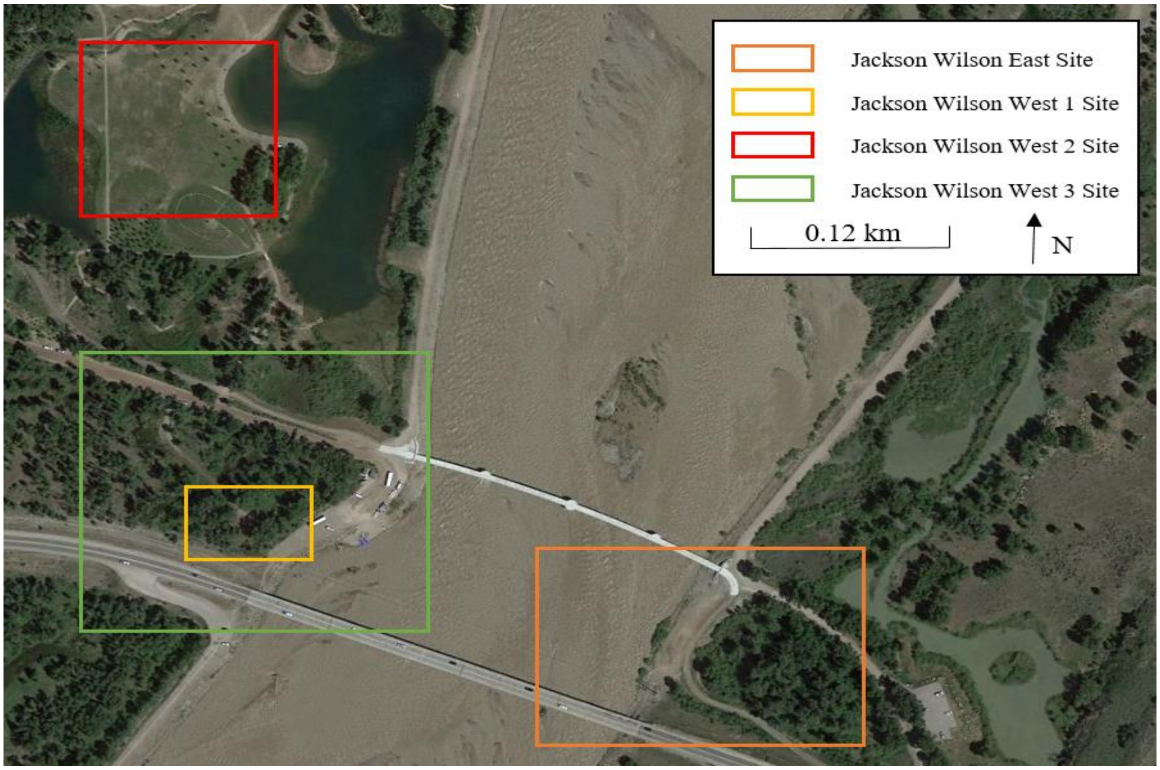

Figure 4. Approximate surface wave testing locations for the Jackson Wilson project site (Google, Inc., 2018).

The subsurface investigation consisted of a number of methods, including: 1) P-wave refraction, 2) MASW, 3) micro-tremor array measurements (MAM), and 4) Horizontal to vertical spectral ratio (H/V). While not every testing method nor array layout was used at each site, each of these methods was used at least once throughout the project. Details concerning specific tests for each trip and/or site will be discussed in sections 3.3 and 3.4.

3.3 East Side of River

Based on the boring logs, differences in lithology and layering from the east and west side of the river are not apparent. As mentioned, it was hoped that the data collected on the east side of the river would yield a depth to bedrock, however, as will be shown the data did not yield discernable data at greater depths (i.e. low frequency). This section presents the surface wave testing, dispersion curve processing, and inversion analyses performed for the east side of the river.

3.3.1 Jackson Wilson East Site Layout and Data Collection

On the east side of the river, multiple tests were performed site including: MASW Rayleigh wave testing, P-wave refraction testing, MAM passive testing, and H/V testing. The testing was performed at the locations and array configurations presented in Figure 5. The P-wave refraction, MASW, and L-array data were collected on the levee parallel to the river, while the triangular passive data were collected on an exposed gravel bar in the middle of the river at a time of low water. Due to the lack of open space and vegetation, the ideal testing location within the stand of trees was not possible.

The P-wave refraction and MASW data were collected using 24, 4.5 Hz vertical oriented geophones spaced at 2 m (6.6 ft.) intervals, and a single Geometrics Geode seismograph. This yielded a total array length of 46 m (150 ft.). The total P-wave record length was 2 seconds with a sampling rate of 0.125 ms and shot locations at each end of the array. As the active source, a 5.4 kg (12 lb.) sledgehammer hitting a steel plate was used. Data collection was triggered using a trigger switch attached to the hammer with a -0.25 second delay. While the P-wave refraction data can be used in the inversion, it was only used on this project to determine the depth to the water table, which was found to be 1.5 m (5 ft.). The depth to bedrock was not determined using refraction testing because the array length was not long enough to sample to these depths and other surface wave testing were expected to yield an accurate estimate of bedrock depth.

MASW data were collected using the same layout and equipment as the P-wave refraction data; only the trigger delay, sampling rate, and record length were changed to 0 second, 4 ms, and a 4-second recording interval, respectively. The MASW testing were completed using six shot locations of 5 m (16 ft.), 10 m (32 ft.), and 20 m (65 ft.) from each end of the array. Individual traces were stacked using 10 records at each shot location. Data for each end of the array were analyzed separately and the better of the two data where used for further analyses, which will be discussed, further in section 3.3.2.

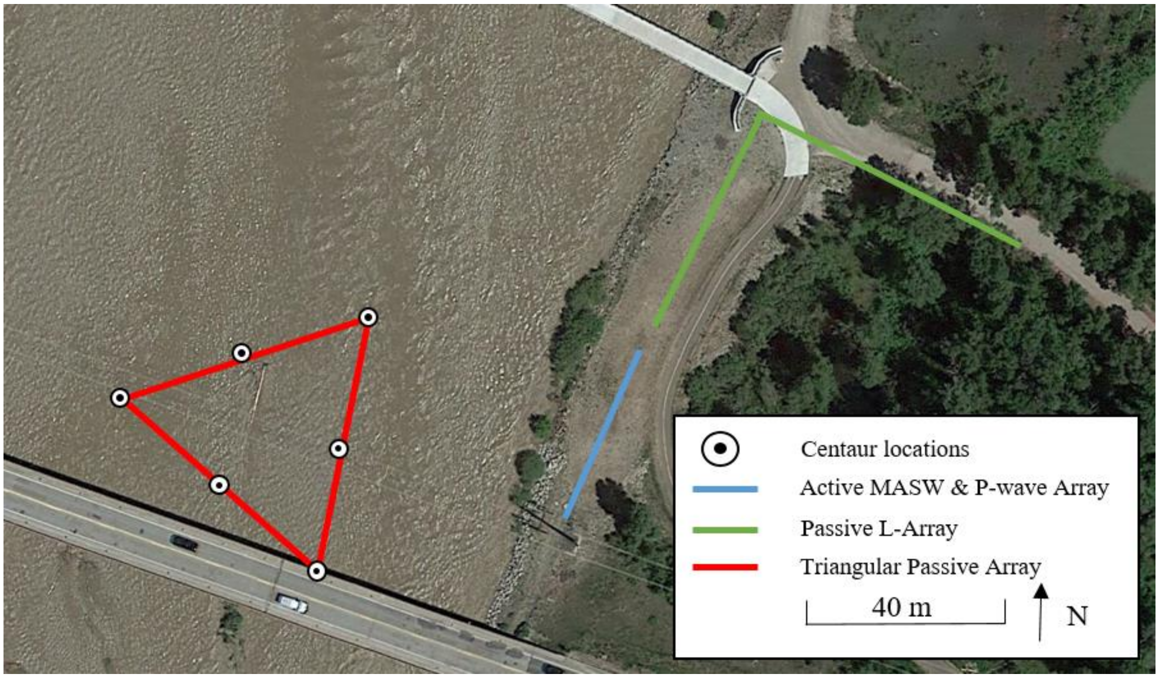

Figure 5. Array locations for the Jackson Wilson surface wave investigation on the east side of the Snake River (JWE). Passive triangular array were performed at time of low water level on a gravel bar (Google, Inc., 2018).

The L-array MAM used the same 24, 4.5 Hz vertical geophones as the P-wave and MASW testing, however, the spacing between geophones were increased to 5 m (16.5 ft.). The layout for the L-array consisted of half of the 24 sensors oriented north/south with the remaining sensors turned ninety degrees east/west, as presented in Figure 3.4. This produced an L-array with leg lengths of 55 m (180 ft.) and 60 m (196 ft.). The L-array data were recorded for 30 minutes without an active source using a sampling rate of 8 ms. The L-array was placed along a paved bike and pedestrian path. Coupling the sensors to the paved surface was accomplished using moist mounded sand. While this coupling may not produce ideal results, the researchers were forced to think creatively when lack of open space and limited options were available.

MAM testing using the nested triangular arrays utilized six Nanometrics Centaur digitizers connected to 120-second three component Trillium compact broadband seismometers. A gravel bar between two flowing channels of the Snake River provided the best open space option available where the researchers had permission to test. Sensors were laid out in a nested equilateral triangular pattern as large as the gravel bar would allow, with the larger triangle having leg lengths of 55 m (180 ft.), and the smaller triangle having leg lengths of 27.5 m (90 ft.). MAM data were collected over a period of 30 minutes and were also used to perform H/V analyses. Once the MASW and MAM data were collected, a measured dispersion curve (DC) was estimated for the site using the procedures discussed in the next section.

3.3.2 Jackson Wilson East Data Analyses

The DC consists of the MASW data above 10 Hz and the L-array MAM and triangular MAM passive data below 15 Hz. Overlap of the frequency from the two data types established the measured DC used to model the site under investigation. The MASW Rayleigh data were analyzed using the frequency domain beamformer (FDBF) method (Zywicki, 1999). Passive L-array and triangular array data were analyzed using frequency-wave number transformation (FK) and the high-resolution frequency-wave number transformation (HRFK), respectively. The FDBF method produced a measured DC of all the MASW Rayleigh wave shot locations. The FK and HRFK produced measured DC for the L-array MAM and triangular array MAM data, respectively. The modified spatial auto-correlation (MSPAC) method (Capon, 1969; Bettig et al., 2001) was also used to spatially analyze the triangular MAM data.

The passive and active dispersion data were combined into a single plot and, any data that did not follow a clear trend were trimmed and thrown out as part of the data refinement process. The remaining data were combined into a single DC with uncertainty determined as +/- one standard deviation at select frequencies. This data combination resulted in a single DC for the JWE site, where MASW and MAM data were combined to preserve uncertainty, as described by Wood and Cox (2012).

As stated previously, one of the goals of this research project was to determine the depth to competent rock. This requires data at low frequencies (i.e., less than 10 Hz). The triangular array data included data at the lower frequencies desired (less than 10 Hz) but it did not produce any discernable trend and was disregarded in its entirety. The lack of coherent data at low frequencies made determination of the depth to bedrock an impractical goal at this site. It is also noteworthy to mention that the H/V data obtained from the triangular array did not produce any discernable H/V peaks at reasonable frequencies. The poor triangular array MAM passive data is thought to be a factor of the poor coupling experienced between the sensors and gravels and cobbles (2.5 in. to 10 in. diameter material) on the gravel bar as well as the small size of the array.

Figure 6 presents the trimmed measured DC for the JWE site. The L-array MAM and MASW Rayleigh wave data overlap between 10 and 15 Hz, which is expected and helps with the data analyses. Included in Figure 6 is the median DC that will be used in the inversion process, along with a +/- one standard deviation. While the depth and low frequency data did not allow for an accurate determination of depth to bedrock, the maximum depth of resolution, as outlined by Comina et al. (2011), allowed the soil profile to be constructed to a depth of 42 m (138 ft.). AASHTO (2014) defines the site classification based on the top 30 m (100 ft.) of soil, thus the 42 m resolution allowed the research team to construct a soil model deep enough to accurately determine the site class. The measured DC was used as the input for the inversion, which is used to determine the Vs profile at the JWE site.

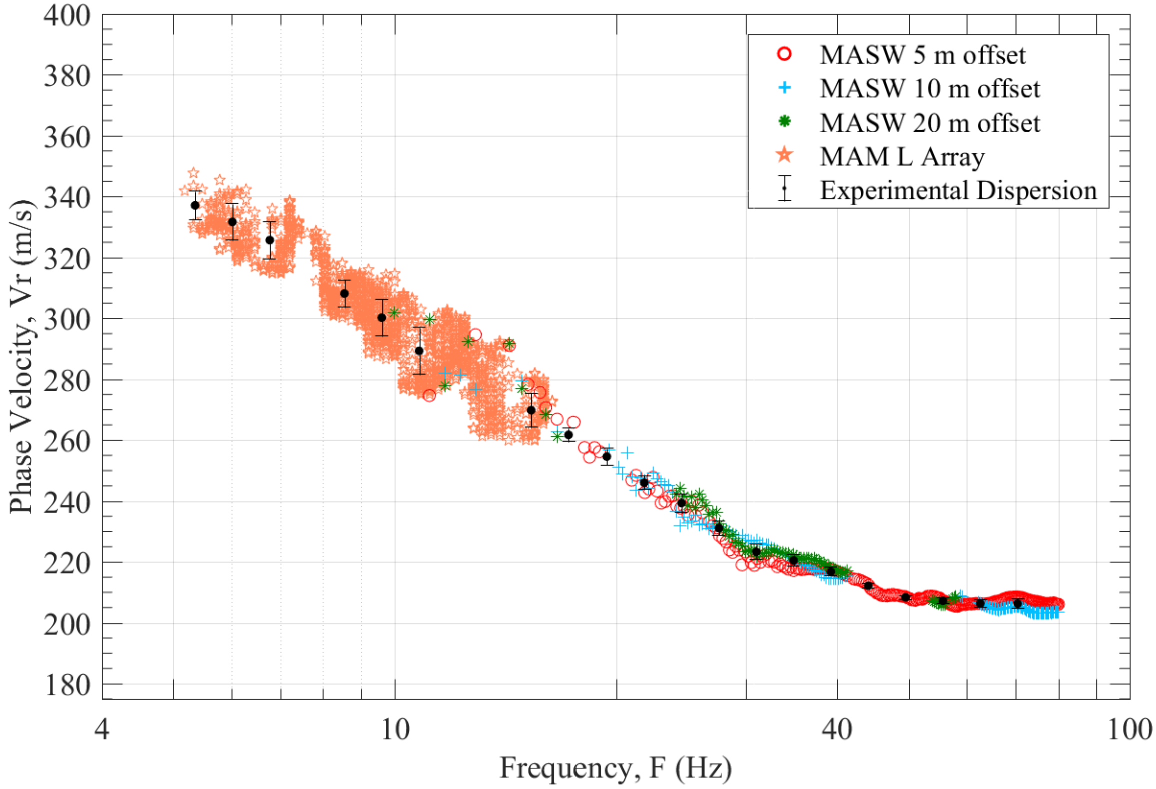

Figure 6. East abutment combined active and passive dispersion data for JWE, after trimming incoherent data, with the median data points included.

3.3.3 Jackson Wilson East Inversion

The *Geopsy* (2005) software was used for the inversion process in obtaining the Vs profile. Within *Geopsy* (2005), the median DC, calculated from the measured DC was used as the starting model to which the theoretical DC would be compared. To determine the initial Vs parameters a combination of borehole data and the layering ratio approach, as presented by Cox and Teague (2016), was used. In total, 37 starting models were investigated with around 200,000 inversions performed for each model. After observing many starting models involving the borehole data and layering ratio information, it was determined; that the borehole information was constraining the top soil layers and negatively affecting the deeper soil layers. Although millions of models were tested only the best 1000 DC and Vs profiles, determined by the minimum misfit values (Wathelet et al., 2004), were extracted from the inversion process.

A layering ratio of 1.4 produced an acceptable DC, with a minimum misfit of 0.538. It is understood that when using the layering ratio approach (Cox and Teague, 2016) without any initial layer boundary conditions that the layering boundaries may not correspond to actual layer interfaces. However, the standard penetration test (SPT) and other boring information did not produce any conclusive boundary layers and the chosen boundaries produced acceptable solutions to the experimental data. The experimental data includes a maximum useable wavelength of 84 m (230 ft.), resulting in a maximum depth of resolution of 42 m (115 ft.) (Comina et al., 2011).

The 1000 theoretical DC are presented in Figure 7 with the corresponding Vs profiles presented in Figure 8. These Vs profiles were each determined using the same starting model (i.e., layering ration of 1.4) described previously. The 1000 theoretical DC fill the uncertainty boundaries for this model (Figure 7), which will allow for uncertainty to be accounted for in subsequent Vs profiles and site response analyses. The large variation in the 1000 theoretical Vs profiles is due to the inversion analyses producing a number of satisfactory theoretical solutions that fit within the uncertainty bounds of the experimental dispersion data. Variation in the 1000 theoretical models is apparent in depth and velocity throughout all depths considered (Figure 8). The 1000 theoretical Vs profiles each increase in stiffness as depths increases because velocity reversals where not allowed in the inversion analyses. Included in Figure 8 is the minimum misfit Vs profile (blue solid line) and the counted median Vs profile (red dashed line), which was calculated from the 1000 theoretical Vs profiles. The median Vs has an average Vs over the top 30 m ($Vs_{,30}$) of 321 m/s (1,056 ft. /s), which classifies as a soil site class D according to AASHTO (2014). In fact, for this site the velocities range from 195 m/s (640 ft./s) to 434 m/s (1420 ft./s), and all 1000 of the theoretical Vs profiles classify as soil site class D (AASHTO, 2014). The minimum misfit profile is the theoretical profile that fit the measured DC the most accurately, while the counted median is a median Vs profile from each of the 1000 best misfit solutions. The JWE inversion analysis presented in Figure 8 will be used to model the east side of the river in the SRA in future chapters.

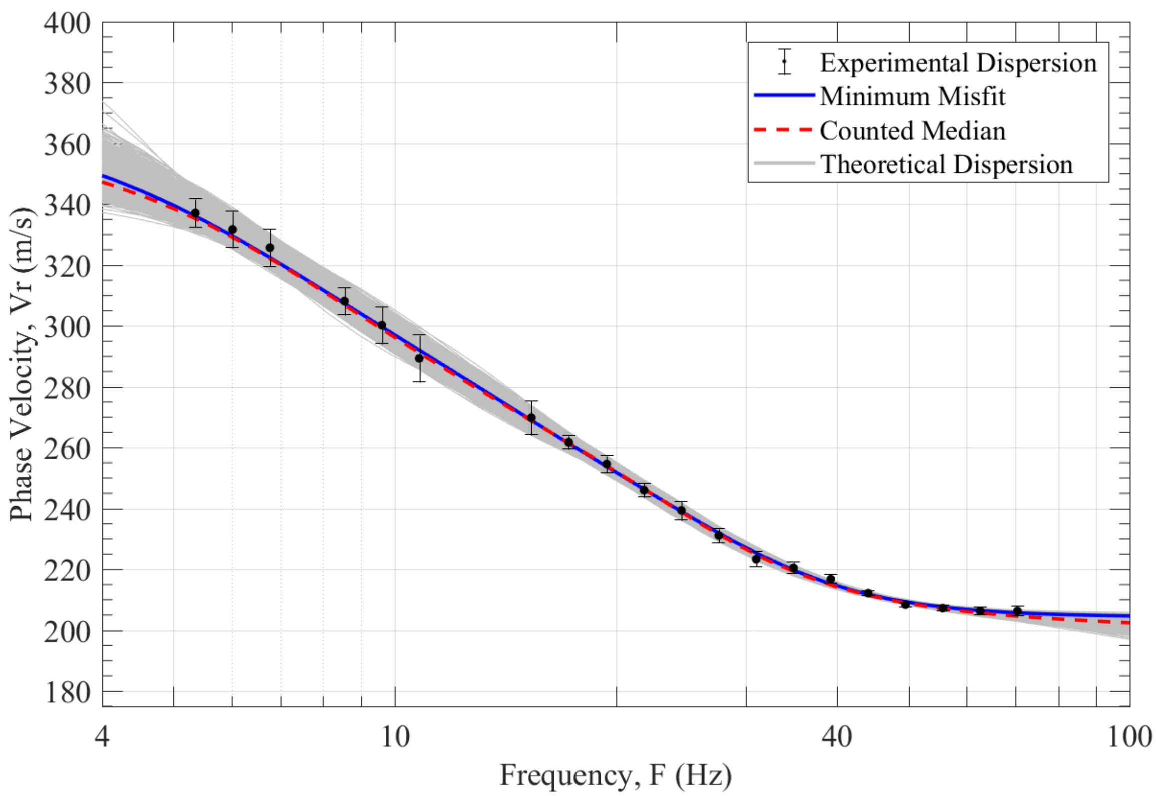

Figure 7. East abutment dispersion data with layering ratio 1.4 including 1000 minimum misfit, counted median dispersion, and experimental dispersion.

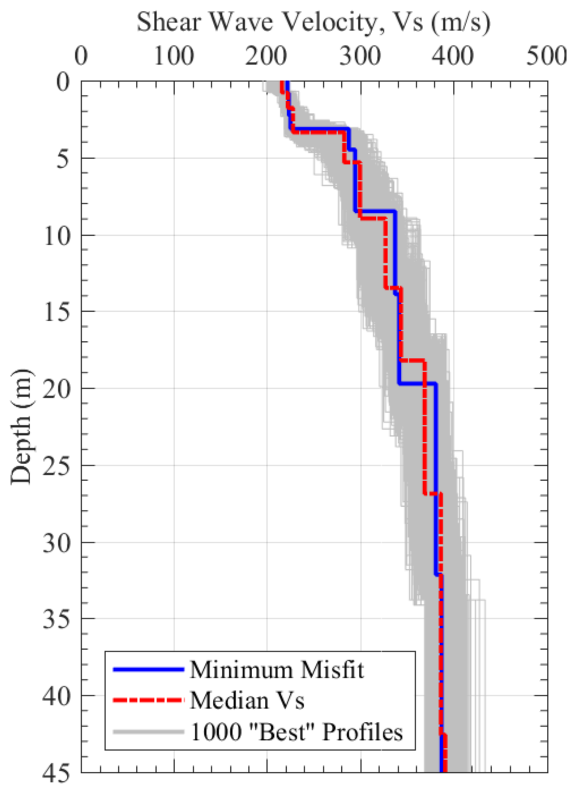

Figure 8. East abutment Vs profiles from the 1000 minimum misfit dispersion results with the minimum misfit and counted median Vs profile.

3.4 West Side of River

This section presents the surface wave testing, dispersion curve processing, and inversion analyses performed for the west side of the river for data collected during JWW3. Data analyses of the other two data collection sites on the west side of the river are discussed in Frazier (2019).

3.4.1 Jackson Wilson West Three Site Layout and Data Collection

At JWW3 multiple tests were performed including: MASW Rayleigh wave testing, MASW Love wave testing, triangular array MAM passive testing and H/V testing. Figure 9 presents the location and orientation of the data collection arrays. The MASW Rayleigh and Love wave *data* were collected on a levee parallel to the river, while the triangular array MAM data was collected in the park to the west of the river.

The MASW Rayleigh wave data were collected using the same sensors and data collection parameters outlined at the JWE site. P-wave refraction data was not collected during this data collection trip. The same shot locations utilized at the JWE site were used at the JWW3 site with 10 records staked for each shot location.

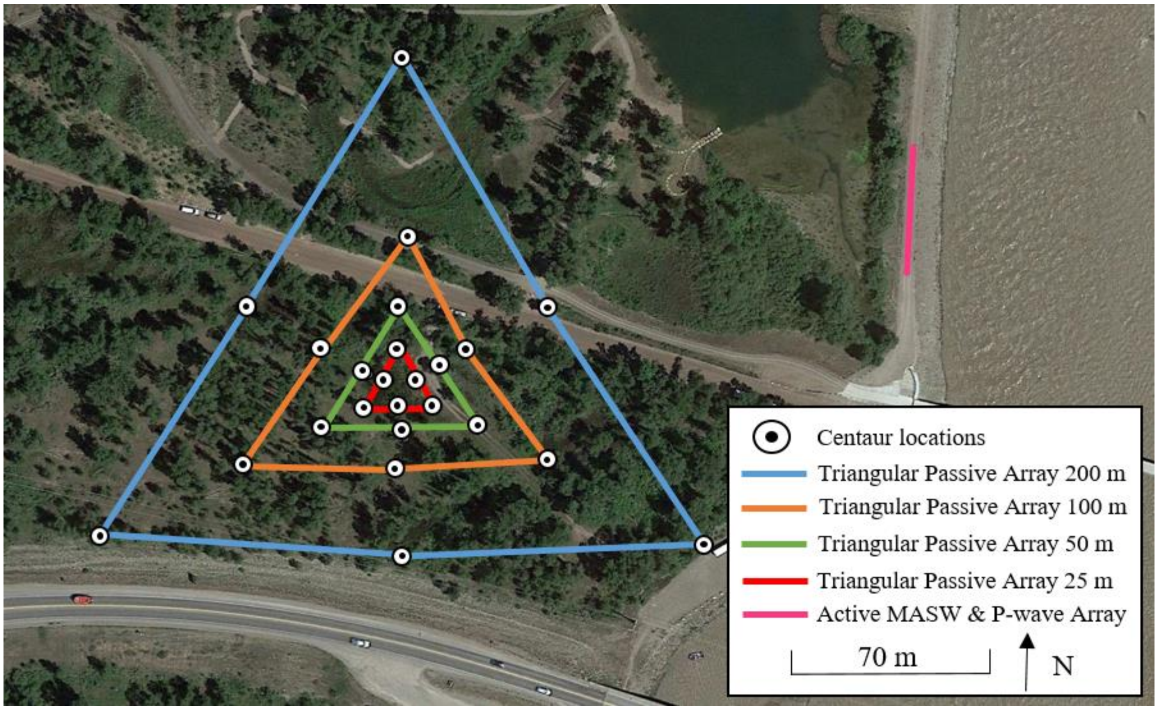

Figure 9. West abutment array locations for the Jackson Wilson surface wave investigation including passive and active data at JWW3 (Google, Inc., 2018).

The MASW Love wave data were collected using 24 horizontal oriented geophones spaced at 2 m (6.6 ft.) intervals using the Geometrics Geode seismograph as the MASW Rayleigh wave testing. The MASW Love wave sensors were placed at the same locations as the MASW Rayleigh wave sensors. The same acquisition parameters, active source 5.4 kg (12 lb.) hammer (striking a shear plank) used for the MASW Rayleigh wave testing were also used for Love wave testing. The Love wave plank was used to generate the horizontal motion of Love waves, instead of the rolling motion of Rayleigh waves. The MASW Rayleigh and Love wave data were analyzed separately from one another and will be discussed further in section 3.4.6.

MAM testing using the nested triangular arrays was performed using the Trillium Broadband Seismometers. Four nested triangle arrays were used for this location as described in the following sequence for the long sides of the array; 1) 25 m (80 ft.), 2) 50 m (160 ft.), 3) 100 m (320 ft.) and 4) 200 m (640 ft.). The four triangular MAM arrays discussed all had smaller, nested triangular MAM arrays within them consisting of leg lengths half the distance of the long leg. Sensor locations were determined before the data collection fieldwork using Google Earth (2018) and global position system (GPS) coordinates. Each array was collected individually beginning with the 25 m (80 ft.) array and moving outward until the 200 m (640 ft.) array was constructed. MAM triangular data were collected over a period of 60 minutes for each individual array and were also used to perform H/V analyses. Once the MASW and MAM data were collected, a measured DC was constructed for the JWW3 site through the data analysis process.

3.4.6 Jackson Wilson West 3 Data Analyses

The measured DC consists of the MASW Rayleigh wave data above 10 Hz and the four triangular MAM passive data sets below 15 Hz similarly to the data discussed at the JWE site. The Love wave data collected from the active and passive sensors was only used to check that the correct mode was being used for the analyses. The MASW Rayleigh wave data and the triangular array MAM data were analyzed using same procedure described in section 3.3.2.

The passive and active dispersion data were combined into a single plot and trimmed using the same guidelines described for the JWE site. Figure 10 presents the trimmed combined passive and active dispersion data. The triangular array MAM data from the 25 m nested triangle overlaps the MASW Rayleigh wave data from 10 Hz to 13 Hz. The lower frequencies collected by the triangular array MAM data relate to the longer wavelengths necessary to sample deeper depths into the soil profile

The +/- one standard deviation median DC begins to increase where the triangular array MAM and MASW Rayleigh wave data begin to overlap because there is more variance in the data at this point. The maximum depth of resolution outlined by Comina et al. (2011) allowed the soil profile to be constructed to a depth of 82 m (138 ft.). This depth of resolution allowed the research team to construct a soil model well beyond the code required 30 m depth for the west side of the river.

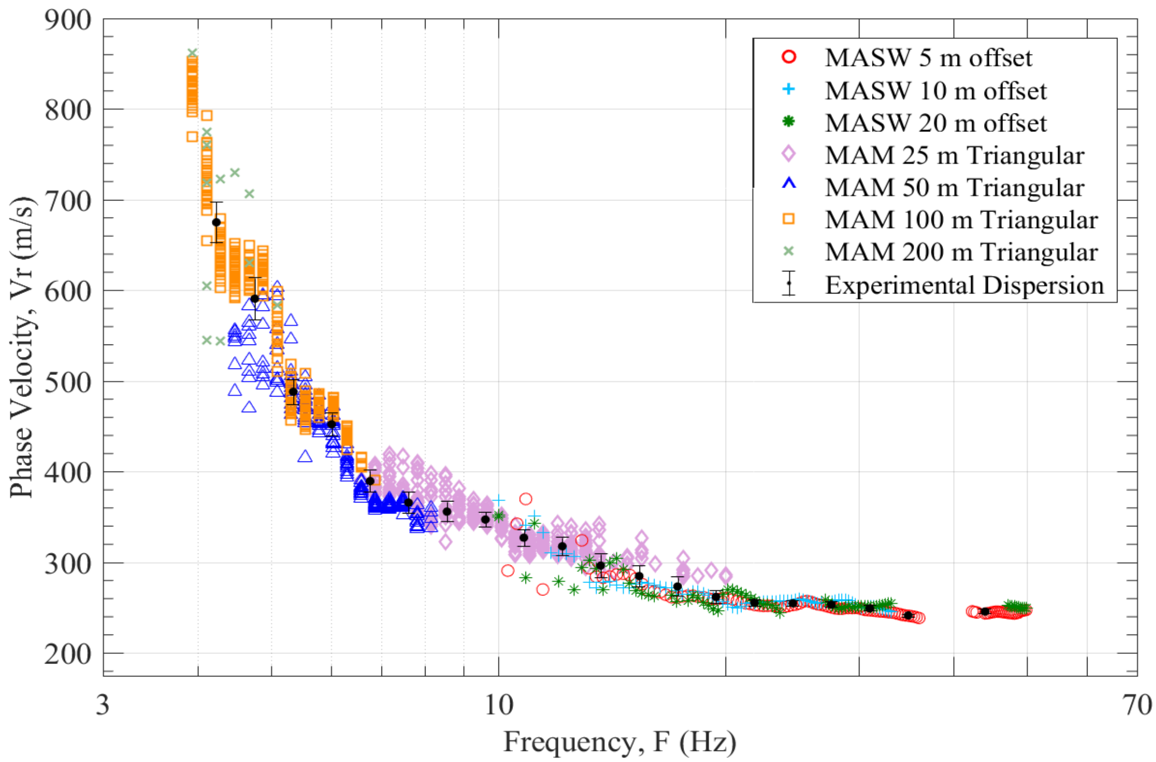

Figure 10. West abutment combined active and passive dispersion data after trimming for JWW3, with experimental dispersion curve.

3.4.7 Jackson Wilson West 3 Inversion

The inversion calculations for the JWW3 site follow the same procedures outlined in the JWE inversion discussion in section 3.3.3. The theoretical DC from the inversion procedure for the west side of the river are presented in Figure 11 with the corresponding Vs profiles presented in Figure 12. As expected, the uncertainty in the dispersion increases at lower frequencies. For this analysis, 24 starting models were investigated with around 200,000 inversions performed for each starting model. A starting model using a layering ratio of 1.5 produced the most acceptable results, yielding a minimum misfit of 0.834. Figure 11 includes the 1000 “best” fit models that are all acceptable DC when considering the experimental data (i.e. all the 1000 theoretical models are within the error bars of the experimental data). Highlighted in Figure 11 and 12 are the median Vs profile (blue solid line) with its theoretical DC and the minimum misfit Vs profile (red dashed line) with its theoretical DC. The layering ratio approach outlined by Cox and Teague (2016) defined the layer boundary depths for the inversion solutions. The depth of investigation was more than enough to determine an accurate site classification according to the AASHTO LRFD Design Manual (AASHTO 2014); however, it still did not reveal an accurate thickness of soil above competent rock.

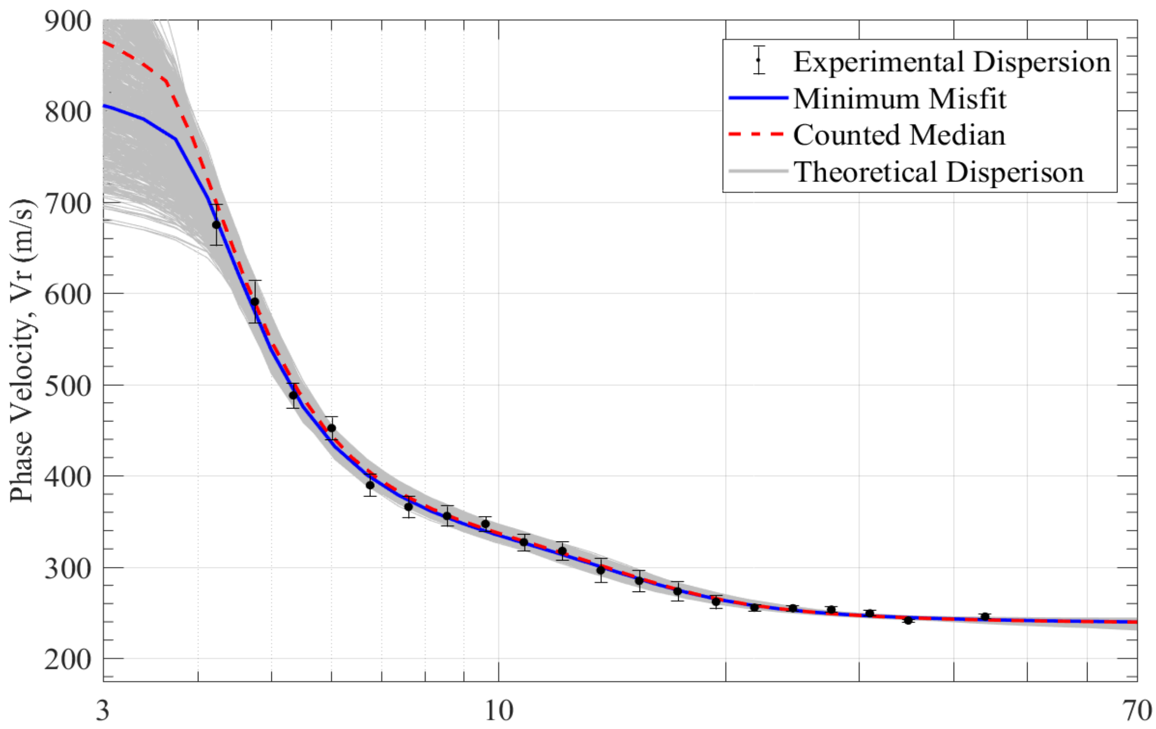

Figure 11. West abutment dispersion data with layering ratio 1.5 including 1000 minimum misfit, counted median dispersion, and experimental data.

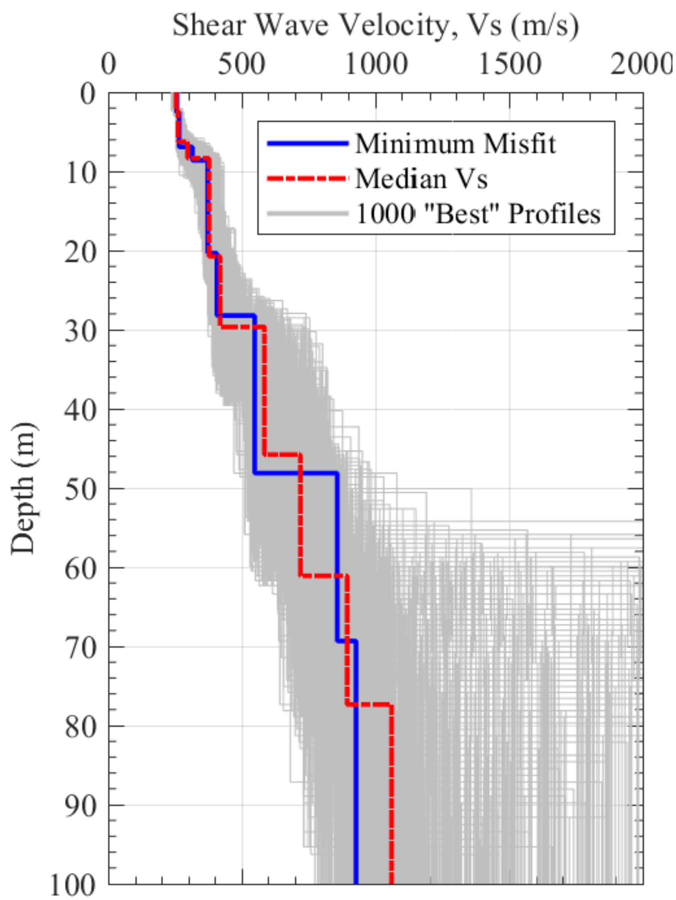

Figure 12. West abutment Vs profiles from the 1000 minimum misfit dispersion results with the minimum misfit and counted median Vs profile.

The Vs profiles presented in Figure 12 were each determined using the same starting model (i.e. layering ration of 1.5). The 1000 theoretical DC fill the uncertainty boundaries for this model (Figure 11), which will allow uncertainty to be accounted for in subsequent SRA. Variations in the 1000 theoretical models are apparent in depth and velocity throughout all depths considered (Figure 3.19). The 1000 theoretical Vs profiles each increase in stiffness as depths increase, much like the east side models, because velocity reversals were not allowed. The median Vs has an average Vs over the top 30 m ($Vs_{,30}$) of 349 m/s (1,146 ft./s), which classifies as a site class D according to AASHTO (2014).

3.5 Conclusion

The Jackson-Wilson bridge site was characterized using borehole data from 11 borings collected from 1958 to 2018. The location of the bridge is within 4.0 km of the Teton fault, which is the only earthquake hazard for this site. The borehole data combined with the well logs confirm gravelly soil to a depth around 90 ft. On the east side of the river depths up to 45 m were sampled and on the west site depth of 100 m where sampled. The west abutment provided larger areas, where passive sensors could be spread out to sample much greater depths. Although the Vs profiles confirmed stiff soils at depths near 100 m (on the west abutment), no large stiffness contrast was evident in either the dispersion or Vs profiles. Thus, an accurate depth to bedrock could not be determined. Vs profiles for both the east and west abutments that will be used for further site response analyses were presented along with the dispersion curves used to estimate the Vs profiles.

CHAPTER 4 SITE-SPECIFIC SEISMIC SITE RESPONSE ANALYSIS FOR THE JACKSON WILSON BRIDGE SITE

4.1 Introduction

This chapter presents the site response analysis (SRA) for the Jackson Wilson bridge site. The desire of the SRA is to predict the design spectral accelerations at the surface of the site. The prediction will be used for the design of the bridge. SRA includes 1) input ground motion selection and scaling, 2) the determination of the analysis type, and 3) modeling of the dynamic soil properties, which includes modulus reduction curves (G/G_{max}) and Vs profile information.

The analyses presented within this chapter modeled the east and west sides of the bridge independently. According to AASHTO design guidance (AASHTO, 2014), both sides of the river should be modeled separately and the side with the larger spectral accelerations should be used in design. Uncertainty analyses have been completed by using additional Vs profiles which were selected from the 1000 “best” profiles. The uncertainty analyses were only completed on the west abutment because this side of the river yielded greater spectral accelerations than the east side.

4.2 Site Specific Seismic Ground Motions

In order to perform site response analyses input ground motions are propagated up through the “soil” column (i.e. model developed with Vs and dynamic soil properties). These motions can be obtained through a variety of methods as stated in chapter 2. For this analysis, ground motions where obtained from the PEER database and scaled to the hazard expected at the site by developing a uniform hazard spectrum using the USGS (USGS, 2018) design tool.

4.2.1 Input Ground Motions

The input ground motions used in the site response analysis come from recorded time histories of past seismic events at surface recording stations. Using the USGS de-aggregation tool (USGS, 2018), a uniform hazard spectra (UHS) was developed for the Jackson Wilson site. This UHS was given as a site class B/C boundary site condition from the USGS tool (2018), and was used as a target to find input ground motions for the project. The USGS (2018) de-aggregation tool returned seismic parameters at the Jackson Wilson site of a magnitude (M_W) 6.5 earthquake and a Joyner-Boore distance of 7 km. Figure 13 presents the de-aggregation results from the USGS (2018) results. The Joyner-Boore distance is defined as the shortest distance from a site to the surface projection of the rupture surface (Kramer, 1996). The information from the USGS de-aggregation tool is used as the search criteria in the PEER (NIED, 2012; PEER, 2018) ground motion database to search for recorded acceleration time histories.

The PEER database includes recorded time histories from many countries as mentioned in section 2.3.2. The search of ground motion database using a magnitude range of 6-7 and a distance of 10-50 km resulted in 116 unique time histories. These ground motions do not account for soil characteristics of the site nor do they match the expected hazard. To account

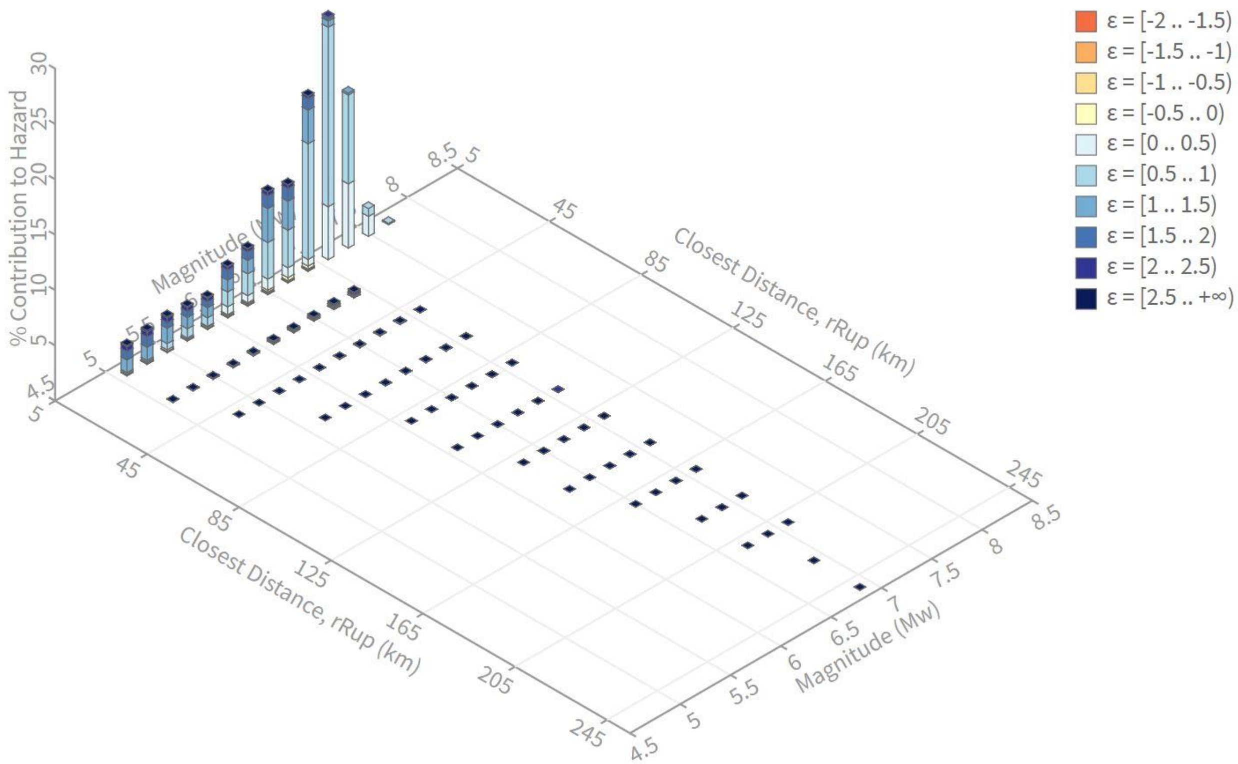

Figure 13. USGS (2018) de-aggregation results returned for the Jackson Wilson Bridge site.

for the hazard at the site, Sigma Spectra (Rathje and Kootke, 2013) was used to select a suite of ground motions most similar to the UHS developed for the site.

Jackson Wilson site, as well as the 16 input ground motions that were selected, are the most similar to the UHS. Once the 16 input ground motions were selected to represent a seismic event at the Jackson Wilson site, they were scaled to match so that on average they matched the UHS. Figure 14 presents the UHS, the scaled ground motions, and the median of the 16 time histories in terms of the response spectra. Presented in Table 1 are the scaling factors used for each input ground motion along with the distance, magnitude and fault type for each time history.

4.2.2 Analysis Types

The analysis types used in this research are the equivalent linear (EQL) nonlinear (NL) analyses. Both solutions use the recorded time history of seismic events from surface recording stations, as mentioned in section 4.2.1. The EQL solution solves the site response analysis in the frequency domain while the NL solution solves the site response analysis in the time domain.

The EQL solution, as discussed in section 2.3.1, uses an iterative solution to account for the nonlinearity of the dynamic soil properties in each individual layer. The NL solution, also discussed in section 2.3.1, tracks the stress strain history as a function of time using the hysteric stress-strain relationship defined at the beginning of each time step. To eliminate filtering of input ground motions in thick layers, the NL solution requires smaller sublayers to be used in the model construction.

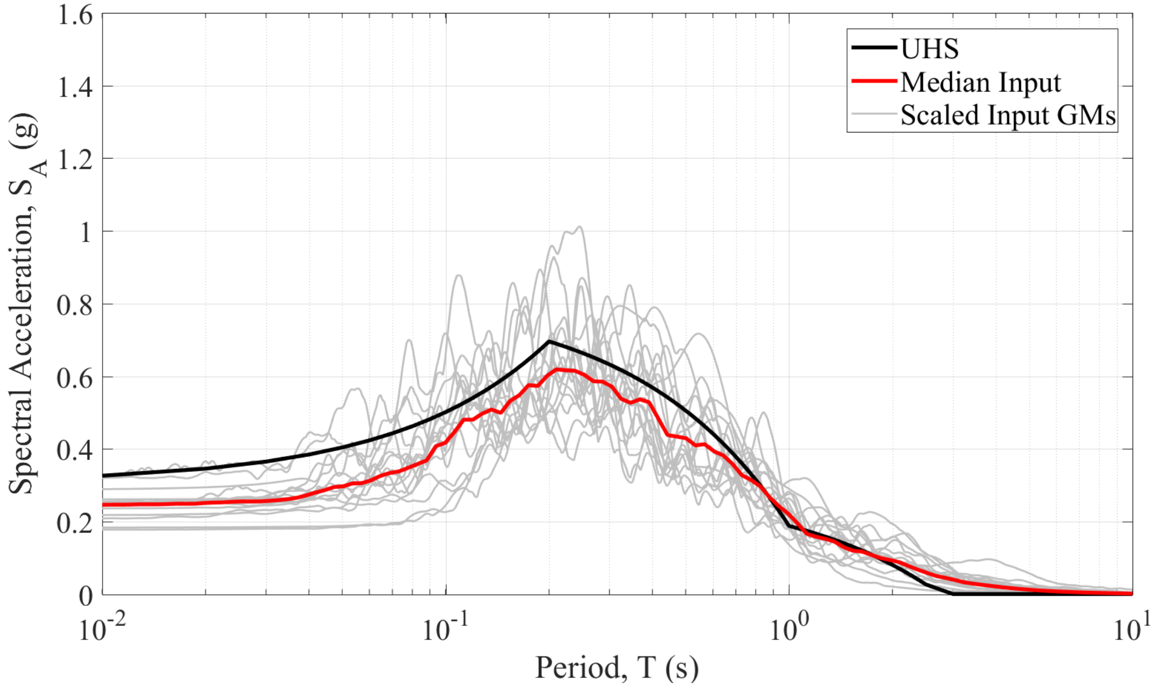

Figure 14. Uniform hazard spectrum, scaled input ground motions and the lognormal median of the input motions.

Table 1. Ground motion scaling factors used for each time history.

Motion	Scaling Factor	R_{jb} (km)	M_W	Mechanism
1125 Kozani	11.66	47.79	6.4	Normal
1137 Dinar	4.45	35.59	6.4	Normal
1139 Dinar	4.25	43.13	6.4	Normal
1141 Dinar	0.66	0.0	6.4	Normal
1752 China	0.96	9.98	6.1	Normal
284 Italy	4.44	9.52	6.9	Normal
287 Italy	5.5	44.62	6.9	Normal
291 Italy	2.08	27.49	6.9	Normal
298 Italy	8	43.5	6.2	Normal
313 Corinth	0.84	10.27	6.6	Normal Oblique
4472 Aquila	3.47	17.82	6.3	Normal
4475 Aquila	8.82	19.08	6.3	Normal
4478 Aquila	8.5	11.12	6.3	Normal
4480 Aquila	0.58	0.0	6.3	Normal
4503 Aquila	7.37	39.04	6.3	Normal
587 New Zeal	0.92	16.09	6.6	Normal

The defined dynamic soil properties for each layer are dependent on the soil type in the individual layer and are used to calculate the shear modulus (G) and damping (D) curves for the EQL solution.

4.2.3 Dynamic Soil Properties

The Jackson Wilson site consists of gravelly soils to a depth of 30 m (100 ft.). Consequently, the pressure dependent Stokoe and Menq (2003) dynamic soil property relationship for gravely soils was used to define the G and D characteristics for all layers and all analyses performed as part of this research. The Stokoe and Menq (2003) G and D relationships are programmed into DEEPSOIL (2016), and didn't require any modification while building the model. However, the dynamic soil properties for each individual layer require shear strength information. The shear strength was calculated at the midpoint of every input layer in the model using the equation Mohr-Coulomb strength criteria.

4.3 East Side of the River

To model the site properly, both the east and west sides of the river need to be modeled independent of one another and the site with the higher spectral acceleration values will be passed on for the final design recommendation. The counted median Vs profile, discussed previously, was subdivided and used for the initial SRA.

4.3.1 Modeling Profile

The nine layer counted median model, referred to as the median Vs model for the east side of the river, was used as the constructed 1D soil profile for the east side of the river. The nine-layer model needed to be subdivided into smaller layers for the NL solution. Subdivision of all layers in the 1D soil model was done so that frequencies less than 50 Hz were not numerically filtered. This resulted in a NL model with twenty-three sub-layers. DEEPSOIL (2016) has the ability to calculate the EQL and NL solution simultaneously. As such, the subdivided twenty-three sub-layered model was used to represent the original nine-layer model for both the EQL and NL analyses.

Figure 15 presents the counted median Vs profile east side of the river. The $Vs_{,30}$ for the profile is 317 m/s (1039 ft./s), classifying the site as site class D. AASHTO (2014) states the scaled input motions shale be introduced into the model at a defined bedrock layer or where a large contrast in Vs is observed. Due to sampling limitations experienced at the test site, the deepest layer of the model is the bedrock layer that has a Vs of 387 m/s (1270ft./s) at a depth of 42 m (138 ft.).

4.3.2 Results

Once the model was constructed, and every sub-layer had the dynamic soil properties defined, the input motions from all sixteen seismic events were run through the model using DEEPSOIL (2016). The analyses included both the EQL and NL surface time history prediction. This resulted in a total of 32 individual analyses. In order to compare answers from each input time history, the lognormal median was calculated for both the EQL and NL solutions independently.

Figure 16 presents the EQL (red) and NL (blue) response spectra for the east side of the river, with +/- one standard deviation. Figure 16 also includes the site class D design response spectra, determined using AASHTO (2014), and the reduced two-thirds site class D response spectra. The site class D response spectra is used in this Figure because it represents the design response spectra if no advanced testing and analyses had taken place.

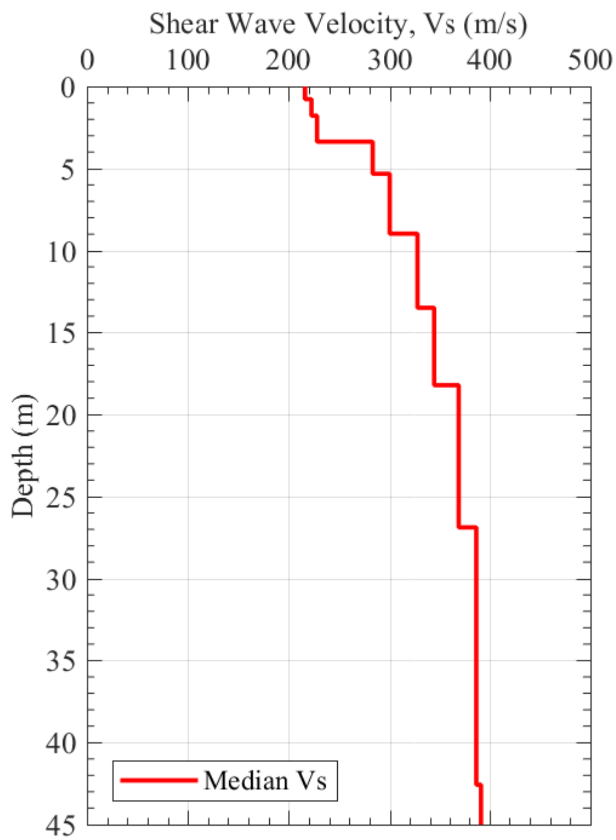

Figure 15. Median Vs profile from the JWE site, used for the site response analysis for the east side of the river with bedrock at 42 m (138 ft.).

The EQL solution provides higher spectral accelerations than the NL solution for all periods observed. The EQL solution surpasses the spectral acceleration defined by the 2/3 reduced site class D design response spectra from periods slightly greater than 0.1 seconds to 1.0 seconds. The NL solution surpasses the spectral acceleration defined by the 2/3 reduced site class D design response spectra from periods 0.2 seconds to 0.4 seconds, and 0.6 seconds to 0.8 seconds. Neither solution exceeds the site class D design response spectra in terms of spectral acceleration. The +/- one standard deviation response spectra from the EQL and NL analyses are included so the user of the response spectra can have an idea of the magnitude of the uncertainties associated with the analyses.

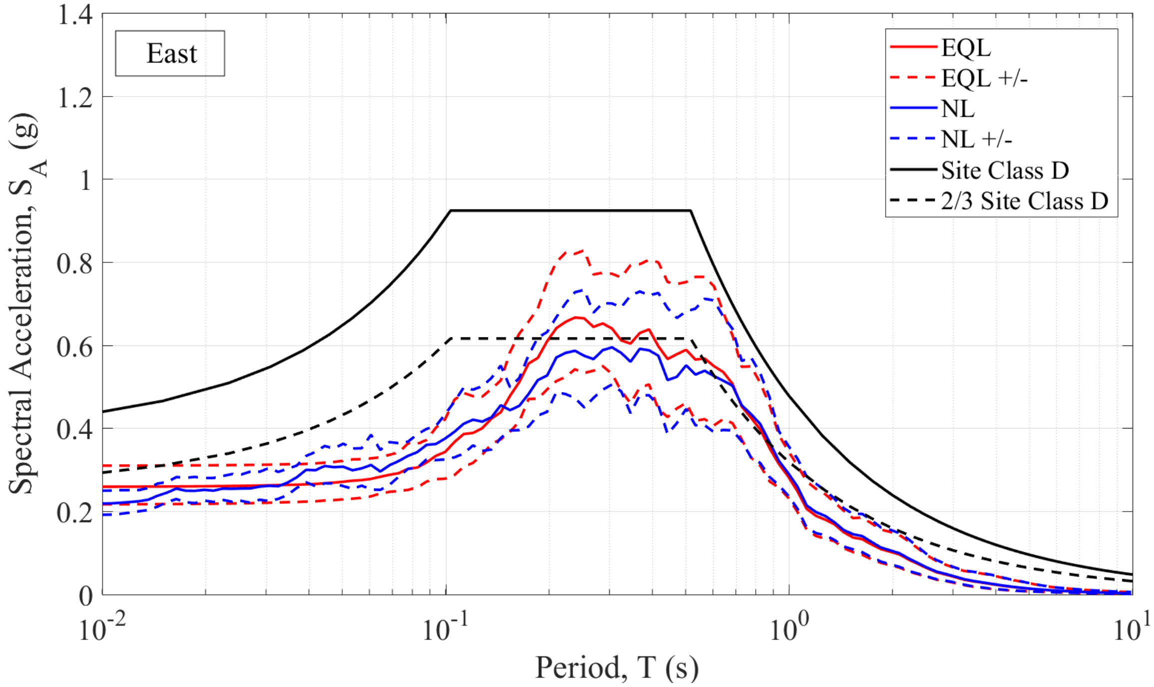

Figure 16. East side (JWE) EQL and NL response spectra with the site class D design response spectra and 2/3 reduced site class D design response spectra.

4.4 West Side of the River

This section presents Vs and predicated response spectra from the west side of the river. Although multiple sites and data were initially tested, only data from the JWW3 site was used in the analyses. As such, this site was used to determine the Vs, and was used for the model development, and was used to predict the spectral accelerations on the west side of the river. In a later section, the predicted spectral accelerations from the east and west sides of the river will be compared prior to recommending a final design response spectra.

4.4.1 Modeling Profile

The counted median profile, referred to as the median Vs model for the west side of the river, was used as the 1D soil profile for the west side of the Snake River. The Vs model for the west side of the river was subdivided into thirty-five sub-layers. The greater number of sublayers used in the west, compared to the east, is due to the greater depth of investigation on the west abutment. Figure 17 presents the eight layer median Vs model (35 sub-layer) used to model the west side of the river. The $Vs_{,30}$ for this Vs profile is 349 m/s (1146 ft./s), classifying the site as site class D. Due to the inability to accurately locate bedrock, the deepest layer of the model is used as the bedrock layer. The bedrock Vs for this model is 1062 m/s (3484 ft./s), at a depth of 78 m (256 ft.). The input bedrock classifies as a site class B, justifying scaling the input ground motions to the site class B/C UHS.

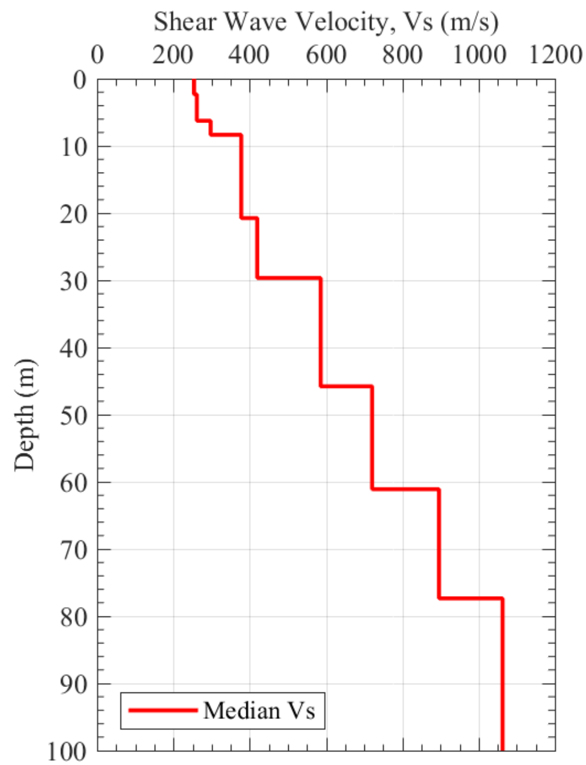

Figure 17. Median Vs profile from the west site, used for the site response analysis soil profile for the west side of the river at 78 m (256 ft.).

4.4.2 Results

Once the west side model was constructed into the thirty-five layer model, and every layer had the dynamic soil properties defined, the input motions from all sixteen scaled time histories were run through the model, in the same manner as the east side model. This resulted in 32 unique predicted response spectra. The lognormal median of the EQL and NL solution were calculated for each of the sixteen input time histories. This yielded two RS for the west side of the river. Figure 18 presents the EQL (red) and NL (blue) solution for the west side of the river, as well as the site class D design response spectra (RS), and the two-thirds site class D design response spectra (AASHTO, 2014). Figure 18 also presents the +/- one standard deviation for both the EQL and NL solutions.

The EQL solution provides higher spectral accelerations than the NL solution for all periods observed. The EQL solution surpasses the spectral acceleration defined by the 2/3 reduced site class D design response spectra from periods slightly greater than 0.1 seconds to 2.0 seconds. The NL solution surpasses the spectral acceleration defined by the 2/3 reduced site class D design response spectra from periods 0.2 seconds to slightly greater than 1.0 seconds. The EQL solution exceeds the site class D design response spectra for periods of 0.2 seconds to 0.4 seconds, and again from 0.6 to 0.9 seconds. The +/- one standard deviation response spectra from the EQL and NL solution provide a measure of the uncertainty associated with the solution.

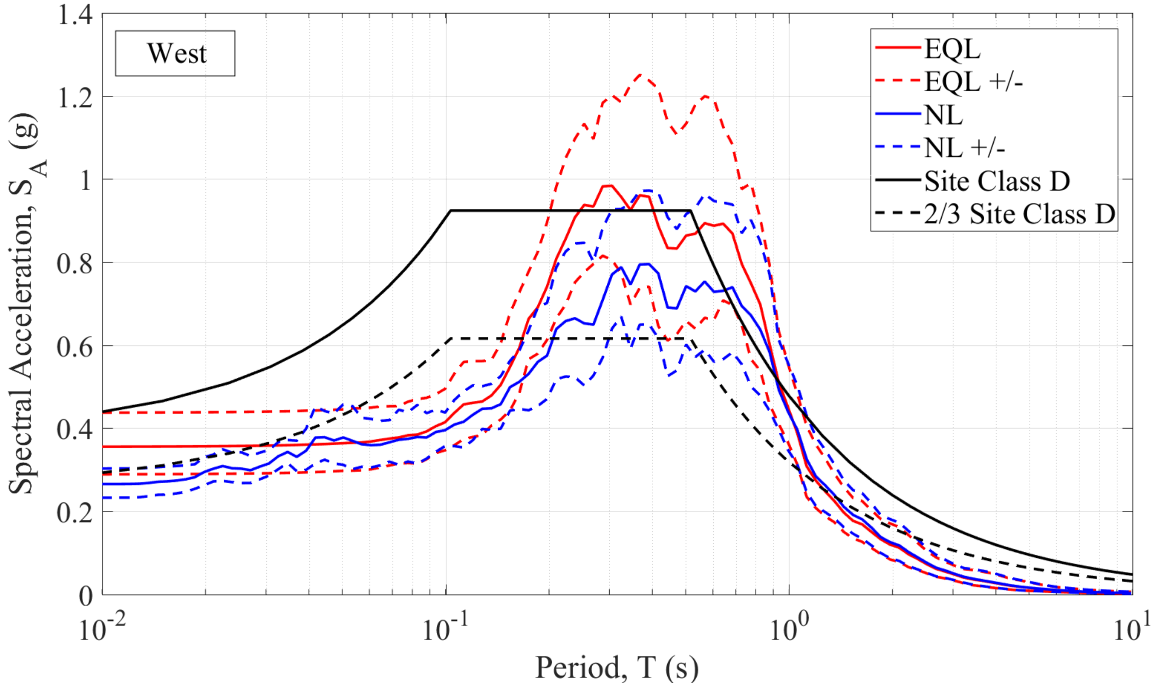

Figure 18. West side EQL and NL response spectra with the site class D design response spectra and the 2/3 reduced site class D design response spectra.

The west side of the river yielded larger spectral accelerations for both the EQL and NL solutions than the predicted spectral acceleration on the east side of the river. Due to the larger spectral accelerations, the west data will be used in the final design recommendations. Accordingly, the west side data will also be used for perform uncertainty analyses that are used to better account for uncertainty in the final design response spectra recommendations.

4.5 Uncertainty and Near Fault Analysis

AASHTO (2014) requires that uncertainty be for accounted when performing SRA. However, how to account for these uncertainties is not clarified. The EQL and NL solutions will account for a portion of the uncertainty in the final design RS recommendation. However, to account for a larger amount of uncertainty in the solution, 9 additional profiles were randomly selected from the 1000 "best" fit Vs profiles. In addition to accounting for uncertainty AASHTO (2014) also requires SRA to account for near fault effects, if the site is located within 10 km (6 miles of the fault). The Jackson-Wilson bridge is located 7 km from the fault, and as such, near fault effects must be accounted for in design.

4.5.1 Additional Profiles

Nine additional Vs profiles were selected from the 1000 best fit Vs profiles. In order to make sure these nine profiles are acceptable solutions to the inversion they are plotted with the experimental data (Figure 19).

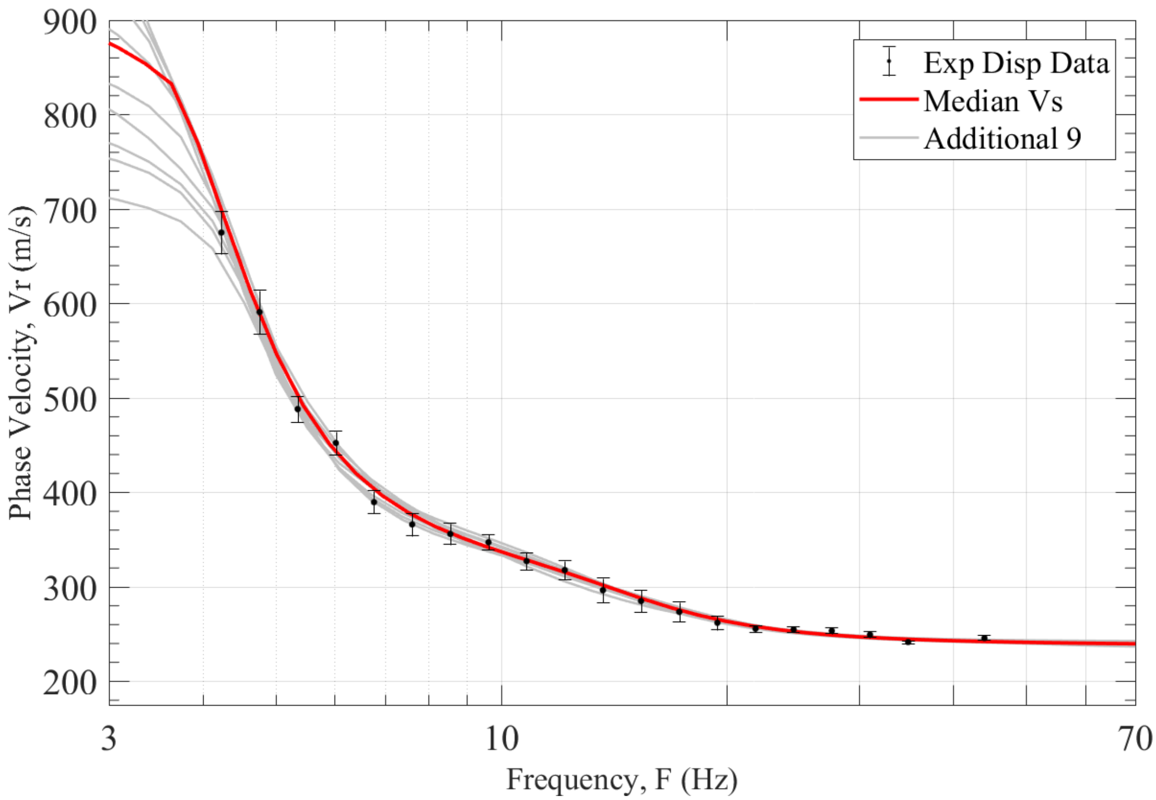

Figure 19. Counted median DC with the additional nine DC from the randomly sampled profiles.

It is clear the nine additional Vs profiles provide acceptable answers to the measured data, and they also provide answers within the uncertainty bounds of the experimental data, which will help carry the uncertainty from the experimental data into the final predicted response spectra.

In addition to the DC of the nine additional profiles, the Vs profiles were compared to the 1000 “best” fit Vs profiles to ensure that soil models constructed for the site response analysis would capture a multiple Vs and depth profiles of the soil models presented in the inversion process. Figure 20 presents the Vs profile associated with the nine additional DC, in Figure 19, and the median Vs profile from the west site. Due to inversion constraints, less variability is present towards the surface of the inversion models. This is expected due the small uncertainty measured at high frequencies. Alternatively, the DC has greater uncertainty at low frequencies resulting in greater variation in the Vs profiles at depth. These trends are visible in both Figure 19 and 20.

Figure 21 presents the sigma natural logarithm of the nine Vs profiles along with sigma ln of the 1000 Vs profiles generated in the inversion process. The similarity of the sigma ln ensures that the nine additional profiles statistically represent the 1000 Vs profiles.

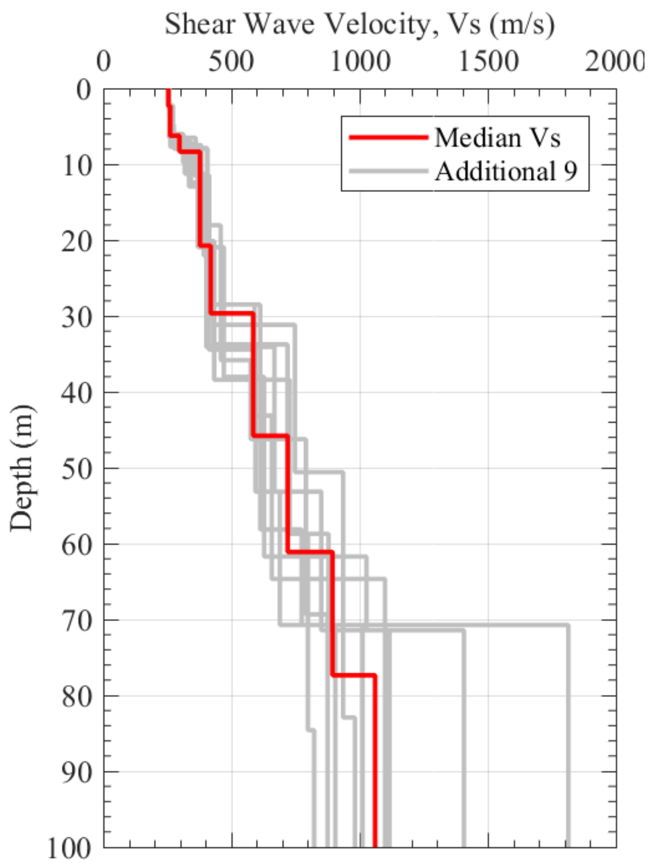

Figure 20. Counted median Vs with the additional nine Vs from the randomly sampled profiles.

4.5.2 Additional Ground Motions

In order to account for near fault effects, time histories that contain significant near fault effects are to be used in the SRA. AASHTO (2014) includes a procedure to determine the proportion of pulse time histories versus regular time histories to use for SRA. Following that procedure 72 percent of the time histories used at the Jackson Wilson bridge site should be pulse time histories. AASHTO (2014) also recommends using a minimum of seven time histories, which corresponds to at least five pulse time histories being used. However, because we used 16 time histories in our analyses, approximately 11 should be pulse time histories. Accordingly, 12 pulse time histories were obtained, scaled to the UHS, and used to perform SRA.

4.5.3 Uncertainty Results from Additional Profiles

The nine additional Vs profiles were used to construct models in DEEPSOIL (2016). The same scaled input ground motions, analysis types, and dynamic soil property model procedures were used for the additional nine profiles as described in the previous analyses. All nine additional Vs profiles were divided into sub-layers to ensure numerical filtering below 50 Hz did not occur.

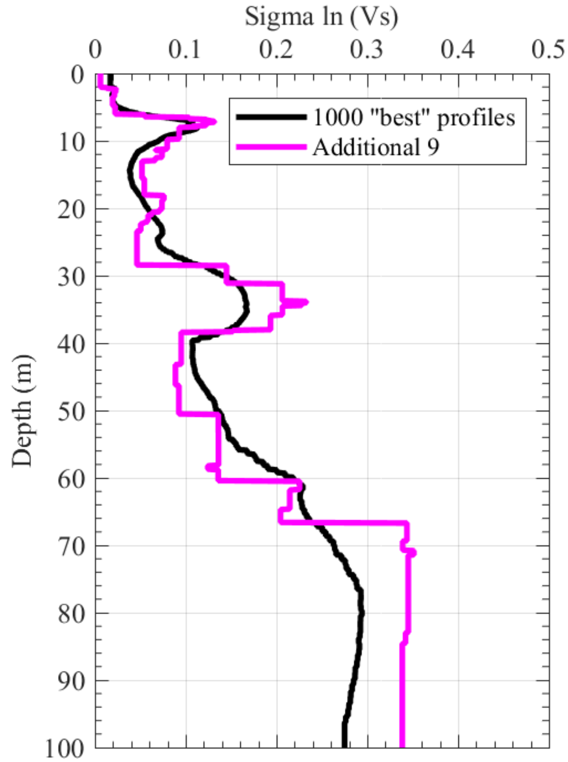

Figure 21. Calculated sigma ln of the 1000 "best" fit profiles and the nine additional profiles.

The shear strain and PGA values were calculated at the layer interfaces in DEEPSOIL (2016). These plots are not included here for brevity, however, the interested reader is referred to Frazier (2019). It is noted that the PGA's and shear strains predicted for this analyses are within the range that is appropriate for and EQL solution as reported by Kaklamanos et al. (2013).

Figure 22 presents the EQL median solution from all nine additional Vs profiles with the median Vs EQL solution for the west site. The median EQL solution from the nine additional profiles and the median Vs EQL solution for the west is also presented. Also presented in Figure 22 is the median plus one standard deviation and the median minus one standard deviation for all ten Vs profiles. Because each of the additional Vs profiles was subject to 16 input time histories, Figure 22 represents 160 total analyses.

From Figure 22 the predominant period of the calculated median solution of the 10 profiles is approximately 0.4 seconds. The nine additional solutions share the predominant period and the median Vs profile solution is approximately 0.3 seconds. Figure 23 presents the NL median predicted response spectra from all nine additional profiles with the median Vs NL predicted response spectra for the west site. Similar to Figure 22 expect in this figure the NL instead of the EQL predicted response spectra are presented.

The predominant period of the calculated median solution of the 10 profiles is approximately 0.4 seconds,. and most of the additional profiles yielded response spectra that is less than the 2/3 site class D response spectra except periods from 0.5 to 1.0 second.

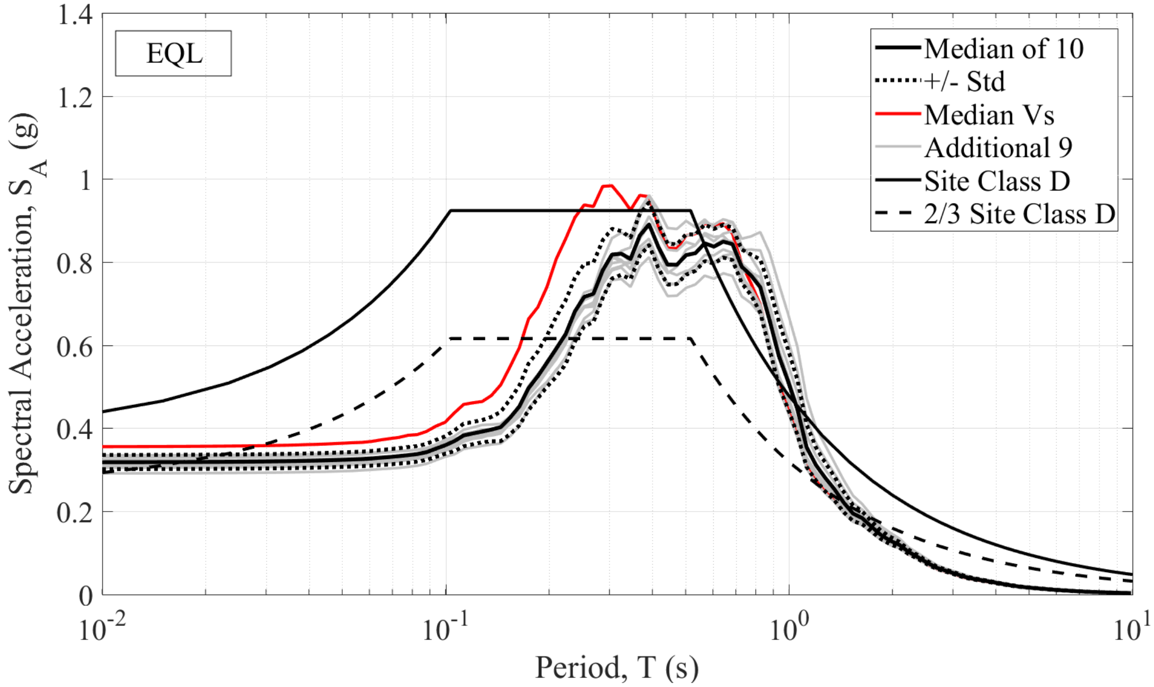

Figure 22. Additional Profile EQL response spectra, including the lognormal median of the nine additional profiles, and the counted median profile with the site class D design response spectra and the 2/3 reduced site class D design response spectra.

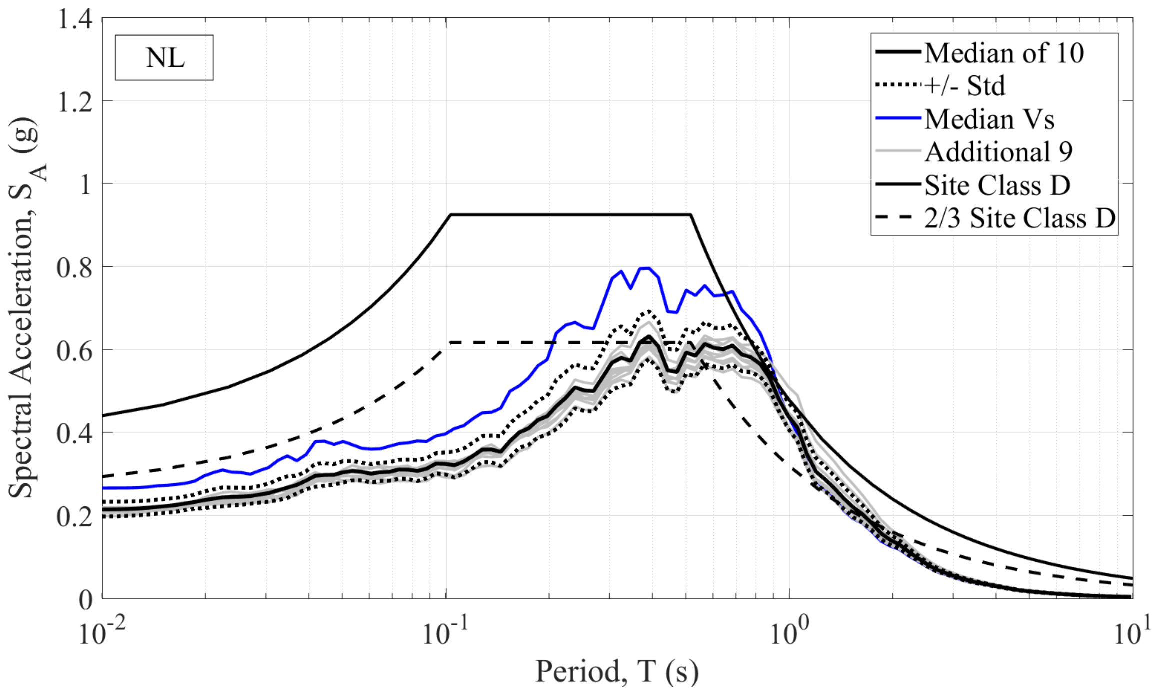

Figure 23. Additional Profile NL response spectra, including the lognormal median of the nine additional profiles, and the counted median profile with the site class D design response spectra and the 2/3 reduced site class D design response spectra.

4.5.4 Results from Near Fault Effects

Figure 24 presents a comparison between the predicted response spectra using the original 16 scaled ground motions, and the response spectra using just the near fault ground motions for both the NL and EQL analyses. The predicted response is very similar for both the directionality (pulse) time histories, and the original scaled 16 time histories. However, accounting for near fault effects should be performed by selecting 12 pulse time histories from a total of 16 ground motions. This would result in a total of 12 pules and 4 non-pulse motions being combined to determine the final design response spectrum.

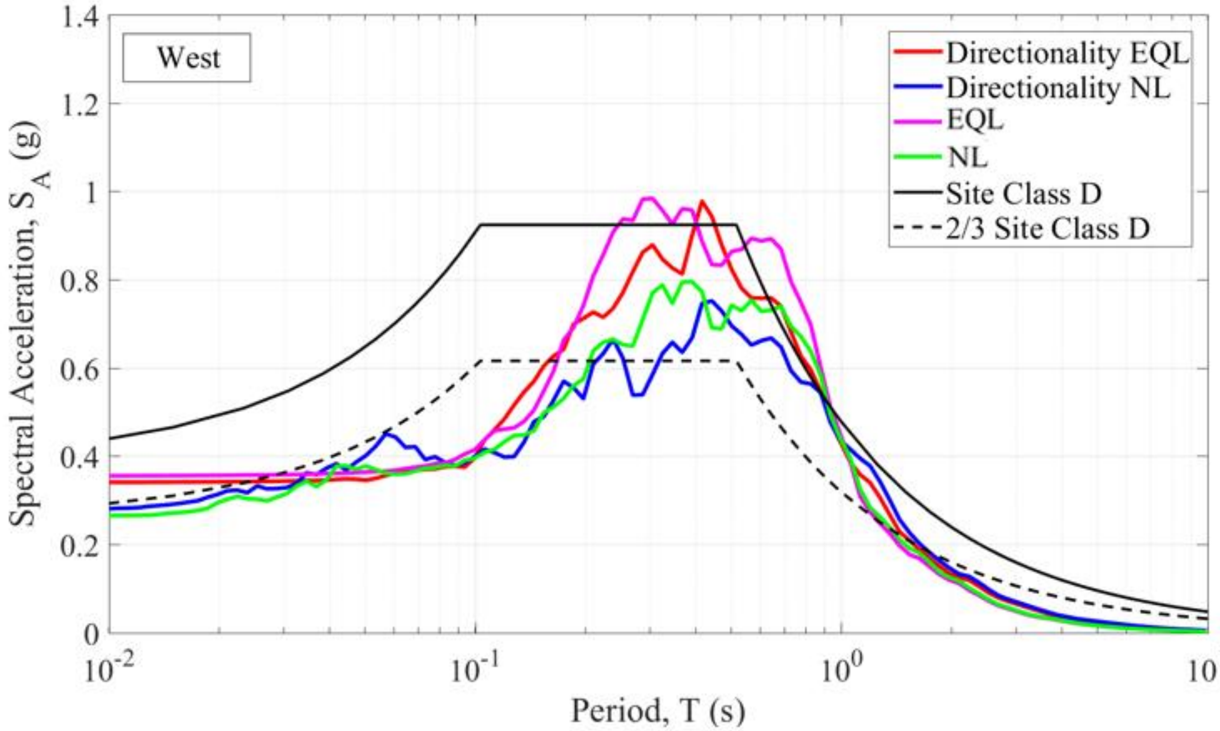

Figure 24. Response spectra comparison between the pulse and the non-pulse time histories.

4.6 Conclusion

The site response analysis for the Jackson Wilson bridge site was presented in this chapter. To reduce overall design costs, the site response analysis aims to lower the spectral accelerations used for design. The site response analysis includes 1) input ground motion selection and scaling, 2) the determination of the analysis type, 3) modeling of the dynamic soil properties, and 4) the model of the soil stiffness. To properly design for the highest spectral accelerations predicted at the site, both sides of the river have been modeled independent of each other. The counted median Vs profile is used to perform the site response analysis on each side of the river. With the predicted response from the west side of the river yielding greater spectral accelerations. To account for more uncertainty in the design, additional profiles were sampled from the 1000 “best” fit Vs profiles, and additional analyses where performed. The next chapter will combine these result and determine a final design recommended response spectra.

CHAPTER 5 DESIGN RECOMMENDATIONS

5.1 Introduction

This chapter present the design recommendations based on the EQL and NL solution types from the ten Vs profiles, and includes accounting for the near fault effects. Only the analyses from the west side of the river are used because this side of the river produced the largest predicted response spectra. The design recommended response spectra cannot be lower than 2/3 of the site class of the site according to AASHTO (2014). Accordingly, the final design recommended response spectra is a combination of the many analyses performed and the allowable reductions according the site class of the site.

5.2 Preliminary Solutions

The uncertainty analyses discussed at the conclusion of chapter 4, took into account ten Vs profiles, the NL and EQL analyses, and near fault effects. Figure 25 presents the median of the ten solutions from the EQL and NL solutions with the median Vs profile EQL and NL solution. Figure 26 presents the median EQL and NL solutions of the ten profiles used to model the west side of the river with the calculated plus/minus on standard deviation of the ten solutions, alongside the site class D design response spectra and 2/3 reduced site class D design response spectra. The EQL and NL solution were previously discussed and presented in chapter 4. For structural analyses these response spectra must be combined into a single response spectrum, which will be a composite spectrum from each of these analyses.

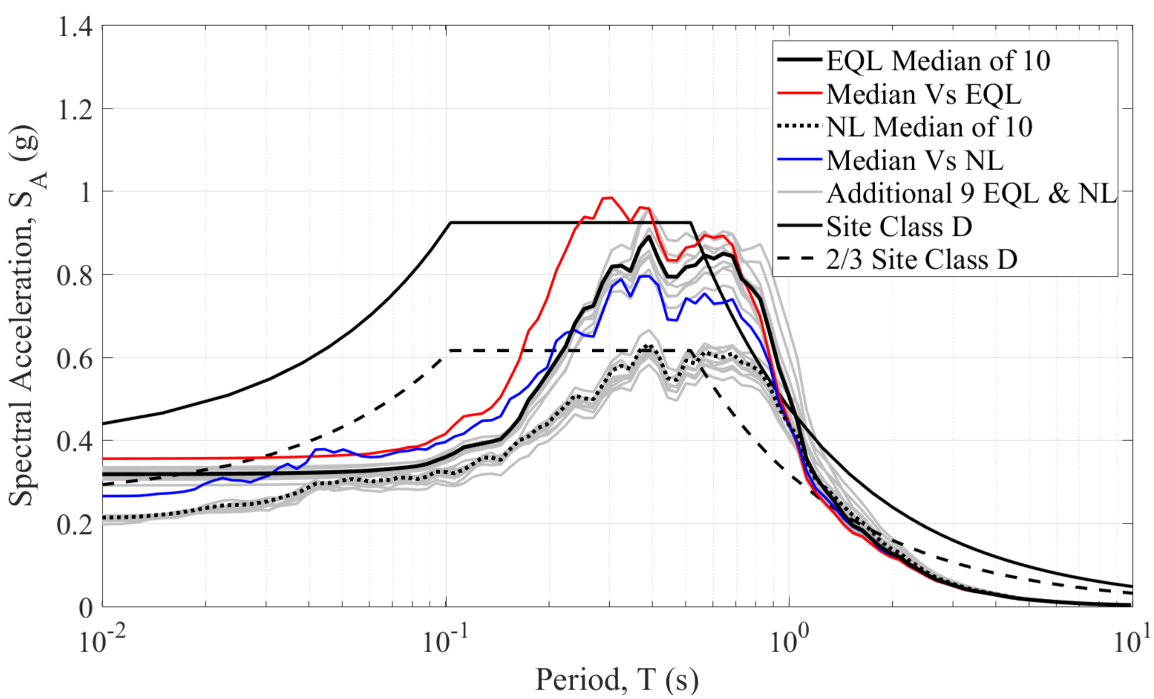

Figure 25. Median EQL and NL solution from the ten individual Vs profiles with the ten individual RS and the median Vs EQL and NL solution.

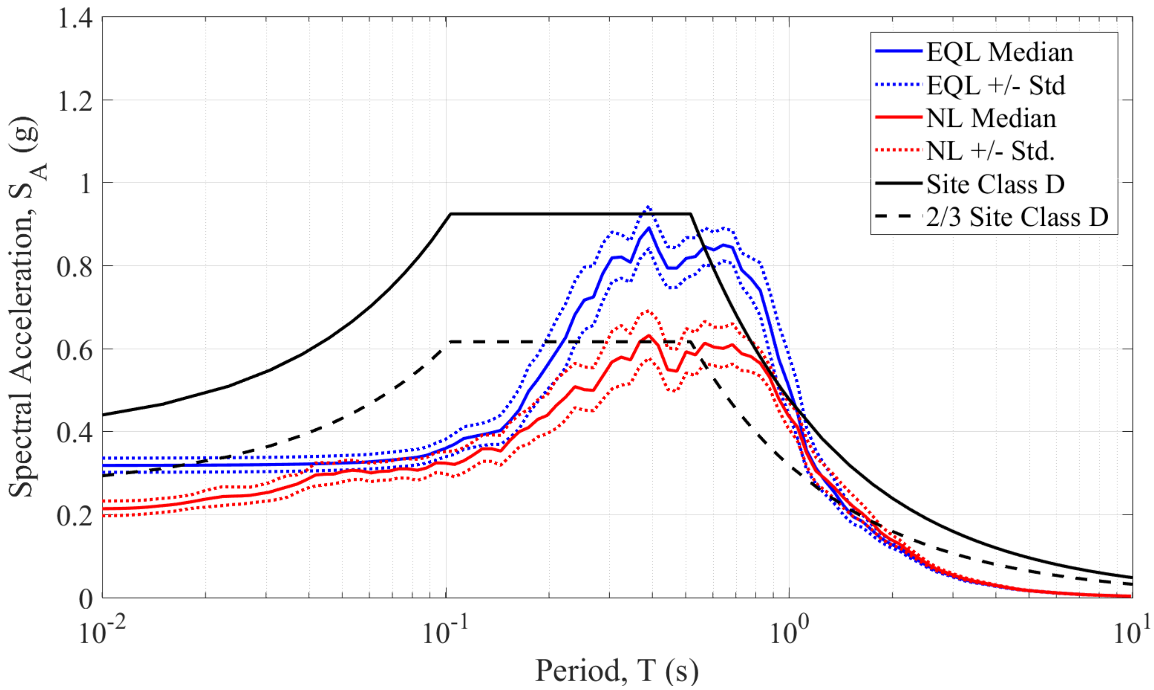

Figure 26. EQL and NL median RS from the ten Vs profiles with the calculated Plus/Minus 1 standard deviation of the ten profiles.

5.3 Design Recommendations

AASHTO (2014) recommends, though does not mandate, a procedure that takes into account 1) the median input GM, 2) the composite response spectrum at the surface, 3) the amplification spectra of the soil column, and 4) the input bedrock site class response spectrum. The recommended, though not mandated, procedure requires the user to determine the amplification spectrum by dividing the predicted surface response spectrum by the average input ground motion response spectrum to obtain the amplification spectrum. Then the amplification spectrum is multiplied by the code designated bedrock site class to determine the design response spectrum. The code allows the user to smooth out the determine design response spectrum by lessening the peaks and increasing the valley's with the final response spectrum.

Once completed, the user then compared the smoothed spectrum to the code based site class spectrum, and the user is not allowed to reduce the design spectral acceleration by less than 2/3 code based site class spectrum. However, this procedure of determining the amplification spectrum, and multiplying it by the site class of the bedrock, is seen as an unnecessary and over-complicated task. The predicted response spectra inherently account for the bedrock stiffness in the EQL and NL formulation. As such, the authors have chosen to diverge from the recommended AASHTO procedure and use the predicted response spectra from the EQL and NL analyses directly.

Figure 27 presents the NL and EQL response spectra. These response spectra were combined into a weighted composite response spectra by weighting the NL and EQL solutions by 50 percent each.   The median scaled input GM used in the site response analysis is also presented in Figure 27, which allows the user to easily see the period that have been attenuated or amplified.

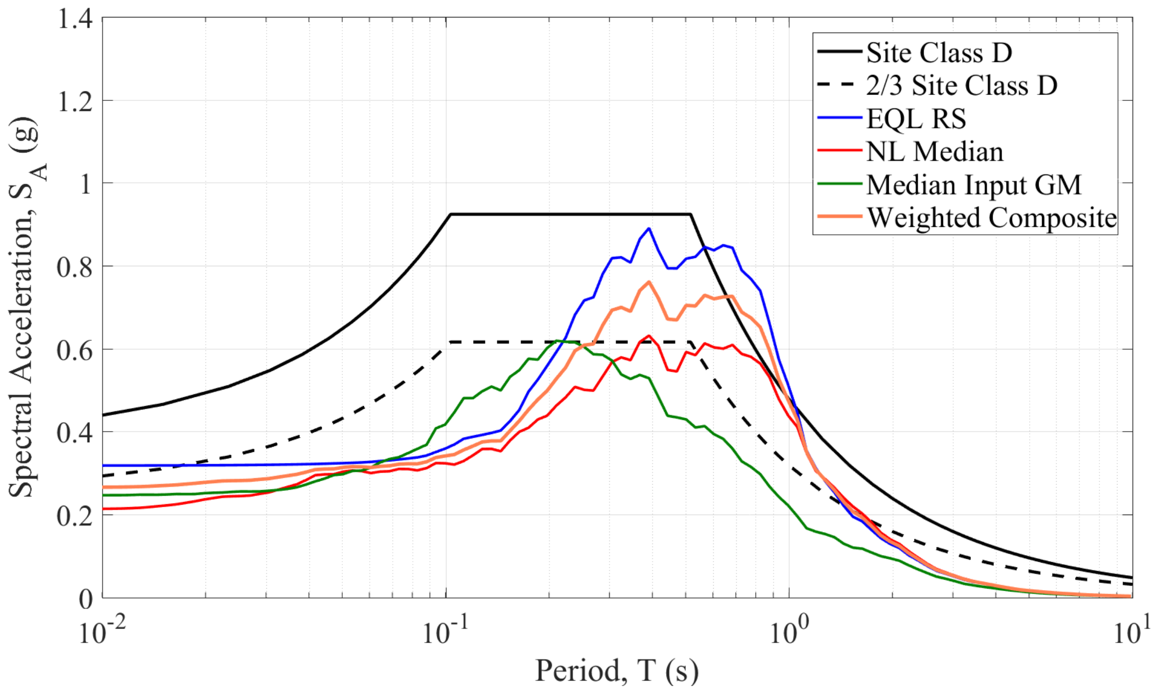

Figure 27. EQL and NL calculated median from the ten Vs profiles, weighted composite RS from equal parts of the EQL and NL solution and the median input GM.

The final design recommended design response spectra for the Jackson Wilson site is a combination of the weighted composite response spectrum that accounts for the variations in the Vs profiles, analyses type, and pulse motions.   Figure 28 presents the weighted composite response spectra with the site class D and 2/3 reduced site class D design response spectra. The design recommendation cannot be reduced to lower than 2/3 the site class D design response spectra. The final design recommendations presented in Figure 28 are accompanied by Table 2

5.4 Conclusions

At periods below 0.3 seconds, the design recommended response spectrum suggests reductions to the 2/3 reduced site class D response spectrum. For periods of 0.6 seconds to 1.0 seconds, the design recommended response spectrum can be lowered to the predicted spectral accelerations. The analyses and design response spectra results in a reduction in the design spectral accelerations at nearly all periods when compared to the site class D response spectra, which represents the response spectra that would have been used for design if no advanced testing or analyses would have been performed. Hence the advanced testing and analyses likely represents a significant cost saving for WYDOT.

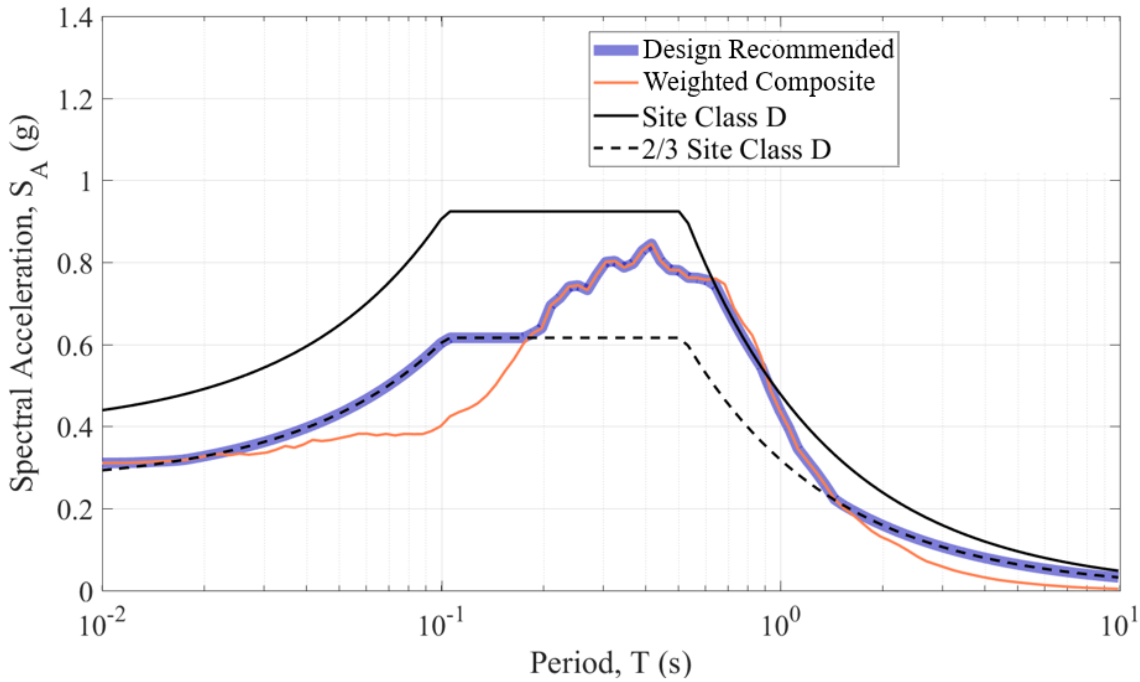

Figure 28. Design recommended RS for the Jackson Wilson Bridge considering the composite spectra and the code allowable reduction in the design accelerations.

Table 2. Final design recommended response spectra.

Period (s)	S_A (g)		Period (s)	S_A (g)		Period (s)	S_A (g)		Period (s)	S_A (g)
0.0100	0.311		**0.0570**	0.382		**0.3247**	0.803		**1.8498**	0.146
0.0106	0.311		**0.0606**	0.382		**0.3455**	0.787		**1.9685**	0.132
0.0113	0.311		**0.0645**	0.378		**0.3676**	0.797		**2.0947**	0.122
0.0120	0.311		**0.0687**	0.382		**0.3912**	0.829		**2.2290**	0.111
0.0128	0.312		**0.0731**	0.378		**0.4163**	0.845		**2.3719**	0.098
0.0136	0.313		**0.0777**	0.382		**0.4430**	0.804		**2.5240**	0.084
0.0145	0.315		**0.0827**	0.381		**0.4714**	0.782		**2.6858**	0.072
0.0155	0.315		**0.0880**	0.381		**0.5016**	0.781		**2.8580**	0.064
0.0164	0.317		**0.0937**	0.389		**0.5338**	0.763		**3.0413**	0.057
0.0175	0.318		**0.0997**	0.401		**0.5680**	0.762		**3.2363**	0.049
0.0186	0.322		**0.1061**	0.425		**0.6044**	0.757		**3.4438**	0.044
0.0198	0.327		**0.1129**	0.435		**0.6431**	0.761		**3.6646**	0.039
0.0211	0.331		**0.1201**	0.444		**0.6844**	0.747		**3.8995**	0.034
0.0224	0.333		**0.1278**	0.456		**0.7283**	0.689		**4.1496**	0.029
0.0239	0.330		**0.1360**	0.476		**0.7750**	0.651		**4.4156**	0.026
0.0254	0.334		**0.1447**	0.502		**0.8246**	0.622		**4.6987**	0.023
0.0270	0.331		**0.1540**	0.535		**0.8775**	0.569		**5.0000**	0.020
0.0288	0.334		**0.1639**	0.565		**0.9338**	0.489		**5.3206**	0.018
0.0306	0.337		**0.1744**	0.601		**0.9937**	0.439		**5.6617**	0.016
0.0326	0.343		**0.1856**	0.626		**1.0574**	0.397		**6.0248**	0.013
0.0347	0.352		**0.1975**	0.639		**1.1252**	0.346		**6.4110**	0.012
0.0369	0.348		**0.2101**	0.695		**1.1973**	0.317		**6.8221**	0.010
0.0392	0.356		**0.2236**	0.714		**1.2741**	0.288		**7.2595**	0.009
0.0418	0.367		**0.2379**	0.741		**1.3558**	0.257		**7.7250**	0.007
0.0444	0.364		**0.2532**	0.744		**1.4427**	0.222		**8.2203**	0.006
0.0473	0.367		**0.2694**	0.733		**1.5352**	0.201		**8.7474**	0.006
0.0503	0.372		**0.2867**	0.769		**1.6336**	0.185		**9.3082**	0.005
0.0535	0.376		**0.3051**	0.801		**1.7384**	0.165		**9.9051**	0.004

CHAPTER 6 CONCLUSION

The objective of this research project was to perform a 1D-site response analysis at the Jackson Wilson bridge site for the Wyoming Department of Transportation (WYDOT). The proposed bridge replacement will likely take place in the next 3-4 years on Highway 22, between Jackson and Wilson, WY, over the Snake River. This report includes; a literature review, data collection, data analyses and design recommendations for the seismic accelerations that can be used for bridge design. The surface wave testing performed as part of this project includes the site classification, which is an important aspect for design accelerations and seismic design category.

Passive and active surface wave testing was performed on both sides of the river. Dispersion data was analyzed using the FDBF (Zywicki, 1999), as well as the MSPAC (Capon, 1969; Bettig et al., 2001) for the active and passive data, respectively. The resulting composite dispersion curves where used along with the layering ratio approach (Cox and Teague, 2016), to determine acceptable shear wave velocity profiles. Inversion analyses where completed using the open source computer software, *Geopsy* (2005), which uses a neighborhood algorithm to search the parameter space for acceptable solutions (Wathelet et. al, 2003). From the millions of searched models a layering ration of 1.4 and 1.5 resulted in acceptable answers for the east and west abutments, respectively. On the east side of the river, the maximum depth of resolution was 42 m and the site classified as a seismic site class D. On the west side of the river, better low frequency data resulted in a depth of resolution of 82 m and a seismic site class D. Although efforts to determine the depth to bedrock were attempted, no evidence based on Vs values greater than 2,500 m/s (8,200 ft./s) concerning a hard, stiff bedrock layer were discovered in the surface wave, borehole or well log data.

A disaggregation determined the site was approximately 7 km (4.4 miles) from the Teton Fault, a normal fault that is capable of producing a magnitude 6.6 earthquake (USGS, 2018). Input ground motions for the site response analyses were obtained using the PEER strong motion database (NIED, 2012; PEER, 2018), and scaled to the hazard at the site using a target UHS based on the probability exceedance of 7 percent in 75 years (USGS, 2018). In total, to predict uncertainty accurately 16 ground motions were used in the site response analyses. The freely available software program DEEPSOIL (Hashash et al., 2016) was used to perform both EQL and NL analyses for both sides of the river. Using 16 time histories, two Vs profiled and both EQL and NL analyses, resulted in over 64 site response analyses being performed. The west side of the river proved to have higher predicted spectral accelerations than the east site, and in order to account for uncertainty in the Vs profiles, an additional nine Vs profiles were randomly selected from the west side data for additional site response analyses. This resulted in an additional 288 analyses. Near fault effects were accounted for by collected an additional 12 time histories.

The AASHTO (2014) design guide recommends accounting for uncertainty associated with site response analyses but does not specify the manner of accounting for this uncertainty. Therefore, the authors used a few different methods to determine a final design response spectrum. A median response spectrum was calculated using 10 spectra for both EQL and NL analyses. These median responses were then combined into a single composite spectrum using an average of the two computed medians. The input time histories included all 16 non pulse and 12 pulse motions that were scaled the UHS for this site. The composite spectrum was determined and can be used for design if the governing agency (i.e. WYDOT) chooses. Alternatively, the AASHTO (2014) code encourages additionally conditioning to determine the design response spectra. This conditioning is done by computing the amplification spectra then multiplying the amplification spectra by the generic bedrock code response spectra on a period-by-period basis to determine the final design response spectra. The final design spectra can then be used for design. It is the author's opinion, the composite spectra, determined from the median results, adequately captures the response from the site and additional conditioning is not necessary, especially if uncertainty bounds are included with the final design response spectrum.

Regardless of the method used to determine the final design response spectra, AASHTO (2014) does not allow the final design spectral acceleration to be less than 2/3 of the seismic site class determined response spectra (2/3 of site class D, for this site). Likewise, the design response spectra does not need to be greater than the seismic site class D response spectra. Therefore, the final design response spectra is a combination of the site class D, 2/3 site class D and the composite spectra. In a period-by-period fashion the design response spectra is the greater of the 2/3 site class D response spectra or the composite response spectra, and it is never greater than the site class D response spectra. The final design response spectra when combined with the generic site class shows that at many periods the design spectra is less than the site class D response spectra. Depending on the period of the bridge (not yet determined), this may result in a substantial cost savings. The site response analysis provided more accurate seismic loading scenarios for the bridge site than the standard code based design.

Future work concerning pile driving and shear wave velocity correlations could result in better predictions of drivability. The difficulties of measuring surface wave data at gravelly sites, is a concern and future work may include attempting to develop better analytical tools or analyses procedures to get better signals in these conditions. Future research could also include a determination of how much cost saving were achieved due to advanced studies or revolve around how to better predict modulus reduction and damping curves for a gravely site current work in the latter area has not yet been performed as gravel are difficult to sample and large lab apparatus are needed for lab tests.

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APPENDIX A

WYOMING DEPARTMENT OF TRANSPORTATION	LOG OF BORING	BORING NO. ST18-01	PAGE 1 OF 4

PROJECT ______ STATION ______ NORTHING ______ LATITUDE 43.49939
LOCATION Snake River Bridge Jackson- Wilson OFFSET ______ EASTING ______ LONGITUDE -110.8401
DATE STARTED 09/25/2018 DATE COMPLETED 09/25/2018 SURFACE ELEVATION 6161.175 ft TYPE ST TOTAL DEPTH 87 ft
DRILLER Authentic Drilling GEOLOGIST T. Ree AT TIME OF DRILLING 6156.175 (5 ft)
DRILL RIG CME 550 ALKALI SAMPLES Yes AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
5.5 - inch ODEX	0														Cobbles at surface. Quartzite, gneiss, granite and other igneous and metamorphic rocks. Cobbles up to 10". *8:30 am*	6161.175
	5														*8:44 am - Groundwater. Added short NQ to take casing to 5'.*	6160 / 6155
	10	Geo 01			015-18-22 (40)										10.0 Cobbles; spoon empty, same as at surface. *8:48 am - Put new show on split spoon.*	6150
ODEX	15															6145
	20	Geo 02			09-14-18 (32)										20.0 Cobbles; Few pieces of well rounded gravel up to 1 1/2" in size, quartzite. *Need to put sand catcher in spoon for next test. New shoe is damaged already*	6140
	25															

(Continued Next Page)

BORING LOG W/ LAB DATA WO DRIVE POINT - WORKING DATA TEMPLATE.GDT - 10/8/18 07:41 - \\CHEYENNE-HQ\PROJECTS\200\00058\GEVDRAFT\2000058_WILSON BRIDGE.GPJ

WYOMING DEPARTMENT OF TRANSPORTATION

LOG OF BORING

BORING NO. ST18-01

PAGE 2 OF 4

PROJECT ______ STATION ______ NORTHING ______ LATITUDE 43.49939

LOCATION Snake River Bridge Jackson- Wilson OFFSET ______ EASTING ______ LONGITUDE -110.8401

DATE STARTED 09/25/2018 DATE COMPLETED 09/25/2018 SURFACE ELEVATION 6161.175 ft TYPE ST TOTAL DEPTH 87 ft

DRILLER Authentic Drilling GEOLOGIST T. Ree AT TIME OF DRILLING 6156.175 (5 ft)

DRILL RIG CME 550 ALKALI SAMPLES Yes AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
ODEX	25														Cobbles; Few pieces of well rounded gravel up to 1 1/2" in size, quartzite. *(continued)* *9:24 am - Added linseed oil to hole to lubricate bit.*	6135
	30	Geo 03			030-25-16 (41)		GP-GM	A-1-a		6.2	NV	NP			30.0 Muddy Sands and Gravel; medium to coarse grained sand with angular and rounded gravel. Broken rock fragments probably from drillings. Gravel up to 1". May not be enough sample for gradation. *9:33 am - Cloudy drill fluid.*	6130
	35															6125
	40	Geo 04			022-20-17 (37)										40.0 Sand and Gravel; +1" gravel plugging shoe. Medium to coarse, subangular to rounded sand, clean with 2 1" gravel. *9:57 am - Not enough material to collect sample.* *Added linseed oil to lube bit.*	6120
	45														45.0 Cobbles with some sand, and dirty/muddy fluid. *10:15 am - Drilling staying consistent and at the same rate.*	6115
	50														50.0	

(Continued Next Page)

WYOMING DEPARTMENT OF TRANSPORTATION

LOG OF BORING

BORING NO. **ST18-01**

PAGE 3 OF 4

PROJECT ______ STATION ______ NORTHING ______ LATITUDE 43.49939

LOCATION Snake River Bridge Jackson- Wilson OFFSET ______ EASTING ______ LONGITUDE -110.8401

DATE STARTED 09/25/2018 DATE COMPLETED 09/25/2018 SURFACE ELEVATION 6161.175 ft TYPE ST TOTAL DEPTH 87 ft

DRILLER Authentic Drilling GEOLOGIST T. Ree ▽ AT TIME OF DRILLING 6156.175 (5 ft)

DRILL RIG CME 550 ALKALI SAMPLES Yes ▼ AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
ODEX	50	Geo 05			015-50-36 (86)		GP	A-1-a		9.3	NV	NP			Sand and Gravel; medium to coarse grained sand with rounded gravel up to 1". Sand is light quarz with darker grains. Gravel is quartzite. Angular pieces may bave been broken from larger gravel or cobbles. Clean water in barrel. *10:30 am - Drill fluid is cleaner, slightly gray.*	6110
	55															6105
	60	Geo 06			015-11-20 (31)		GP	A-1-a		8.2	NV	NP			60.0 Coarse to Granular sand and 3/4" subangular to subrounded gravel. Quartzite gravel. Shoe plugged with +1 1/2" gravel. *11:08 am*	6100
	65														*Slower rate of drilling. Drill fluid dark brown. Very Hard.*	6095
	70	Geo 07			026-19-29 (48)		GW	A-1-a		8.1	NV	NP			70.0 Sand and Gravel; fine to medium grained, dark sand (0.3') over tan, coarse sand and gravel. Gravel of varying angularity. +1 1/2" gravel in shoe. *11:50 am* *Cuttings sandier, with angular fragments of quartzite. Slower drilling, but not necessarily harder - sand? sandier?*	6090
	75															

(Continued Next Page)

WYOMING DEPARTMENT OF TRANSPORTATION

LOG OF BORING

BORING NO. **ST18-01**

PAGE 4 OF 4

PROJECT ____ STATION ____ NORTHING ____ LATITUDE 43.49939

LOCATION Snake River Bridge Jackson- Wilson OFFSET ____ EASTING ____ LONGITUDE -110.8401

DATE STARTED 09/25/2018 DATE COMPLETED 09/25/2018 SURFACE ELEVATION 6161.175 ft TYPE ST TOTAL DEPTH 87 ft

DRILLER Authentic Drilling GEOLOGIST T. Ree ▽ AT TIME OF DRILLING 6156.175 (5 ft)

DRILL RIG CME 550 ALKALI SAMPLES Yes ▼ AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)
	75												
ODEX	80	Geo 08			010-20-45 (65)		SP	A-3		23.8	NV	NP	
		Geo 08B					SP	A-1-a		9.6	NV	NP	
	85												

MATERIAL DESCRIPTION

Unified (Visual) Classification System for Soil

Graphic Log based on Unified (Visual) Classification

(Consistency, Color, Soil Type, Degree Saturation)

ELEVATION (ft)

Sand and Gravel; fine to medium grained, dark sand (0.3') over tan, coarse sand and gravel. Gravel of varying angularity. +1 1/2" gravel in shoe. *(continued)*

6085

Sandier, fine to medium grained, dark, hammer clogging causing drill to advance slower.

80.0

Hard last 0.2 of drive 1.1 ft of black, fine to coarse sand, over broken fragments of white quartzite. Sand is angular to subangular.

6080

12:53 pm - Bagged 2 samples. Geo 08A 80-81.1 Geo 08B 81.1-81.5. Bottom of casing was soft and probably full of sand when setting spoon. Driller tapped spoon 5 times to seat in material. Material collected may not represent material at depth and SPT value may be erroneous. However, rods seemed to be at correct height (relative to casing) before test and spoon was not plugged. Probably OK.

1:20 pm - Drilling slow, but steady. More pressure building downhole. Still sandy.

1:55 pm - Very hard drilling. 11 min/ft. Cuttings show angular fragments of quartxite, metamorphic and igneous rocks with small percentage of sand. Very large (3/4 - 1") fragments blows out of hole after adding another jount. Fragments same rock type.

6075

87.0

2:32 pm Very slow advancement and hard drilling. Possibly due to pressure down hole. Driller tried to clean out/spud hole, but hole is collapsing below bit. He is going to keep trying to get 100 ft, but is not sure if it will happen. A lot of water coming up with cuttings.

Pull casing and inner @ 2:45 pm. May have separated or broke casing. Pull to check.

4:10 pm casing broke at joint on lead casing, 5' from bottom. 10' of casing - 2 joints - destroyed.

Bottom of borehole at 87.0 feet.

BORING LOG W/ LAB DATA W/O DRIVE POINT - WORKING DATA TEMPLATE.GDT - 10/8/18 07:41 - \\CHEYENNE-HQ2\PROJECTS2\2000058\GE\DRAFT\2000058_WILSON BRIDGE.GPJ

WYOMING DEPARTMENT OF TRANSPORTATION

LOG OF BORING

BORING NO. **ST18-02**
PAGE **1** OF **3**

PROJECT ______ STATION ______ NORTHING ______ LATITUDE 43.49961
LOCATION Snake River Bridge Jackson- Wilson OFFSET ______ EASTING ______ LONGITUDE -110.8409
DATE STARTED 09/25/2018 DATE COMPLETED 09/26/2018 SURFACE ELEVATION 6160.528 ft TYPE ST TOTAL DEPTH 69.2 ft
DRILLER Authentic Drilling GEOLOGIST T. Ree ▽ AT TIME OF DRILLING 6156.528 (4 ft)
DRILL RIG CME 550 ALKALI SAMPLES ______ ▼ AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
5.5 - Inch ODEX	0															6160.528
															Cobbles up to 10', gravel, and sand at surface. Quartzite, tufts, gneiss with minor amounts of other igneous and metamorphic rocks at surface of channel. *5:30 pm*	6160
	5														▽ *5:35 pm - Groundwater*	6155
															9.2	
	10	Geo 09			05-6-3 (9)										+1" gravel with medium to coarse grained sand. *5:42 pm - not enough sample to bag.* *6:00 pm - Stop for day. Resume in morning.* *9/26/2018 8:05 am - Start drilling. Impossible to see cuttings with foam.*	6150
ODEX	15															6145
															19.2	
	20	Geo 10			023-26-30 (56)		GP	A-1-a		5.6	NV	NP			Sand and broken pieces of quartzite, probably from first 0.5' of drive. Material at bottom of hole probably cobbles with medium to coarse grained sand. *8:20am*	6140
	25														*Drilling very fast and smooth - cobbles.*	

(Continued Next Page)

PROJECT ______ STATION ______ NORTHING ______ LATITUDE 43.49961

LOCATION Snake River Bridge Jackson- Wilson OFFSET ______ EASTING ______ LONGITUDE -110.8409

DATE STARTED 09/25/2018 DATE COMPLETED 09/26/2018 SURFACE ELEVATION 6160.528 ft TYPE ST TOTAL DEPTH 69.2 ft

DRILLER Authentic Drilling GEOLOGIST T. Ree ∇ AT TIME OF DRILLING 6156.528 (4 ft)

DRILL RIG CME 550 ALKALI SAMPLES ______ ▼ AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
ODEX	25														Sand and broken pieces of quartzite, probably from first 0.5' of drive. Material at bottom of hole probably cobbles with medium to coarse grained sand. *(continued)* 29.2	6135
	30	Geo 11			020-28-29 (57)		GP	A-1-a		5.9	NV	NP			Drill cuttings in spoon: +1" angular quartzite gravel, with black sand. Material at bottom of hole probably cobbles with sand. *8:39 am - hollow sound when driving spoon. Very fast drilling. ~30 sec/ft, cobbles with small percent of sand.* 39.2	6130
	35															6125
	40	Geo 12			012-20-32 (52)		SP	A-1-a		10.0	NV	NP			Sand and Gravel; medium to coarse grained black and quartz sand with pebbles and gravel upo to 1". Gravel and pebbles are rounded to well rounded. *9:03 am - Sand intruding casing at bottom of hole. Blow out to seat bit before resuming drilling Flowing Sand and Gravel? 39.2-49.2 - ~30 sec/ft* 49.2	6120
	45															6115
	50	Geo			012-12-33										*9:30 am - Bagged as 2 samples. Geo 13A 49.2 - 49.9. Geo*	

(Continued Next Page)

WYOMING DEPARTMENT OF TRANSPORTATION	LOG OF BORING	BORING NO. ST18-02	PAGE 3 OF 3

PROJECT		STATION	NORTHING
LATITUDE 43.49961			
LOCATION Snake River Bridge Jackson- Wilson		OFFSET	EASTING
LONGITUDE -110.8409			
DATE STARTED 09/25/2018	DATE COMPLETED 09/26/2018	SURFACE ELEVATION 6160.528 ft	TYPE ST
TOTAL DEPTH 69.2 ft			
DRILLER Authentic Drilling	GEOLOGIST T. Ree	∇ AT TIME OF DRILLING 6156.528 (4 ft)	
DRILL RIG CME 550	ALKALI SAMPLES	▼ AFTER N/A	

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
	50	13A Geo 13B			(45)		SP GP	A-3 A-1-a		24.9 3.5	NV NV	NP NP			*13B 49.9 - 50.7* *More sand intruding bottom of casing blow out bottom of hole before advancing casing. Blown out material consists of very muddy water with sand and gravel up to 1". Flowing Sand and Gravel?* 0.7' of medium to coarse grained sand overlying 0.7' of rounded to well rounded quartzite pebbles with sand. Top 0.7' may be cuttings *(continued)*	6110
	55														59.2	6105
ODEX	60	Geo 14			014-24-24 (48)		GW	A-1-a		6.2	NV	NP			Sand and Gravel; medium to coarse sand with quartzite and granite up to 1", gravel is subrounded to rounded with some broken pieces. *10:20 am - *Water flowing from casing for approximately 10 seconds after air pressure dissipated. ODD** *Large gravel in cuttings. ~ 45 sec/ft*	6100
	65														69.2	6095

11:00 am - Drill to 69.2 and spud. Lots of water coming from casing and cannot free bit from casing. Borke casing downhole. Pull out and try to retrieve tooling.

12:00 pm - Casing broke at 50ft. Both casings broke near base of threads. I think that the casing is "walking" in sands/flowing sands and flexing at the joint, weakening the steel.

Bottom of borehole at 69.2 feet.

WYOMING DEPARTMENT OF TRANSPORTATION

LOG OF BORING

BORING NO. ST18-03

PROJECT ____ STATION ____ NORTHING ____ LATITUDE 43.49968
LOCATION Snake River Bridge Jackson- Wilson OFFSET ____ EASTING ____ LONGITUDE -110.84121
DATE STARTED 09/26/2018 DATE COMPLETED 09/26/2018 SURFACE ELEVATION 6160.943 ft TYPE ST TOTAL DEPTH 79.2 ft
DRILLER Authentic Drilling GEOLOGIST T. Ree ▽ AT TIME OF DRILLING NATD
DRILL RIG CME 550 ALKALI SAMPLES ____ ▼ AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
5.5 - inch ODEX	0															6160.943
															Small cobbles, up to 6" max, and fine grained sand at surface. *1:45 - Set up to drill* *1:50 pm - 0 @ 4.2'* 4.2	6160
	5														Cobbles with Sand. *Groundwater at 4.2'* *~20 sec/ft* *2:00 pm - 9.2 - Mix pit with foam. Very difficult if not impossible to determine materials when drilling with foam.* 9.2	6155
	10														Cobbles with Sand. Muddy fluid. *2:00 pm - 20 sec/ft* 14.2	6150
ODEX	15														Cobbles with Sand. Muddy fluid. *2:10 pm - 2 min 30 sec/5 ft = 30 sec/ft.* 19.2	6145
	20														Cobbles and sand, very wet, muddy fluid - dark brown. *2:30 pm - 3 min 33 sec/5 ft = 42.6 sec/ft.* 24.2	6140
	25														*2:38 pm - 1 min 44 sec/5 ft = 20.8 sec/ft.*	

(Continued Next Page)

WYOMING DEPARTMENT OF TRANSPORTATION	LOG OF BORING	BORING NO. ST18-03
		PAGE 2 OF 4

PROJECT ____ STATION ____ NORTHING ____ LATITUDE 43.49968

LOCATION Snake River Bridge Jackson- Wilson OFFSET ____ EASTING ____ LONGITUDE -110.84121

DATE STARTED 09/26/2018 DATE COMPLETED 09/26/2018 SURFACE ELEVATION 6160.943 ft TYPE ST TOTAL DEPTH 79.2 ft

DRILLER Authentic Drilling GEOLOGIST T. Ree AT TIME OF DRILLING NATD

DRILL RIG CME 550 ALKALI SAMPLES ____ AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
ODEX	25														Sand and Gravel with Cobbles. Well rounded gravel in cuttings, fluid not as muddy - tan to red-brown. *(continued)* 29.2	6135
	30														Sand and Gravel with Cobbles. very wet, subrounded gravel. *2:45 pm - 2 min 0 sec/5 ft = 24 sec/ft.* 34.2	6130
	35														Sand Gravel and Cobbles; dirty fluid, dark brown, very wet. *2:53 pm - 2 min 48 sec/5 ft = 33.6 sec/ft.* 39.2	6125
	40														Gravel and Cobbles with Sand; tan fluid. *3:02 pm - 3 min 2 sec/5 ft = 36.4 sec/ft.* 44.2	6120
	45														Cobbles w/ Gravel and some Sand; fluid tan to white, very wet. *3:12 pm - 3 min 36 sec/5 ft = 43.2 sec/ft.* *Stiffened Up/ Harder* 49.2	6115
	50														*3:29 pm - 3 min 46 sec/5 ft = 45.2 sec/ft.*	

(Continued Next Page)

WYOMING DEPARTMENT OF TRANSPORTATION

LOG OF BORING

BORING NO. ST18-03

PROJECT ________ STATION ________ NORTHING ________ LATITUDE 43.49968

LOCATION Snake River Bridge Jackson- Wilson OFFSET ________ EASTING ________ LONGITUDE -110.84121

DATE STARTED 09/26/2018 DATE COMPLETED 09/26/2018 SURFACE ELEVATION 6160.943 ft TYPE ST TOTAL DEPTH 79.2 ft

DRILLER Authentic Drilling GEOLOGIST T. Ree ∇ AT TIME OF DRILLING NATD

DRILL RIG CME 550 ALKALI SAMPLES ________ ▼ AFTER N/A

DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
	50														Cobbles with Gravel and Sand; Gravel is subrounded, sand is medium to coarse grained. Used collander to catch sample. *(continued)* 54.2	6110
	55														Cobbles with Sand and Gravel; subrounded to rounded quartzite and granite gravel, medium to coarse sand. Fluid caries in color from white to dark brown. *3:42 pm - 4 min 43 sec/5 ft = 57 sec/ft.* 59.2	6105
	60														Gravel and Cobbles with Sand; subrounded to well rounded gravel with medium to granul;ar sand. Tan to dark brown fluid. *3:55 pm - 4 min 21 sec/5 ft = 52.4 sec/ft.* 64.2	6100
ODEX																
	65														Cobbles with Sand and Gravel; same as above. Much harder drilling. *4:08 pm - 7 min 15 sec/5 ft = 87 sec/ft.* 69.2	6095
	70														Cobbles and Gravel with Sand; cuttings mostly 1/4=3/4" angular fragments, with sand. Drill fluid tan to dark brown. *4:27 pm - 9 min 50 sec/5 ft = 174 sec/ft.* 74.2	6090
	75														*4:50 pm - 14 min 30 sec/5 ft = 174 sec/ft.*	

(Continued Next Page)

WYOMING DEPARTMENT OF TRANSPORTATION

LOG OF BORING

BORING NO. ST18-03

PROJECT ____ STATION ____ NORTHING ____ LATITUDE 43.49968

LOCATION Snake River Bridge Jackson- Wilson OFFSET ____ EASTING ____ LONGITUDE -110.84121

DATE STARTED 09/26/2018 DATE COMPLETED 09/26/2018 SURFACE ELEVATION 6160.943 ft TYPE ST TOTAL DEPTH 79.2 ft

DRILLER Authentic Drilling GEOLOGIST T. Ree AT TIME OF DRILLING NATD

DRILL RIG CME 550 ALKALI SAMPLES ____ AFTER N/A

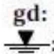
DRILLING METHOD	DEPTH (ft)	SAMPLE TYPE & NUMBER	SAMPLE RECOVERY (ft)	CORE REC % (RQD %)	BLOW COUNTS (N VALUE)	VANE SHEAR (ksf)	USCS	AASHTO CLASSIFICATION	UNCONFINED (ksf)	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION Unified (Visual) Classification System for Soil. Graphic Log based on Unified (Visual) Classification (Consistency, Color, Soil Type, Degree Saturation)	ELEVATION (ft)
ODEX	75														*5:10 pm - Broke Casing when spudding hole.* Cobbles with small amount of Sand and Gravel; white drill fluid, nearly all foam. Cuttings have high percentage of broken fragments and small percent of rounded gravel - some sand. *(continued)* 79.2	6085

Bottom of borehole at 79.2 feet.

GEOTECHNICAL GENERAL NOTES

CORRECTED SPT: Standard Penetration Test values corrected to 60% of the theoretical free-fall hammer energy and for corrected for overburden pressure per Liao and Whitman (1986).

DRILLING, SAMPLING, AND SOIL PROPERTIES ABBREVIATIONS AND SYMBOLS

N: Standard Penetration Test
U_c: Unconfined compressive strength, Pounds/ft² (PSF)
Pp: Pocket Penetrometer values, Ton/ft² (TSF)
FILGC: Fragments indicate gravels and cobbles larger than split spoon diameter.
w: Water content, %
LL: Liquid limit, %
PI: Plasticity index, %
gd: In-situ dry density, lbs/ft³ (PCF)
**:** Ground water level
SS: Split-Spoon Sample
ST: Shelby Tube Sampler
CS: Cylindrical Brass Lined Sample
 Monitoring Well, diagonal hatching indicates screen and sand packed interval

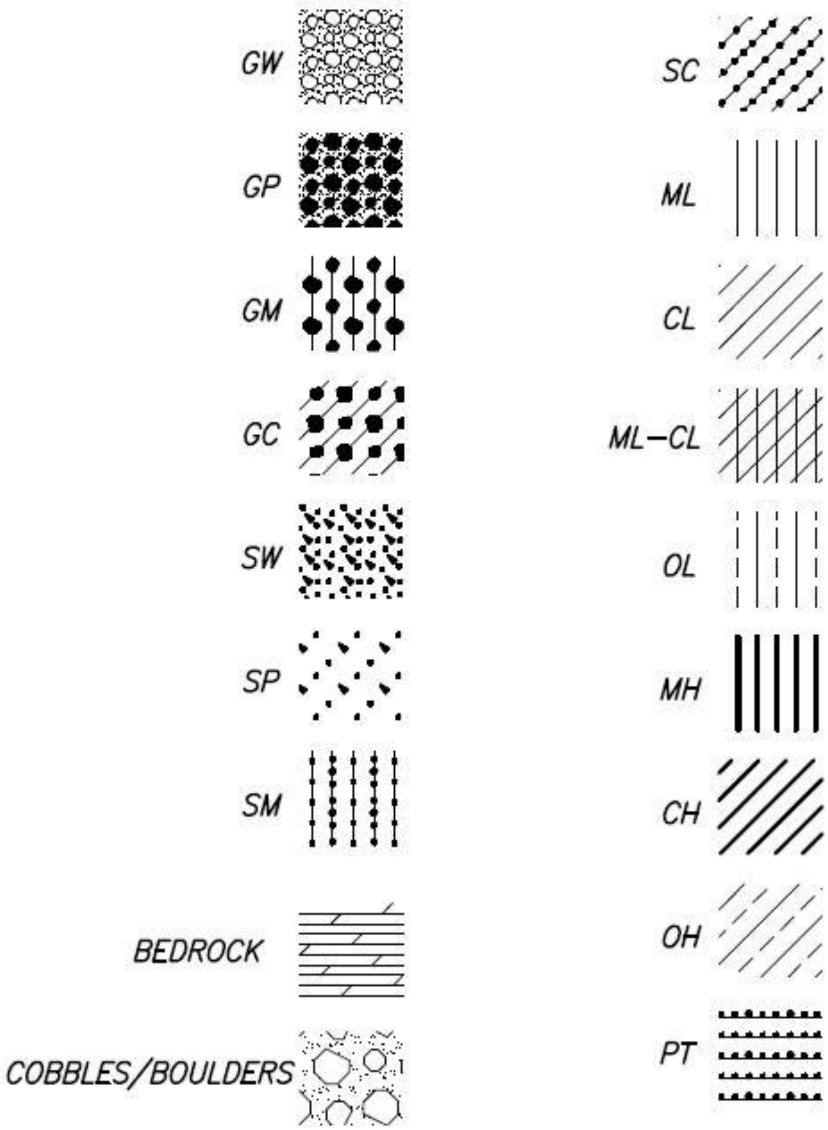
SOIL RELATIVE DENSITY AND CONSISTENCY CLASSIFICATION

Non-Cohesive Soils	Standard Penetration Resistance	Cohesive Soils	Pp-(tons/ft²)
Very Loose	0 - 4	Very Soft	0 - 0.25
Loose	4 - 10	Soft	0.25 - 0.50
Slightly Compact	8 - 15	Firm (Medium)	0.50 - 1.00
Medium Dense	10 - 30	Stiff	1.00 - 2.00
Dense	30 - 50	Very Stiff	2.00 - 4.00
Very Dense	50+	Hard	4.00+

PARTICLE SIZE

Boulders:	12 in.+	**Coarse Sand:**	5 mm(#4)-2 mm(#10)	**Silts and Clays:** <#200
Cobbles:	12 in.-3in.	**Medium Sand:**	2 mm(#10)-0.4mm(#40)	
Gravel:	3in.-5mm(#4)	**Fine Sand:**	0.4mm(#40)-0.075mm(#200)	

SOIL GRAPHICS



NOTE: ANGLED DEMARCATIONS ON THE LOGS INDICATE APPROXIMATE OR POORLY DEFINED BOUNDARIES BETWEEN SOIL TYPES.

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PROJECT NAME: HWY 22 BIKE PATH BRIDGE	DRILL HOLE No. BH−4 PAGE: 1 OF 2
DATE STARTED / FINISHED: 7/17/2012 7/18/2012	DRILLER: AXIS DRILLING
LOGGED BY: PRUETT	DRILL TYPE: DAVEY−KENT AIR ODEX W ROCK HAMMER
BOREHOLE LOCATION/ELEVATION: WEST ABUTMENT/SEE MAP	HOLE DIAMETER: 4.5" OD
	HAMMER TYPE: 140# CATHEAD

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WELL LOG	GRAPHIC LOG	DEPTH (FT)	SAMPLES: DRIVE	SAMPLES: UNDISTURBED	SAMPLES: BULK	SAMPLE ID	RECOVERY (%)	MATERIAL DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	CORR. SPT	DRY DENSITY (PCF)	MOISTURE (%)	REMARKS
		0–5						LARGE GRAVEL AND COBBLE WITH SAND GROUND SURFACE						WEST LEVEE ABUTMENT SLOW HARD DRILLING ON COBBLE FROM 2'−3' D.R. MODERATE TO DIFFICULT DRILLING IN COBBLES TO 5'
		5–6.5	4 4 5			BH4−1 2" SS	11	5'−6.5' DRY MULTICOLORED GRAVEL, ROUNDED TO ANGULAR GRAVELS, WELL GRADED, FILGC, SLIGHTLY COMPACT, MEDIUM DENSE GROUNDWATER ENCOUNTERED AT 6.5' WHILE DRILLING			12			LOOSE EASY DRILLING FROM 5'−15'. D.R. LOOSE GRAVELS DUE TO WATER FLOW, SAND BEING EJECTED FROM CASING AND GRAVELS ARE PERCOLATING AT BOH
		8–9.5	3 7 4			BH4−2 2" SS	17	8'−9.5' SATURATED BROWN GRAVEL WITH MINOR SAND, ROUNDED TO ANGULAR GRAVELS, WELL GRADED, FILGC, SLIGHTLY COMPACT, MEDIUM DENSE			14			
		10.25–11.75	4 4 4			BH4−3 2" SS	0	10.25'−11.75' NO RETURN, SLIGHTLY COMPACT, LOOSE TO MEDIUM DENSE FROM 10'−12.5', SAND AND MINOR GRAVEL EJECTED FROM CASING			10			
		13–14.5	3 4 4			BH4−4 2.5" SS	17	13'−14.5' SATURATED MULTICOLORED GRAVEL, ROUNDED TO ANGULAR GRAVELS, POORLY GRADED, FILGC, LOOSE			7			D.R. MORE DIFFICULT DRILLING STARTING AT 15' (BOTTOM OF LEVEE FILL)
		15.25–16	28 50/3"			BH4−5 2.5" SS	78	15.25'−16' SATURATED BROWN GRAVEL WITH SAND, FINE TO MEDIUM GRAINED SAND, ROUNDED TO ANGULAR GRAVEL, WELL GRADED, FILGC, VERY DENSE			>50			
		20.25–21.75	7 18 TR			BH4−6 2" SS	50	20.25'−21.75' SAME AS ABOVE, DENSE			39			

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CLIENT: TOJ/JH COMMUNITY PATHWAYS DEPARTMENT
HWY 22 SNAKE RIVER CROSSING
TETON COUNTY, WYOMING

PROJECT NAME: HWY 22 BIKE PATH BRIDGE **DRILL HOLE No.** BH-4

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WELL LOG	GRAPHIC LOG	DEPTH (FT)	SAMPLES: DRIVE	SAMPLES: UNDISTURBED	SAMPLES: BULK	SAMPLE ID	RECOVERY (%)	MATERIAL DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	CORR. SPT	DRY DENSITY (PCF)	MOISTURE (%)	REMARKS
		22												D.R. MODERATE TO EASY DRILLING FROM 20'–25'
		23												
		24												
		25												
		26												
		27												
		28												
		29												MODERATE DRILLING FROM 25'–BOH
		30	10											
		31	15 22			NS 2" SS	17	30'–31.5' SAME AS ABOVE, DENSE			36			
		32												
		33												
		34												
		35												
		36												
		37												
		38												
		39												
		40	9 24			BH4–7 2" SS	56	40'–41.5' 0"–1" SAND HEAVE 1"–10" SATURATED BROWN GRAVEL WITH SAND, ANGULAR TO ROUNDED GRAVELS, WELL GRADED, FILGC, VERY DENSE			>50			
		41	29					BOH–41.5'						
		42												
		43												
		44												

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CLIENT: TOJ/JH COMMUNITY PATHWAYS DEPARTMENT
HWY 22 SNAKE RIVER CROSSING
TETON COUNTY, WY

PROJECT NAME: HWY 22 BIKE PATH BRIDGE	DRILL HOLE No. BH-5 PAGE: 1 OF 2
DATE STARTED / FINISHED: 7/19/2012 7/20/2012	DRILLER: AXIS DRILLING
LOGGED BY: PRUETT	DRILL TYPE: DAVEY-KENT AIR ODEX W ROCK HAMMER
BORKHOLE LOCATION/ELEVATION: RIVER WEST/SEE MAP	HOLE DIAMETER: 4.5" OD
	HAMMER TYPE: 140# CATHEAD

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WELL LOG	DEPTH (FT)	SAMPLES: DRIVE	SAMPLE ID	RECOVERY (%)	MATERIAL DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	CORR. SPT	DRY DENSITY (PCF)	MOISTURE (%)	REMARKS
	0				GRAVEL WITH SAND GROUND SURFACE, FINE TO MEDIUM GRAVELS						WEST BOREHOLE ON RIVER ISLAND
▽	3.5				GROUNDWATER ENCOUNTERED AT 3.5' DURING DRILLING						MODERATE TO DIFFICULT DRILLING THROUGH GRAVELS AND COBBLES WITH SAND FROM 0'-15'
	10	18 / 12 / 19	BH5-1 2" SS	44	10'-11.5' SATURATED MULTICOLORED GRAVELS WITH MINOR SAND, ANGULAR TO ROUNDED GRAVEL, WELL GRADED, FILGC, DENSE			42			10'-17' DRILLING WATER IS OPAQUE BROWN, ABUNDANT SAND EJECTED
	12				C.I. GRAVEL WITH SAND						
	15	7 / 27 / 20	BH5-2 2" SS	39	15'-16.5' SAME AS ABOVE, VERY DENSE			57			SMOOTH DIFFICULT DRILLING FROM 15'-17'
	17										MODERATE DRILLING FROM 17'-18'
	18										DIFFICULT DRILLING FROM 18'-BOH
	20	4 / 31 / 31	BH5-3 2" SS	28	20'-21.5' SATURATED BROWN GRAVEL WITH SAND, ROUNDED TO ANGULAR GRAVEL, POORLY GRADED GRAVEL, WELL GRADED SAND, FILGC, VERY DENSE			>50			

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CLIENT: TOJ/JH COMMUNITY PATHWAYS DEPARTMENT
HWY 22 SNAKE RIVER CROSSING
TETON COUNTY, WYOMING

PROJECT NAME: HWY 22 BIKE PATH BRIDGE — DRILL HOLE No. BH-5 — PAGE: 2 OF 2

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WELL LOG	GRAPHIC LOG	DEPTH (FT)	SAMPLES: DRIVE	SAMPLES: UNDISTURBED	SAMPLES: BULK	SAMPLE ID	RECOVERY (%)	MATERIAL DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	CORR. SPT	DRY DENSITY (PCF)	MOISTURE (%)	REMARKS
		22–24												ABUNDANT CONSTANT WATER FLOW FROM CASING WHILE DRILLING D.R. MAJOR GRAVELS COLLECTING BETWEEN DRILL ROD AND CASING CAUSING DRILL ROD TO LOCK ONTO CASING STALLING DRILLING
		25	4 23 21			BH5-4 2" SS	11	24.5'–26' FRACTURED MULTICOLORED GRAVELS, ROUNDED TO ANGULAR GRAVEL, WELL GRADED, FILGC, DENSE			50			
		30	8 22 28			BH5-5 2" SS	56	30'–31.5' SATURATED BROWN GRAVEL WITH SAND, ROUNDED TO ANGULAR GRAVEL, WELL GRADED, FILGC, VERY DENSE			53			DIFFICULT DRILLING FROM 18'–BOH
		35						C.I. SAME AS ABOVE						
		40	6 11 25			BH5-6 2" SS	100	40'–41.5' 0"–9" SATURATED BROWN WELL GRADED SAND 9"–18" SATURATED BROWN GRAVEL WITH SAND, ROUNDED TO ANGULAR GRAVEL, WELL GRADED, FILGC, DENSE BOH=41.5'			34			20" HEAVE EJECTED BEFORE SS AT 40'

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CLIENT: TOJ/JH COMMUNITY PATHWAYS DEPARTMENT
HWY 22 SNAKE RIVER CROSSING
TETON COUNTY, WY

PROJECT NAME: HWY 22 BIKE PATH BRIDGE	DRILL HOLE No. BH-6 PAGE: 1 OF 2
DATE STARTED / FINISHED: 7/19/2012	DRILLER: AXIS DRILLING
LOGGED BY: PRUETT	DRILL TYPE: DAVEY-KENT AIR ODEX W ROCK HAMMER
BOREHOLE LOCATION/ELEVATION: RIVER EAST/SEE MAP	HOLE DIAMETER: 4.5" OD
	HAMMER TYPE: 140# CATHEAD

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WELL LOG	GRAPHIC LOG	DEPTH (FT)	SAMPLES: DRIVE	SAMPLES: UNDISTURBED	SAMPLES: BULK	SAMPLE ID	RECOVERY (%)	MATERIAL DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	CORR. SPT	DRY DENSITY (PCF)	MOISTURE (%)	REMARKS
		0						GRAVEL WITH SAND GROUND SURFACE, MEDIUM TO COARSE GRAVELS						EAST BOREHOLE ON RIVER ISLAND
		1–2						FROM 0'–3.5' C.I. SAND						EASY DRILLING THROUGH SAND FROM 0'–3.5'
▽		4.5						GROUNDWATER ENCOUNTERED AT 4.5' DURING DRILLING						
		6						FROM 3.5'–10' C.I. GRAVEL WITH SAND						MODERATE DRILLING THROUGH GRAVEL WITH SAND FROM 3.5'–20'
		10	19 50/4"			BH6–1 2" SS	80	10'–10.8' SATURATED BROWN GRAVEL WITH SAND, ANGULAR TO ROUNDED GRAVEL, WELL GRADED, FILGC, VERY DENSE			>50			
		12						C.I. SAME AS ABOVE						SLOWER DRILLING ON COBBLE FROM 12.5'–13'
		15	4 12 13			BH6–2 2" SS	28	15'–16.5' SATURATED MULTICOLORED GRAVELS, ROUNDED TO ANGULAR GRAVEL, WELL GRADED, FILGC, DENSE			31			4" HEAVE EJECTED BEFORE SS AT 15'
		18						C.I. GRAVEL WITH SAND						
		20	9 12 14			BH6–3 2" SS	67	20'–21.5' 0"–6" SATURATED BROWN POORLY GRADED SAND, FINE TO MEDIUM GRAINED 6"–12" SATURATED BROWN GRAVEL WITH SAND, ANGULAR TO ROUNDED GRAVEL, WELL GRADED, FILGC, DENSE			32			11" HEAVE EJECTED BEFORE SS AT 20'

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CLIENT: TOJ/JH COMMUNITY PATHWAYS DEPARTMENT
HWY 22 SNAKE RIVER CROSSING
TETON COUNTY, WYOMING

PROJECT NAME: HWY 22 BIKE PATH BRIDGE

DRILL HOLE No. BH-6

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WELL LOG	GRAPHIC LOG	DEPTH (FT)	SAMPLES: DRIVE	SAMPLES: UNDISTURBED	SAMPLES: BULK	SAMPLE ID	RECOVERY (%)	MATERIAL DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	CORR. SPT	DRY DENSITY (PCF)	MOISTURE (%)	REMARKS
		22												MODERATE DRILLING THROUGH GRAVEL WITH SAND FROM 20'–35'
		23												
		24												
		25	8 13 17			BH6–4 2" SS	39	25'–26.5' SATURATED BROWN GRAVEL WITH SAND, ROUNDED TO ANGULAR GRAVEL, WELL GRADED, FILGC, DENSE			34			
		26												
		27												
		28												
		29												
		30	7 9 11			BH6–5 2" SS	6	30'–31.5' TAN/ORANGE FRACTURED GRAVEL, MEDIUM DENSE			21			15" HEAVE EJECTED BEFORE SS AT 30'
		31												
		32												
		33						C.I. GRAVEL WITH SAND						
		34												
		35	4 10 20			BH6–6 2" SS	17	35'–36.5' FRACTURED MULTICOLORED GRAVELS, ROUNDED TO ANGULAR GRAVEL, WELL GRADED, FILGC, MEDIUM DENSE TO DENSE			30			16" HEAVE EJECTED BEFORE SS AT 35', UNABLE TO EJECT 9" HEAVE
		36												
		37												SLOW HARD DRILLING FROM 35'–40', D.R. CASING HAVING TROUBLE ADVANCING DUE OVERBURDEN PRESSURE
		38												
		39												
		40	3 4 20			BH6–7 2" SS	22	40'–41.5' SAME AS ABOVE, MEDIUM DENSE BOH=41.5'			23			3" HEAVE EJECTED BEFORE SS AT 40', UNABLE TO EJECT 10" HEAVE
		41												
		42												
		43												
		44												

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CLIENT: TOJ/JH COMMUNITY PATHWAYS DEPARTMENT
HWY 22 SNAKE RIVER CROSSING
TETON COUNTY, WY

PROJECT NAME: HWY 22 BIKE PATH BRIDGE

DATE STARTED / FINISHED: 7/20/2012

LOGGED BY: PRUETT

BOREHOLE LOCATION/ELEVATION: EAST ABUTMENT/SEE MAP

DRILL HOLE No. BH-7

PAGE: 1 OF 2

DRILLER: AXIS DRILLING

DRILL TYPE: DAVEY-KENT AIR ODEX W ROCK HAMMER

HOLE DIAMETER: 4.5" OD

HAMMER TYPE: 140# CATHEAD

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WELL LOG	GRAPHIC LOG	DEPTH (FT)	SAMPLES: DRIVE	SAMPLES: UNDISTURBED	SAMPLES: BULK	SAMPLE ID	RECOVERY (%)	MATERIAL DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	CORR. SPT	DRY DENSITY (PCF)	MOISTURE (%)	REMARKS
		0						GRAVEL WITH SAND GROUND SURFACE, TOP OF LEVEE						EAST LEVEE ABUTMENT
		2						C.I. GRAVEL WITH SAND						MODERATE TO DIFFICULT DRILLING FROM 0'-5'
		5	6 16 12			BH7-1 2" SS	22	5'-6.5' MOIST BROWN GRAVEL WITH SAND, ROUNDED TO ANGULAR GRAVEL, WELL GRADED GRAVEL, POORLY GRADED SAND (FINE TO MEDIUM GRAINED), FILGC, DENSE			37			
		7												SLOW DIFFICULT DRILLING FROM 5'-16' THROUGH COBBLES
		7.5	5 17 27			BH7-2 2" SS	39	7.5'-9' 0"-2" DRY LOOSE GRAVEL 2"-7" MOIST BROWN GRAVEL WITH SAND, ROUNDED TO ANGULAR GRAVEL, WELL GRADED, FILGC, VERY DENSE			53			
		10	10 50/3.5			BH7-3 2" SS	67	SAME AS ABOVE, BOUNCING SPOON AT 9", VERY DENSE			>50			
		13	4 27 18			BH7-4 2" SS	22	13'-14.5' SAME AS ABOVE, DENSE			49			
▽		15	4 10 8			BH7-5 2" SS	28	15'-16.5' SAME AS ABOVE, SATURATED, MEDIUM DENSE GROUNDWATER ENCOUNTERED AT 16' WHILE DRILLING			19			
		18												MODERATE DRILLING FROM 16'-25'
		20	7 21 19			BH7-6 2" SS	22	20'-21.5' SAME AS ABOVE, DENSE			40			

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CLIENT: TOJ/JH COMMUNITY PATHWAYS DEPARTMENT
HWY 22 SNAKE RIVER CROSSING
TETON COUNTY, WYOMING

PROJECT NAME: HWY 22 BIKE PATH BRIDGE **DRILL HOLE No.** BH-7 PAGE: 2 OF 2

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WELL LOG	GRAPHIC LOG	DEPTH (FT)	SAMPLES: DRIVE	SAMPLES: UNDISTURBED	SAMPLES: BULK	SAMPLE ID	RECOVERY (%)	MATERIAL DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	CORR. SPT	DRY DENSITY (PCF)	MOISTURE (%)	REMARKS
		22												MODERATE DRILLING THROUGH GRAVEL WITH SAND FROM 16'–25'
		27												MODERATE TO DIFFICULT DRILLING FROM 25'–32'
		30	4 10 7			BH7–7 2" SS	56	30'–31.5' 0"–5" SAT BROWN WELL GRADED SAND 5"–10" SATURATED MULTICOLORED GRAVEL, PREDOMINATELY GRAVELS LESS THAN 1" MAXIMUM DIMENSION, MINOR SAND, ANGULAR TO ROUNDED GRAVEL, FILGC, SLIGHTLY COMPACT, MEDIUM DENSE			15			8" HEAVE EJECTED BEFORE SS AT 30'
		36												VERY SLOW HARD DRILLING FROM 32'–40'
		40	6 10 17			BH7–8 2" SS	39	40'–41.5' SATURATED BROWN GRAVEL WITH SAND, ANGULAR TO ROUNDED CLASTS, WELL GRADED, FILGC, MEDIUM DENSE			22			11" HEAVE EJECTED BEFORE SS AT 40'
		41.5						BOH=41.5'						

NELSON ENGINEERING
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CLIENT: TOJ/JH COMMUNITY PATHWAYS DEPARTMENT
HWY 22 SNAKE RIVER CROSSING
TETON COUNTY, WY