Final Report
WY2302F

Characterization of Soil and Rock for Transportation Infrastructure Using Seismic Methods in Wyoming

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16. Abstract

WYDOT utilizes traditional soil borings, drive point analyses, rock coring methods, and lab investigations to identify and characterize the sub-surface soil. Previous experience with these traditional investigation techniques is good; however, current guidance from FHWA encourages the use on non-traditional investigation techniques. These can include a variety of subsurface methods and tools including seismic testing. This study included the data collection and analyses of compression wave and surface wave testing at nine sites throughout Wyoming. The compression wave testing yielded poor quality data and did not result in an ability to see many layers within the subsurface. However, the surface wave testing resulted in a measure of stiffness at each of the nine sites and was a very good predictor of the bedrock depth at each site, with eight of the nine sites in agreement between subsurface investigation and the pile driving records. Correlations between the shear wave velocity and shear strength, rock quality designation and percentage of rock recovered are very poor and yielded no useable correlation data. Lastly, the boundary between weathered and more competent bedrock could not be determined using any of the seismic methods.

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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)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		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)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		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

TABLE OF CONTENTS

Table of Figures

List of Tables

List of Acronyms

FHWA	Federal Highway Administration
Ft	feet
Hz	hertz
Lb	pounds
MASW	multi-Channel Analyses of Surface Waves
Ms	milliseconds
P-wave	Compression wave testing or Refraction testing
SASW	Spectral Analyses of Surface Wave
Sec	second
SPT	Standard Penetration Test
US	United States
Vp	Compression wave velocity
Vs	Shear wave velocity
WYDOT	Wyoming Department of Transportation

CHAPTER 1 PROJECT MOTIVATION AND BACKGROUND

1.1 Research Goals and Objectives

The Wyoming Department of Transportation (WYDOT) currently uses field techniques such as borings, Standard Penetration Test (SPT), rock coring, and a drive point method to investigate subsurface soils. This data in combination with laboratory testing is used to characterize, and determine soil and rock properties prior to foundation design. These methods are a valuable source of information for WYDOT and have produced bridge foundations that have historically performed well. The goal of this project was to enhance subsurface data using seismic methods and compare the results of the seismic testing to the subsurface and construction data.

The motivation for this research comes from recommendations from the Federal Highway Administration (FHWA), which encourages the use of different and new sub-surface investigation techniques (Rivers and Nichols, 2019). It was hoped that the data collected as part of this research would result in high quality data sets at the nine sites tested, and help with correlations of pile drivability, pile design, drilled shaft design, rock mass designations, and depths associated with the weathered bedrock – unweathered bedrock transition. While quality data sets have been compiled at each of the nine sites, and bedrock depths were comparable between seismic and other methods at most sites, correlations between the shear wave velocity (Vs) and other rock and strength parameters are not evident (see chapter 3). Chapter 1 presents a brief review of soil conditions expected in Wyoming as well as some background information on seismic testing.

1.2 Wyoming Rock Types

The surface geology of Wyoming consists mostly of rock outcrops or shallow soil over rock formations. Figure 1 presents a geologic map of Wyoming, the bright yellow and its muted shades represent quaternary rocks and unconsolidated deposits. Review of Figure 1 reveals that there are few quaternary rocks and unconsolidated deposits throughout the state. WYDOT Geology personnel have indicated that even when quaternary soil is present the deposits are not very thick and overlie poor quality bedrock (Kirk Hood, 2019).

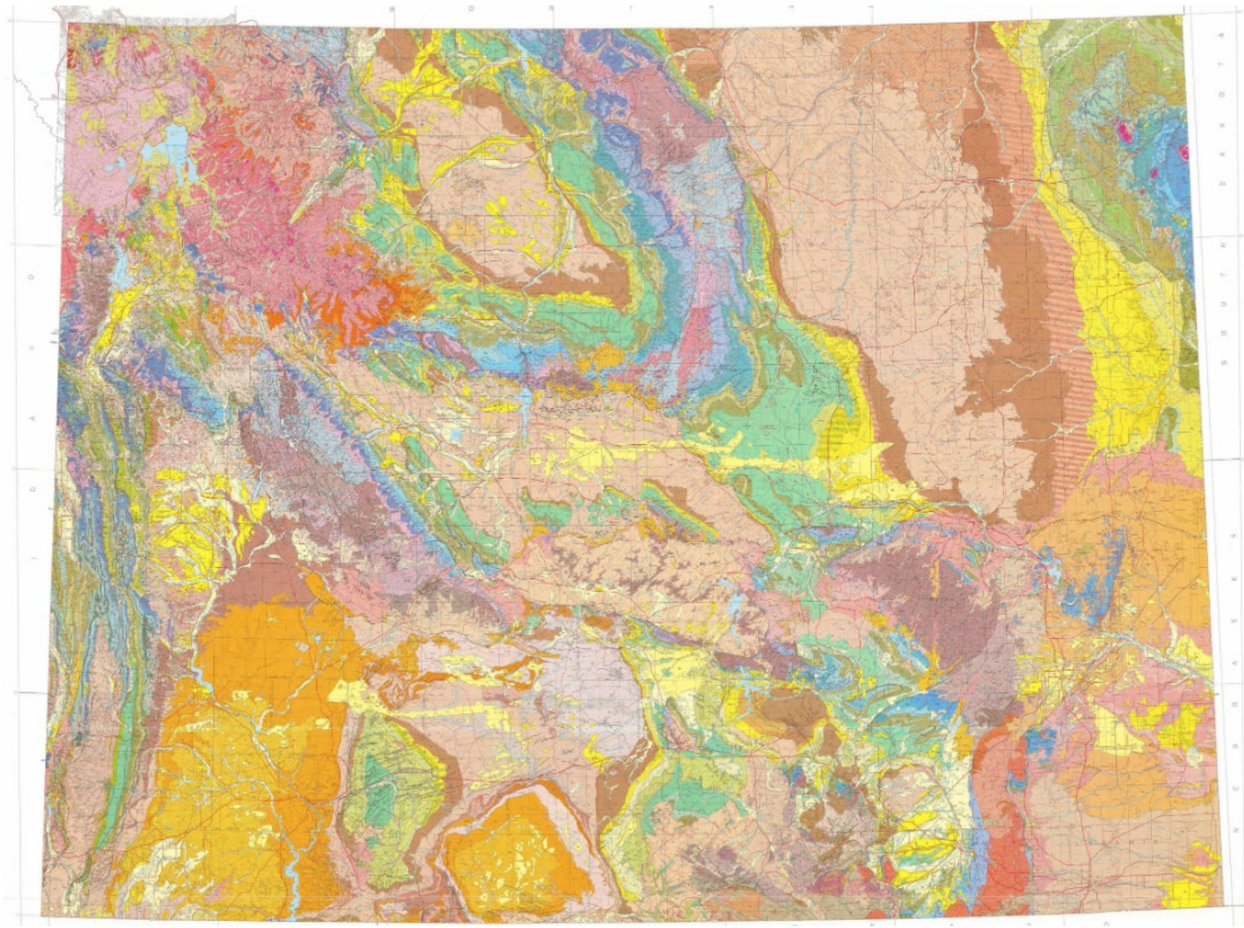

Figure 1 Geologic map of Wyoming, compiled by Love and Christiansen (1985)

The depth of the rock layer beneath the surface is an important design consideration and can be determined accurately over a large area using seismic methods. Rock mass characterization using seismic methods has been performed on a few projects and a variety of rock types (McCann et. al, 1990, El-Naqa, 1996, Yasar and Erdogan, 2004). These studies included using acoustic wave testing in the lab, refraction, and surface wave testing in the field, and some borehole methods. While interesting, this limited amount of information demonstrated that each of the previously published articles was rock type specific and clear correlations for Vs for weak rock material like those found in Wyoming is not present in current literature.

1.3 Seismic Testing

Seismic testing includes surface and borehole methods (Griffiths et. al, 2016). This project will use compression wave refraction and surface wave testing to characterize the subsurface. Both of these methods use the same type of equipment and require analyses of the data after it has been recorded.

Compression wave refraction surveys (P-wave) are common practice in the field of applied geophysics. They can produce a consistent estimate of the velocity and depth of a soft over stiff layer. Figure 2 presents a typical refraction field test. In this method, travel times for each receiver are compared with the distance from the shot location. The arrival times of the geophones nearest the shot location are compared with the arrival times of the geophones further away from the shot locations. Because the velocity of the deeper layers is greater than the surface layer some of the energy will travel through the surface layer, be refracted across the boundary between the surface, and a deeper layer, and then refracted again up toward the surface, and recorded at a geophone before the first arrival travels across the surface layer. This is because the velocity of the deeper layer is much greater than the velocity of the surface. This data is then analyzed to determine travel times, velocities, and depths associated with the various layers. This method requires a large seismic source, or limited background noise (or both) and is dependent on being able to graphically pick the first arrival associated with the source signal. This method was used at each of the nine sites.

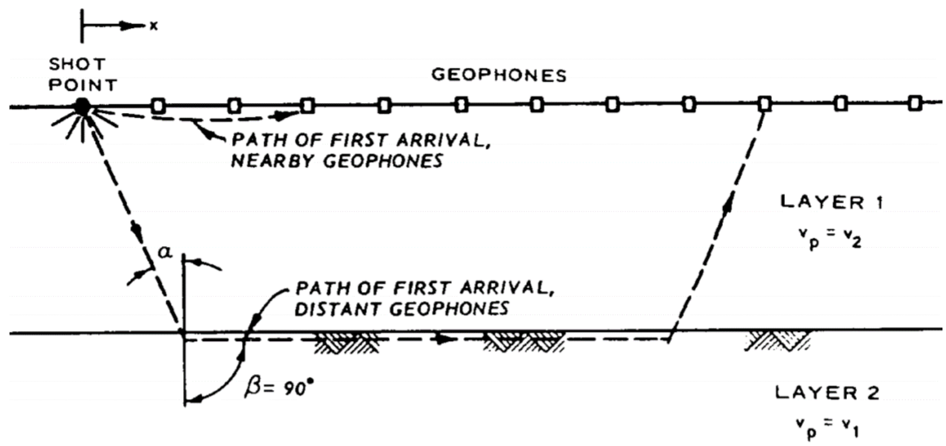

Figure 2 Typical compression wave refraction survey set up, (figure from US Army Corp of Engineers, 1995)

Surface wave methods have been used since the 1980s (Nazarian, 1984, Nazarian et. al 1988) with the advent of the Spectral Analyses of Surface Wave (SASW) method. In this method, two sensors are placed on the ground and the relative phase difference between the sensor readings is used to determine phase velocity. This method was a precursor to the Mutli-Channel Analyses of Surface Waves (MASW) method developed by Park et al. (1999), which is presented as Figure 3. Both of these methods take advantage of the fact that soil and rock are dispersive materials, which means different frequency waves travel at different velocities within the same material. This allows for a determination of the phase velocity and eventual computation of the Vs of a layered earth model. The MASW method requires an inversion analyses of layered earth models, which produce theoretical dispersion curves. These theoretical curves are then compared to the measured in-situ dispersion curve to determine if the layered earth model is a good match, and possible solution to the measured data. These methods are time consuming and require trained personal to develop realistic layered earth models.

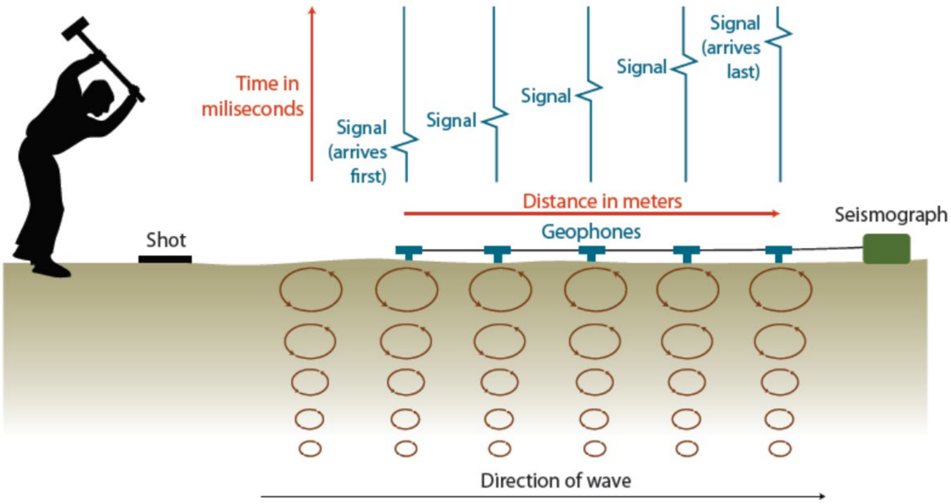

Figure 3 Typical MASW survey set up shown with wave direction, particle motion, and signal times (Ear to the Ground, 2016)

1.4 Research Plan and Goals

The research plan included performing seismic surveys at nine locations within Wyoming, and then comparing that data to the soil borings, drive point, pile driving records, and laboratory rock strengths. The nine sites chosen for testing include those presented in Table 1. The sites were chosen based on availability of data, interesting sub surface conditions, and in coordination with the project champion at WYDOT. At each bridge site, two seismic surveys were completed. Attempts to test in two orthogonal directions and/or opposing abutments were made; however, adjustments to planned surveys were required due to available space, surface conditions, and construction activity. Performing two seismic surveys at each site allowed for redundancy in the data and helped ensure that at least one quality data set was recorded at each site.

Table 1. Nine testing locations for further seismic testing.

S.N.	Location (City)	Site Name
1.	Pine Bluffs	Lodgepole Creek
2.	Pine Bluffs	Parson Street
3.	Pine Bluffs	Beech Ave
4.	Pine Bluffs	Muddy Creek
5.	Rock Springs	Interchange Road
6.	Ranchester	BNSF Rail Road
7.	Gillette	Wildcat Creek
8.	Rawlins	Cedar Street
9.	Thermopolis	Owl Creek

The project goals for this research were to 1) use seismic methods to determine small strain rock and soil properties, 2) correlate the seismic results with other measured soil and rock properties and 3) compare the seismic methods with other rock classification methods. Seismic methods were used to measure Vs at nine sites using both compression wave refraction (P-wave) and

MASW testing methods. For all nine sites Vs data was successfully determined, however, the P-wave data did not produce realistic/expected results. The comparison between the Vs, and other measured soil and rock properties resulted in mostly very poor or non-existent correlations. However, the Vs was a good indicator of competent rock depth. Because correlations between Vs and rock strength yielded such poor quality data, there was no reason to compare this poor quality data with known rock classification methods.

CHAPTER 2 SITE DESCRIPTION, SUBSURFACE INFORMATION AND DATA ANALYSES

2.1 Introduction

Shear wave velocity is a measure of soil stiffness and can be useful for modeling site response analyses, determination of bedrock depth, and site classification. In this chapter a description for each site including; existing borehole, site maps, array locations, and data collection and analyses are provided. A brief comparison between the soil boring, pile-driving record, max case method for pile capacity and Vs is also presented. At sites where some data is not available, comparisons between as much of the data as possible is presented.

Vs compression wave velocity data was collected from nine different bridge sites in Wyoming that were collected during two separate trips made in August 2020. This chapter presents standard information about data collection and analyses used at all sites, and then presents all collected and analyzed information for each site.

Data at each site was collected using similar offsets, spacing, sampling rates, and equipment. Where variations in data collection at any sites were made, they will be noted in the sections below. The data for the MASW (Park et al., 1999) and P-wave were collected using 48, 4.5 Hz vertical oriented geophones spaced at 3.3 ft or 6.6 ft intervals based on space available at each site. Data was recorded using two Geometrics Geode seismographs. The total array length was 154.2 ft or 308 ft. The record length of P-wave refraction testing was 2 sec with a sampling rate of 0.125 ms. A 12 lb. sledgehammer was used as active source with a steel strike plate. A sledgehammer can be used to generate surface wave energy down to frequency of 10-15 Hz and can generate a wavelength long enough to sample the soil to depths of up to 100 ft. When sampling stiff soils longer wavelengths can be generated, resulting in a greater depth of investigation (Wood et al., 2012).

For P-wave refraction, data collection used a trigger switch attached to the hammer with a -0.25 second delay. It was hoped that the P-wave refraction data would allow the researchers to identity compression wave velocity and depth of the first few soil layers, and potentially the transition from weathered bedrock to competent bedrock. However, at most sites the depth of bedrock could not be determined using P-wave refraction testing. At some sights, the lack of sample depth of P-waves is attributed to very noisy (near freeway) environment and at others lack of a strong enough signal. At some sites longer arrays were used, however, due to insufficient source strength and noisy sites, near busy freeways or highways, the P-wave testing did not produce desired results. Therefore, the P-wave data was only used to identify the compression wave velocity of the near surface layers. Keep in mind, unlike MASW testing were a layered earth model is used to determine a theoretical dispersion curve and then compare the theoretical model to the measured dispersion data. Compression wave testing results in a direct measure of the Vp based on the slopes of the lines determined from the first arrival verses the offset distance. Compression wave testing does not require a layered earth model.

MASW testing was performed using the same spacing, geophones, and layout used for the P-wave refraction data collection. The only difference was during the data collection. The trigger delay, recording length, and sampling rate were changed to 0 sec, 4 sec and 4 ms,, respectively.

The data collection for MASW were completed using four shot locations of 16 ft and 32 ft from each end of the array (forward and backward shots). Ten records were stacked at each shot location to minimize disturbance in data due to noise. Data collected from each end were separately analyzed to determine the dispersion curve and the better result of the two data were used for further analysis. MASW data was analyzed using the frequency domain beamformer method (FDBF) (Zywicki, 1999) and the layering ratio method (Cox & Teague, 2016). Geopsy, a free opensource software was used to complete the inversion analyses. Within this software theoretical dispersion curves are compared with experimentally measured dispersion estimates using a misfit value. A lower value of misfit corresponds to a better fit, however because each site in unique, you cannot compare misfit values from one site to another. Instead, the misfit values help rank and compare different theoretical models for the same site.

The rest of this chapter presents the data collection, analysis and comparison at each of the nine bridge sites. With a site presented in each section. In the first section, Lodgepole Creek is discussed in detail and will provide information concerning the data analyses and important data analysis decisions that were made as data was processed. The subsequent sections will provide less detail in terms of data analyses decisions, unless they differ from those in the Lodgepole Creek site, and will instead focus on the unique details associated with the information pertaining to each site.

2.2 Project Site: Lodgepole Creek

Lodgepole Creek, located near Pine Bluffs, Wyoming, was the site of a replacement bridge along State Highway 215, with design and construction overseen by WYDOT. This five span 45 ft long bridge crosses Lodgepole Creek, about 1 mile northwest of Pine Bluffs, Wyoming, is a seasonal drainage river with normally low flows. Based on the existing borehole information, the bedrock under the north abutment was determined to be approximately 50 feet beneath the road surface, and on the south side, approximately 75 ft. Intermediate geomaterial comprised of medium dense to hard sandy siltstone and silty sandstone form the bedrock layer belonging to the Tertiary White River Formation. Layers above the bedrock are generally comprised of moist, loose, silt to clayey sand over saturated loose to medium density silt and sandy silt. Figure 4 shows the soil layers obtained from borehole information (Scott, 2019) as well as the red line which indicates the elevation of the seismic testing which was performed at an elevation of 5039 ft. For our project, data was collected along the north and south abutments about two feet to the east of the road. Figure 5 presents a Google Earth image of Lodgepole Creek with the approximate location of the arrays used for the seismic testing.

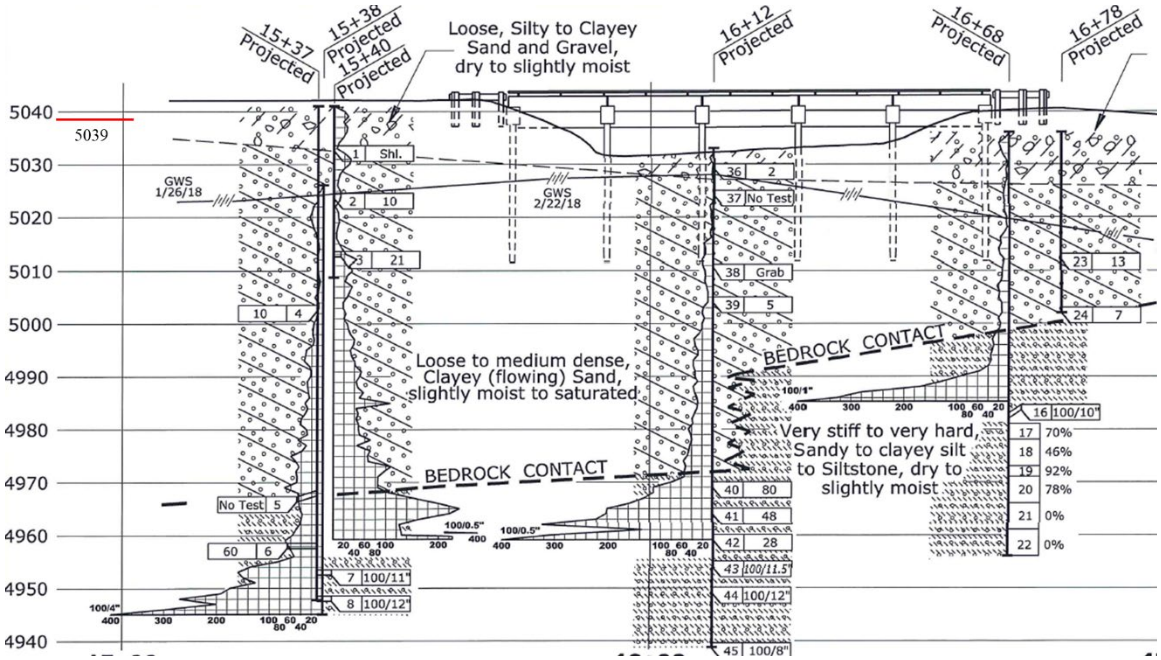

Figure 4 Existing soil profile data for Lodgepole Creek obtained from WYDOT (Scott, 2019), red line indicates elevation of the seismic survey.

Figure 5 Survey location on Lodgepole Creek represented by yellow line (Google Earth Pro, 2021, with alterations)

Data from the north and south arrays was initially analyzed; however, only data from Array 1 (south) is presented herein because it yielded much better results than the data collected on the north side of Lodgepole Creek. This may be due to the presence of a discontinuity in bedrock depths from the north to the south side of the bridge.

P-wave refraction was performed at Lodgepole Creek with results presented in Figure 6, which presents the first arrival times for all sensors with all shot locations. Different layer velocities are shown in the figure by recognizing different slopes between the first arrivals. For Lodgepole Creek, the P-wave refraction analyses yielded the compression wave velocity (Vp) of the top two soil layers. The Vp for the surface layer was determined to be 1121 ft/sec and the layer just beneath this as 3280 ft/sec. The authors had difficulty distinguishing between noise and signal when processing the P-wave data, especially as distance from the shot location increased. This could be overcome in future surveys by using a stronger source and a longer array. Due to the noisy nature of the compression wave data, the Vp determined at the Lodgepole Creek site was not used for any further analyses.

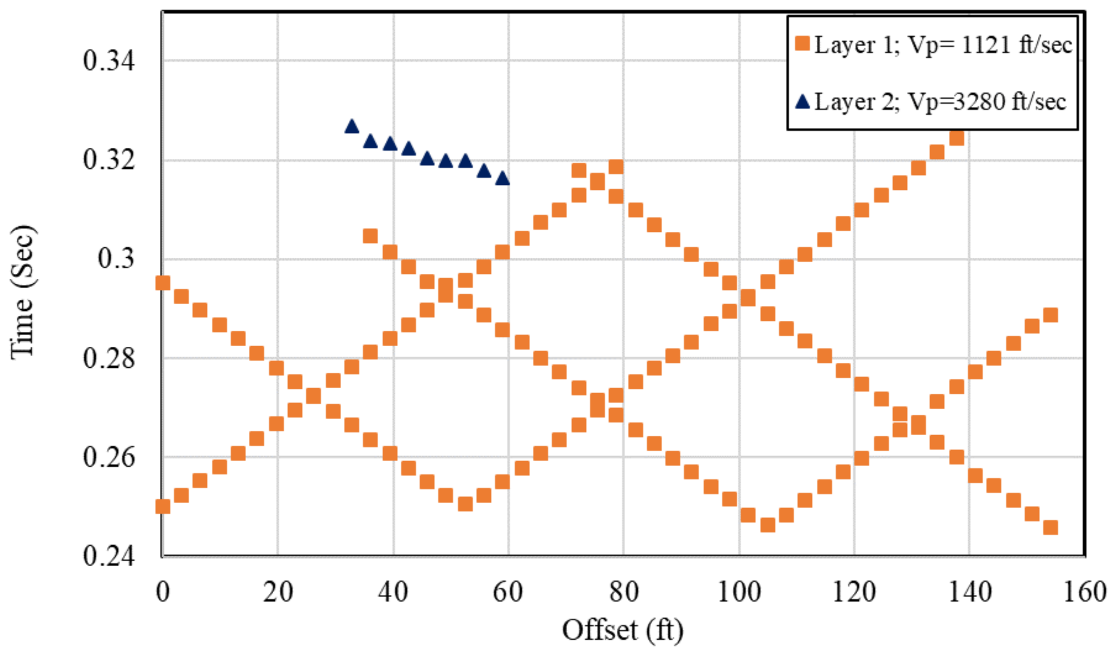

Figure 6 P-wave refraction data for the Lodgepole Creek site

MASW data was measured using an active source at multiple offsets to determine an experimental dispersion curve. The initial data obtained showed a consistent curve from 4-40 Hz as presented in Figure 7. In order to use the data for further analyses, data at very low or high frequencies and sporadic data was trimmed based on recommendations from Wood et. al. in 2012. For the Lodgepole Creek site dispersion data at frequencies less than 4.5 Hz and higher than 40 Hz were removed. The trimmed data is presented in Figure 7.

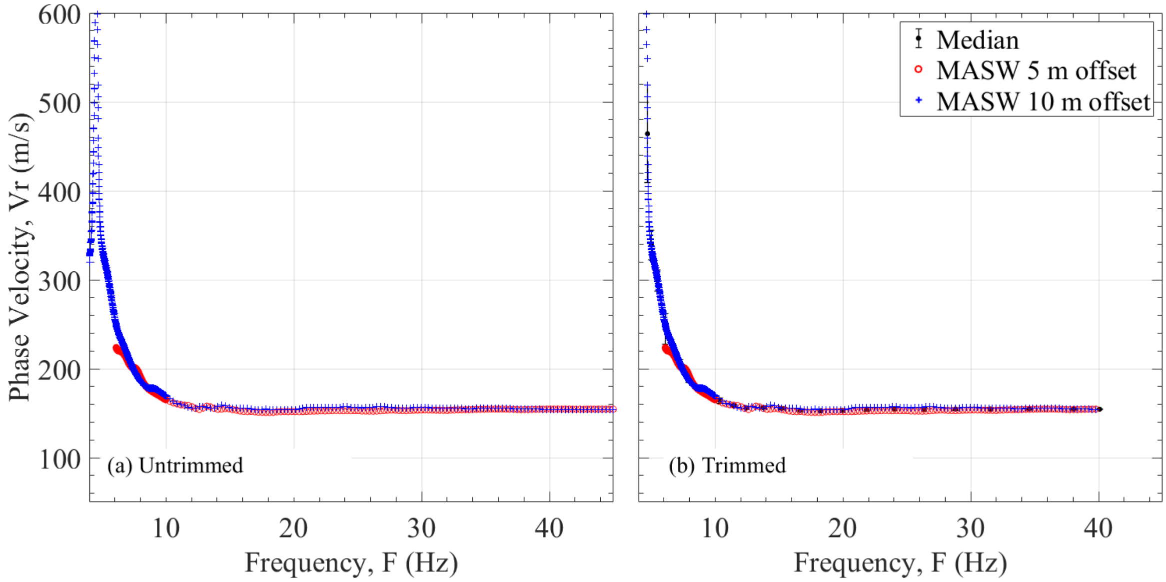

Figure 7 MASW data including (a) Untrimmed and (b) Trimmed measured dispersion data (Array 1) for Lodgepole Creek

Phase velocities at this site range from 150 m/s to 600 m/s. Based on the experimental data the longest useable wavelength measured was 210 ft, which results in a maximum depth of investigation of 105 ft. (Comina et al., 2011). Prior to inversion, the experimentally measured dispersion curve was resampled at pre-determined frequencies ranging from 5-40 Hz.
The inversion was performed using free, open-source software, Geopsy to determine a potential Vs profile. Initial theoretical Vs parameters were determined using the available borehole data, and layering ratio method (Cox & Teague, 2016). At the Lodgepole Creek site, 22 initial models were investigated with over 125,000 inversions performed for each model (total of over 2.7 million models assessed). These inversions are known as theoretical models and are compared with the experimental dispersion curve to determine if they provide a realistic answer.

To aid in the determination of an appropriate answer the theoretical dispersion curve estimates are used to compute a misfit value. With lower misfits corresponding to models with the "better" fit, or inversion that most closely matches the experimental data (Wathelet et. al., 2004). At this site, a five-layer model yielded a minimum misfit of 0.865, which corresponds to a good theoretical fit to the experimental data. This data and the corresponding shear wave velocity profiles are presented in Figure 8. It is important to note that misfit values, while a good comparison between similar Vs profiles for a given theoretical dispersion curve should not be used to compare quality of fit between theoretical and measured dispersion curves from one site to another. In addition, misfit values are not a substitute for a visual inspection of the theoretical and measured dispersion curves.

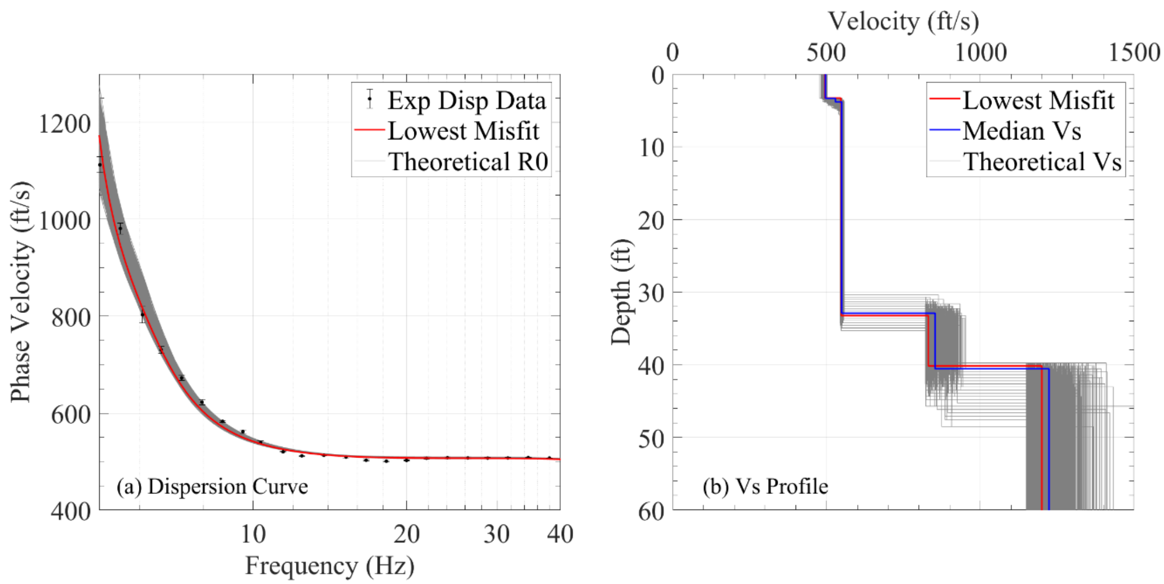

Figure 8 Dispersion curve and Vs profile form the 1000 minimum misfit theoretical data for Lodgepole Creek

In order to characterize some uncertainty within the inversion process, the dispersion curve included uncertainty bounds that were assigned from the experimental data when the phase velocity was resampled. Because of this, and because the authors chose to investigate uncertainty within the solution, a number of satisfactory solutions were determined. This means that a number of Vs profiles provide potential answers to the measured dispersion data. In order to rank them and determine the "best" answer a misfit value was used. Figure 8 presents 1000 Vs profiles and their corresponding dispersion curves that produce reasonably low misfit values. At the Lodgepole Creek site, an increase in velocity with depth is expected based on borehole and drive point data. This was modeled by not allowing velocity reversals during the inversion analysis. Figure 8 includes, the 1000 theoretical Vs profiles with the red line representing the minimum misfit Vs profile. This Vs profile corresponds to the theoretical dispersion curve that most closely matches the experimental dispersion data. The blue line represents the median Vs profile of all 1000 Vs profiles. Based on the minimum misfit Vs profile from Figure 8, the bedrock layer was estimated to be around a depth of 40 ft, with a Vs greater than 1200 ft/sec.

For the Lodgepole Creek site the depth of the bedrock layer as determined by drive point analyses was compared with the Vs profile, as presented in Figure 9. For this comparison, the Vs data was adjusted to the correct elevation, and then existing borehole and drive point data was scaled and plotted next to the Vs data so that side-by-side comparisons could be made. For the Lodgepole Creek site, where Array 1 was used (south of the bridge structure), the Vs profile predicts bedrock depths around an elevation of 5000 ft, which do not match up with borehole data taken near this same abutment of the bridge structure.

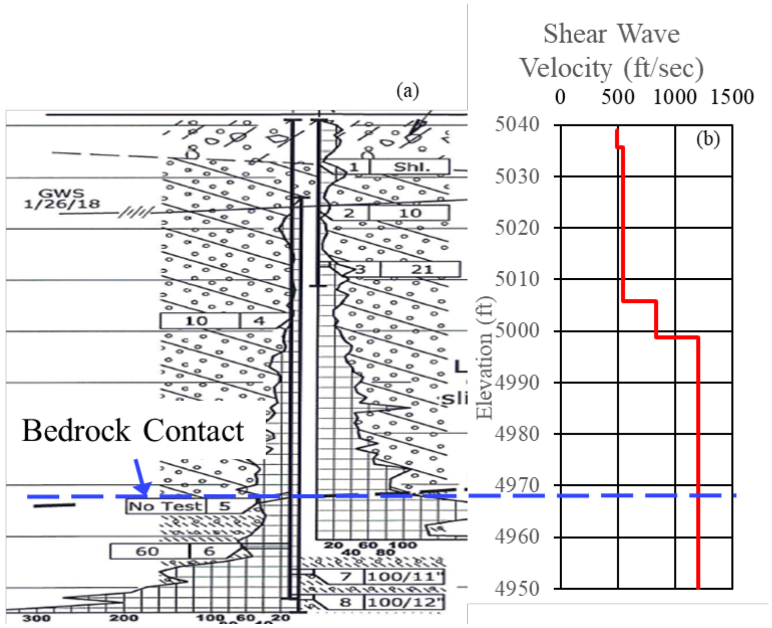

Figure 9 Comparison between soil boring and Vs profile for Lodgepole Creek including a) Soil Boring data and, b) Vs profile for Lodgepole Creek

2.3 Project Site: Parson Street

Parson Street in Pine Bluffs, Wyoming, was a site for construction of 180 ft, 3 span bridge on Interstate 80 over the Parson Street Interchange. Based on the existing borehole data, the subsurface layers are mainly medium dense silty sand with some sandy silt and dense weathered siltstone. The siltstone layer in part of the Tertiary White River Formation. The bedrock elevations were 5032 ft and 5039 ft for Abutments 1 and 2, respectively. Figure 10 shows the subsurface profile which was obtained from WYDOT (Hanson, 2014a). Seismic data was collected at an elevation of 5095 ft at the locations shown in Figure 11. The MASW and P-wave refractions surveys were performed parallel to Parson St. under the overpass in a nearly north south direction and at the base of the embankment nearly perpendicular to Parson Street. in a west-east orientation. Due to data, coupling and quality issues associated with Array 2 only data from Array 1 will be presented and discussed.

P-wave refraction data was not used for the inversion, due to the lack of depth and mis-match between the MASW and P-wave velocities.

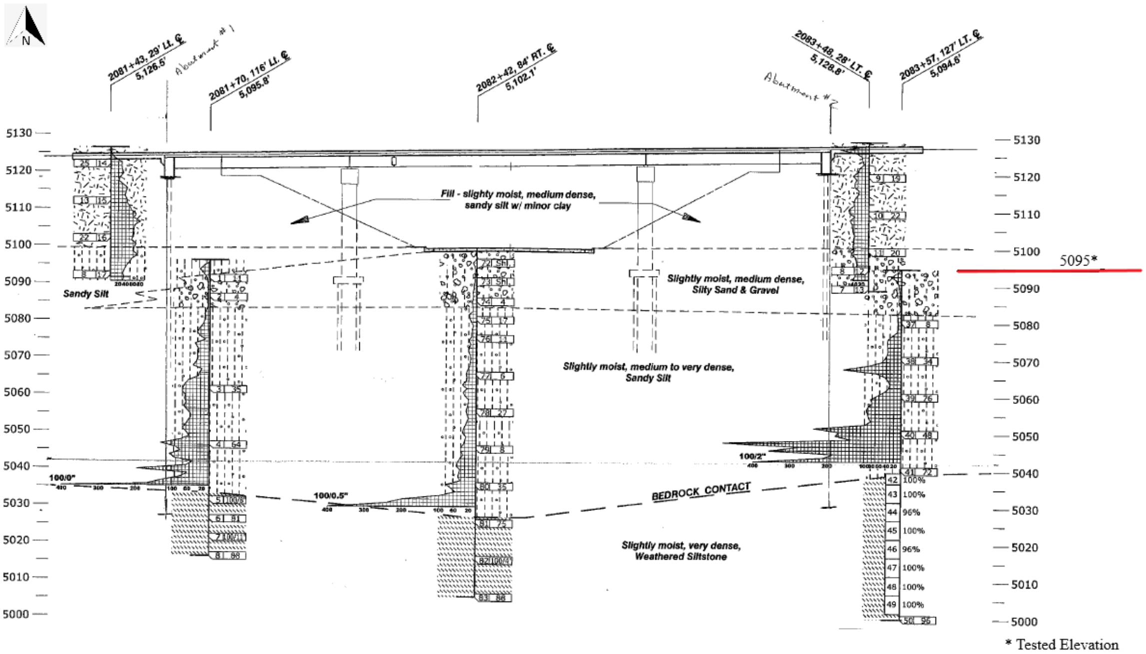

Figure 10 Existing soil profile data for Parson Street Eastbound Lane obtained from WYDOT (Hanson, 2014a), red line indicates elevation of the seismic survey.

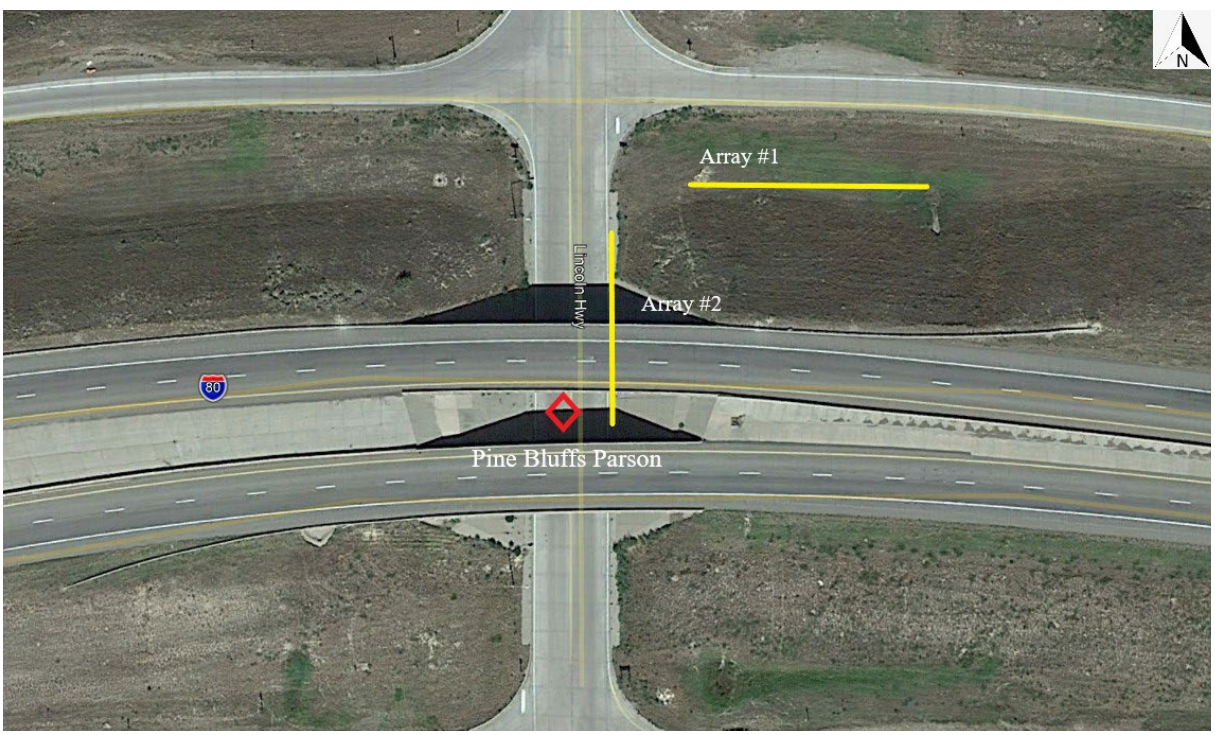

Figure 11 Survey location on Parson Street represented by the yellow lines. (Google Earth, 2021 with alterations)

For completeness the initial arrival times for the P-wave testing is presented in Figure 12. The P-wave refraction analyses yielded the Vp of the top two soil layers. The first layer yielding a Vp = 1263 ft/sec and the second layer yielding a Vp = 2525 ft/sec. The compression wave testing may have produced more model layers if the source had been bigger and the array longer. However, with the equipment used in this study, at the Parson Street Site, only the Vp of the top two layers could be determined.

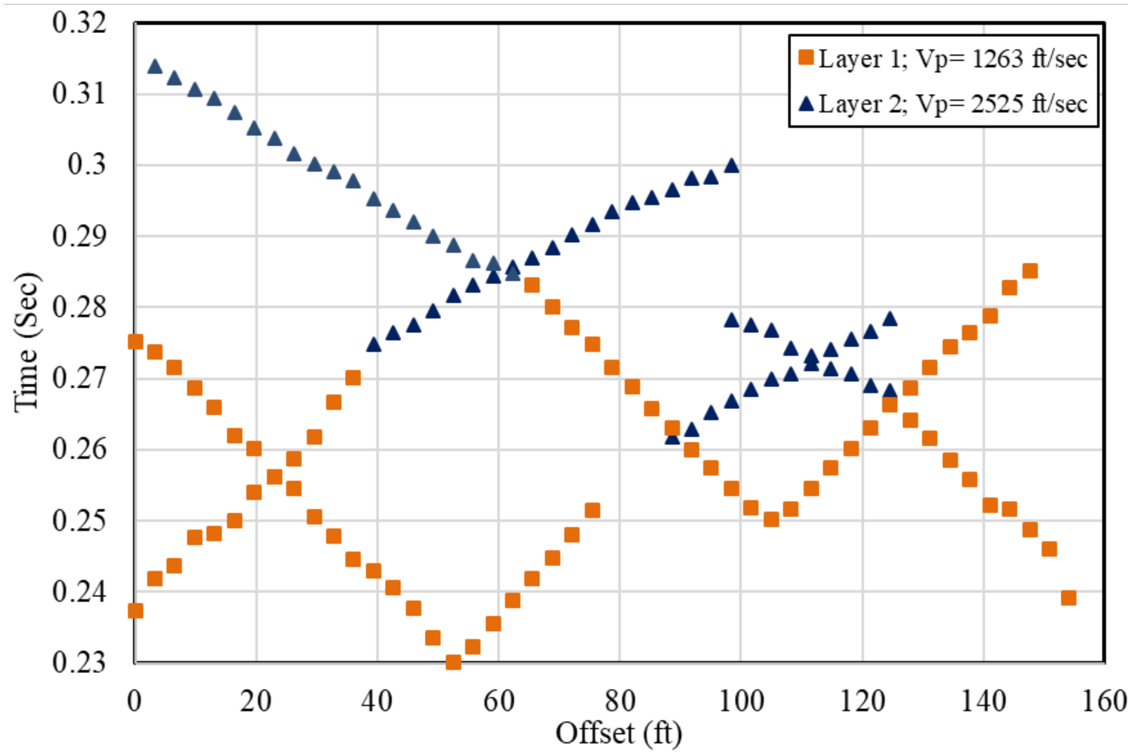

Figure 12 P-wave refraction data for Parson Street

MASW data was measured using an impulse source (sledgehammer) at multiple offsets to determine an experimental dispersion curve. The initial data obtained yielded a constantly increasing phase velocity as frequencies decreased, as presented in Figure 13. At frequencies less than 14 Hz the phase velocity data does not have a clear trend. The Parson Street site experimental dispersion data was trimmed at frequencies less than 10 Hz and higher than 40 Hz, as presented in Figure 13b. The experimental data has a longest useable wavelength of 132 ft that results in a maximum depth of investigation of 65 ft (Comina et al., 2011).

The inversion was performed using free open-source software Geopsy to determine a potential Vs profile. At the Parson Street site, 33 initial models were investigated with over 125,000 inversions performed for each model (total of over 4.1 million models assessed). At this site, a five-layer model yielded a good fit to the experimental data with a minimum misfit of 0.406. The five layer models and their corresponding shear wave velocity profiles are presented in Figure 14.

Uncertainty was accounted for by developing 1000 Vs profiles, and presented in Figure 14. Velocity reversals were not allowed in the inversion analyses. Included in Figure 14 is the Vs profile the lowest misfit value as well as the median Vs profile.

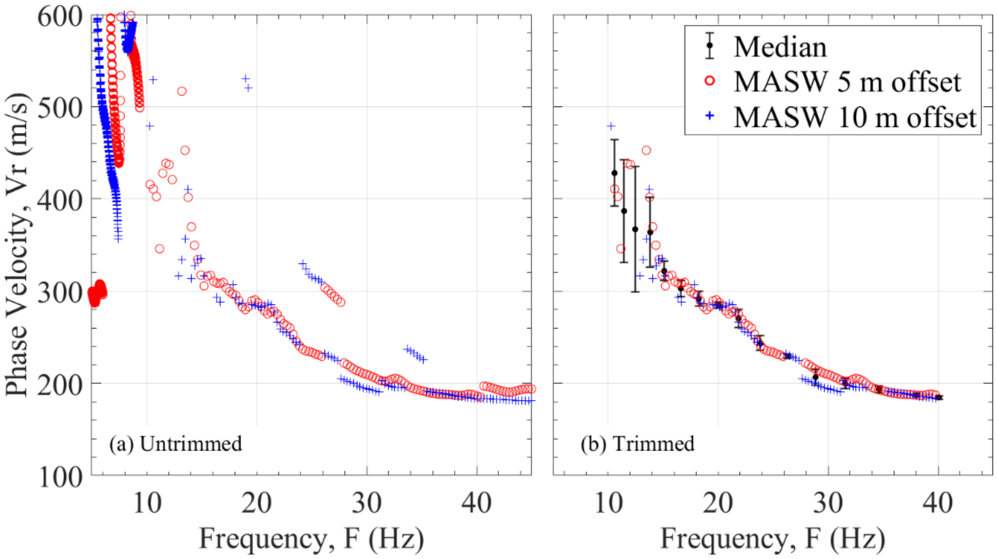

Figure 13 MASW experimental (a) untrimmed and (b) trimmed measured dispersion data (Array 1) for Parson Street

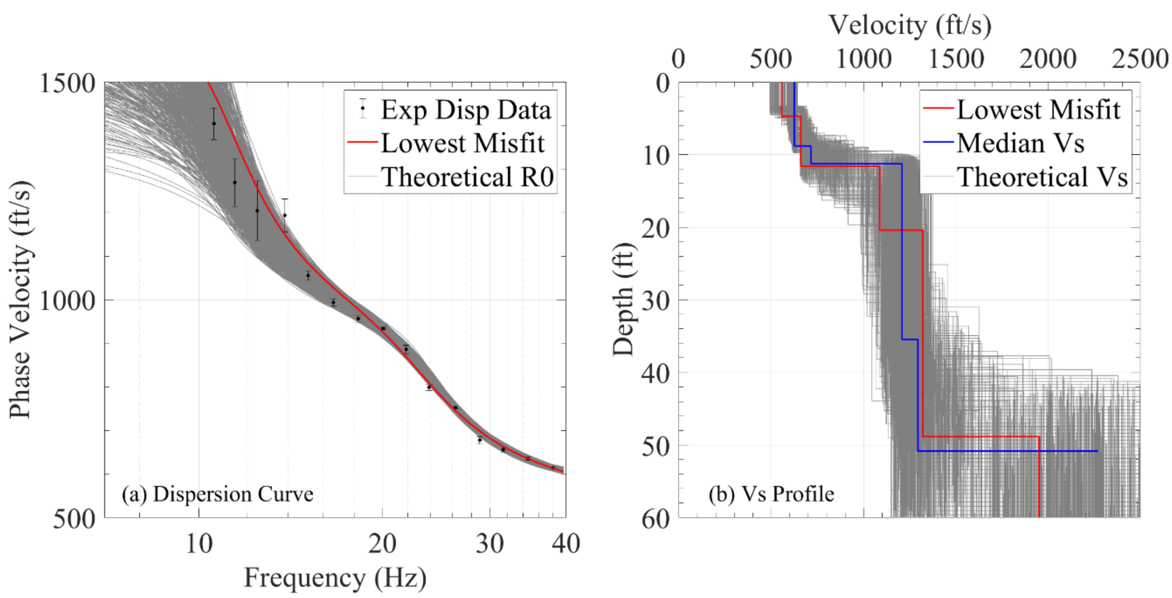

Figure 14 Dispersion curve and Vs profile form the 1000 minimum misfit theoretical data for Parson Street

Based on the minimum misfit Vs profile, a bedrock depth at this site is expected to be around 50 ft below the surface and corresponds to an increase in Vs from 1400 ft/sec to about 1900 ft/sec.

A comparison between the soil boring data, pile driving blow count, maximum case method and Vs data is presented in Figure 15. In order to make this comparison, the Vs data was corrected to its corresponding elevation data. At the Parson Street site, the depth of bedrock predicted by the Vs profile is consistent with the pile driving record and the soil boring information.

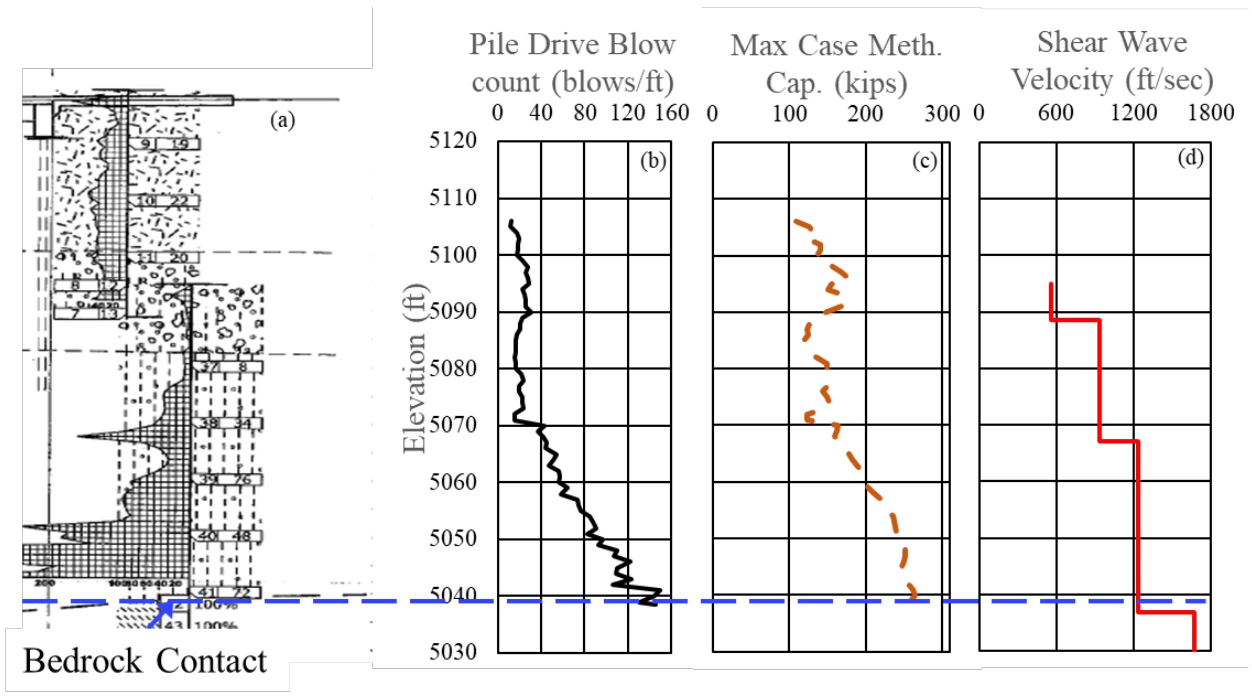

Figure 15 a) soil boring information, b) pile driving blow count, c) max case method capacity and d) Vs profile all as a function of depth for Parson Street

2.4 Project Site: Beech Street

The Beech Street bridges in Pine Bluffs are two approximately 150 ft long, 3 span bridges along I-80 that crosses over Beech Street (east and westbound). Based on the existing borehole information, the subsurface layer near Beech Street mainly consists of medium dense sandy silt from the surface to a depth of approximately 27 feet below the surface as presented in Figure 16. The bedrock layer consists of siltstone and belongs to the Tertiary (Oligocene) White River Formation. At this site, the Vs data was collected at an elevation near 5097 ft in two arrays, which is shown in Figure 16 as a red line. Array 1 was conducted parallel to Beech St. and Array 2 was conducted along the base of the embankment as presented in Figure 17. Due to data coupling and quality issues associated with Array 1 only data from Array 2 (parallel to freeway) will be presented in this report.

P-wave refraction data is presented in Figure 18. The P-wave refraction data analyses resulted in determination of the Vp of the top two soil layers. The first layer has a Vp = 1131 ft/s and the second layer has a Vp = 3647 ft/s. While this data was used as an initial guesses of the Vp used in the MASW inversion analyses for the top two layers, it was not used for any further.

MASW data was measured using and impulse source at multiple offsets to determine an experimental dispersion curve. The initial data obtained showed a smooth curve at frequencies down to 24 Hz. At frequencies less than 24 Hz the data quality was not as good as at higher frequencies. The measured phase velocities are presented in Figure 19, which includes both trimmed 19b and untrimmed 19a data. For the Beech Street site data at frequencies less than 8 Hz and higher than 40 Hz were removed. Phase velocities at this site range from 180 m/s to 420 m/s. Based on the experimental data the longest useable wavelength measured was 135 ft which results in a maximum depth of investigation of 67 ft (Comina et al., 2011). The experimentally measured dispersion curve was resampled at pre-determined frequencies prior to inversion.

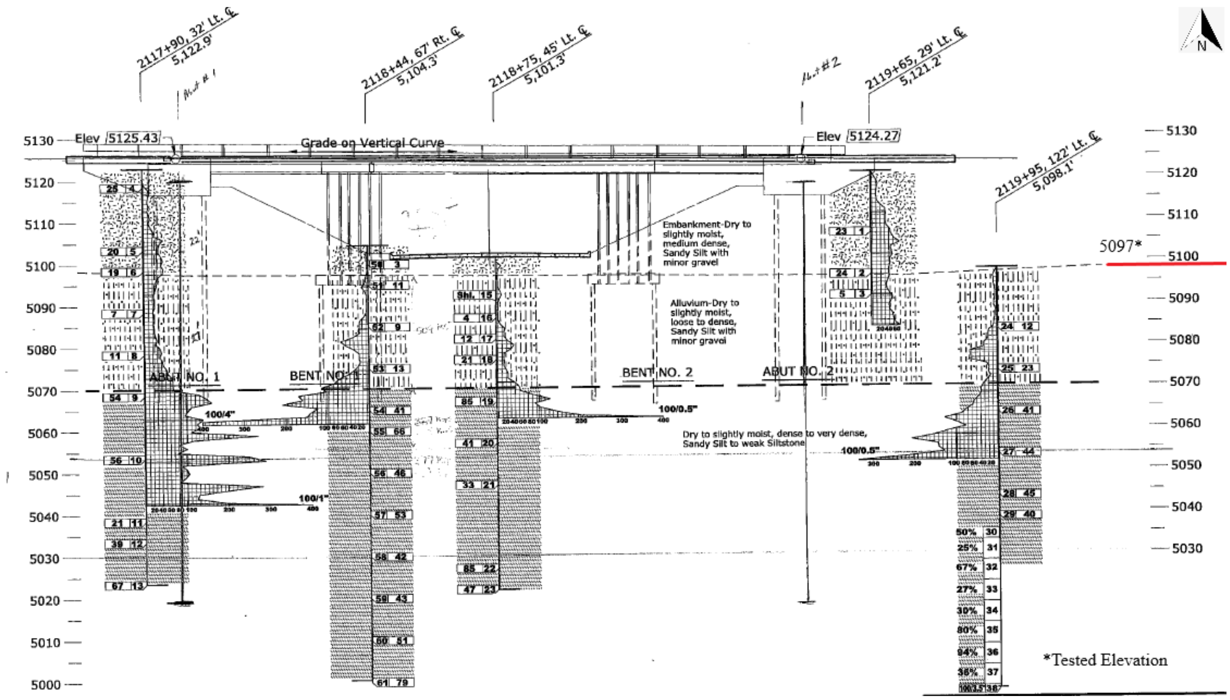

Figure 16 Existing soil profile data for Beech Street Westbound Lane obtained from WYDOT (Green, 2014), red line indicates elevation of the seismic survey.

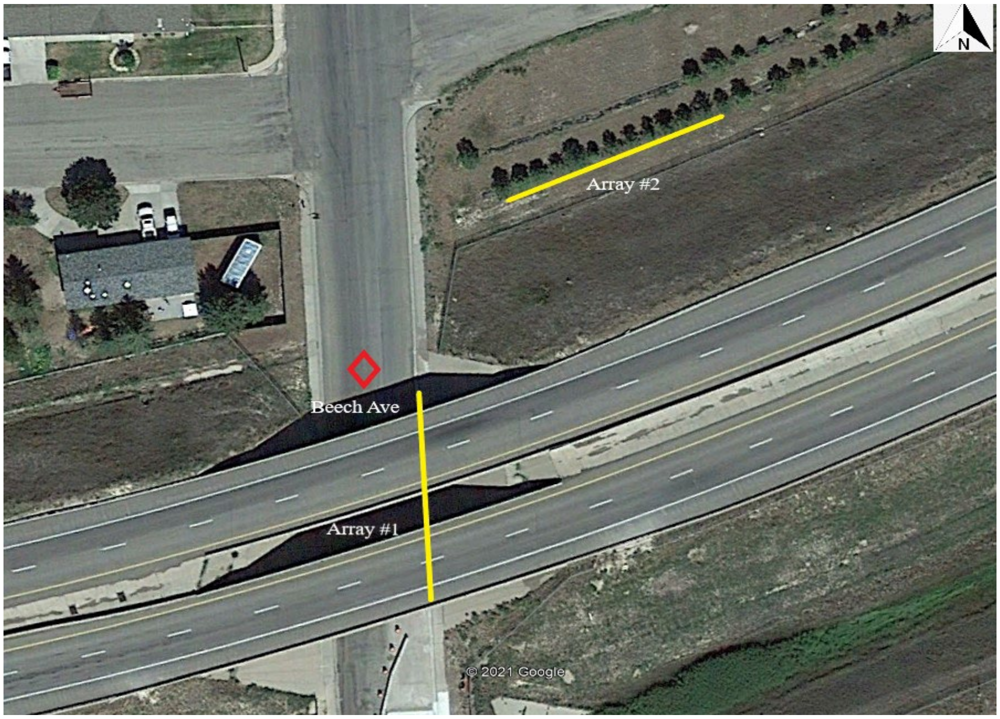

Figure 17 Survey location on Beech Street represented by yellow line (Google Earth, 2021, with alterations)

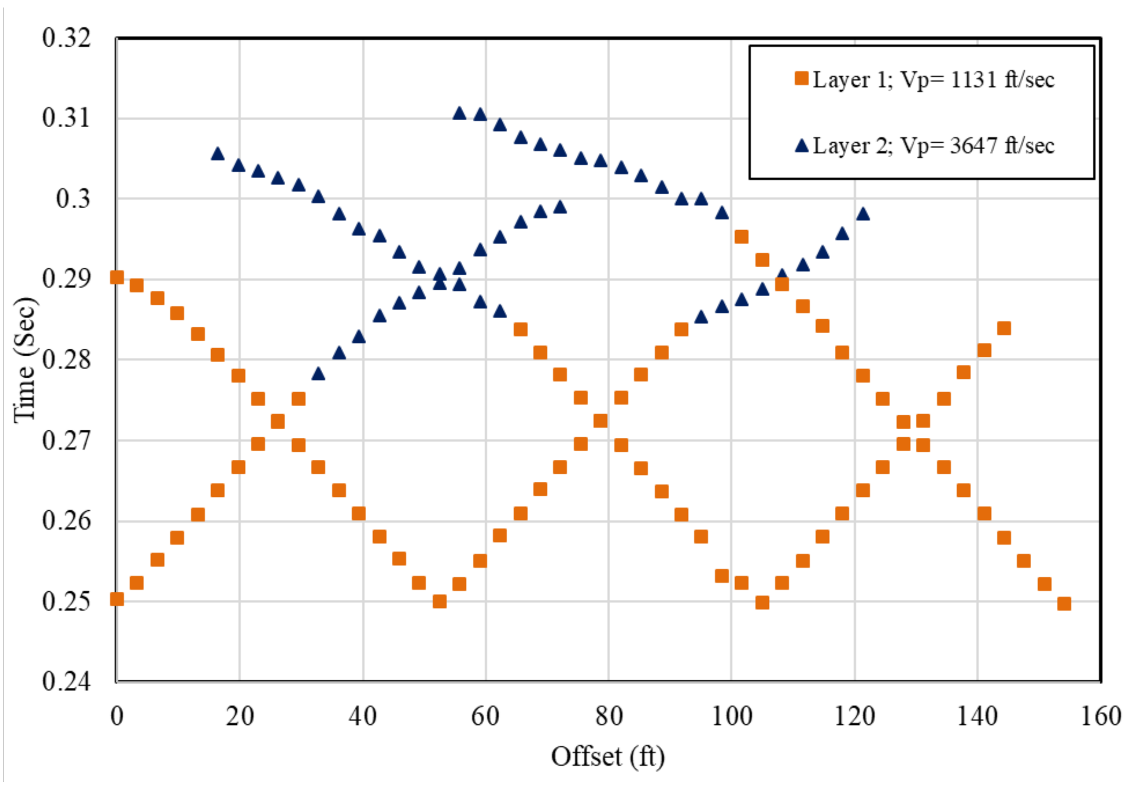

Figure 18.Figure 18 P-wave refraction data for Beech Street

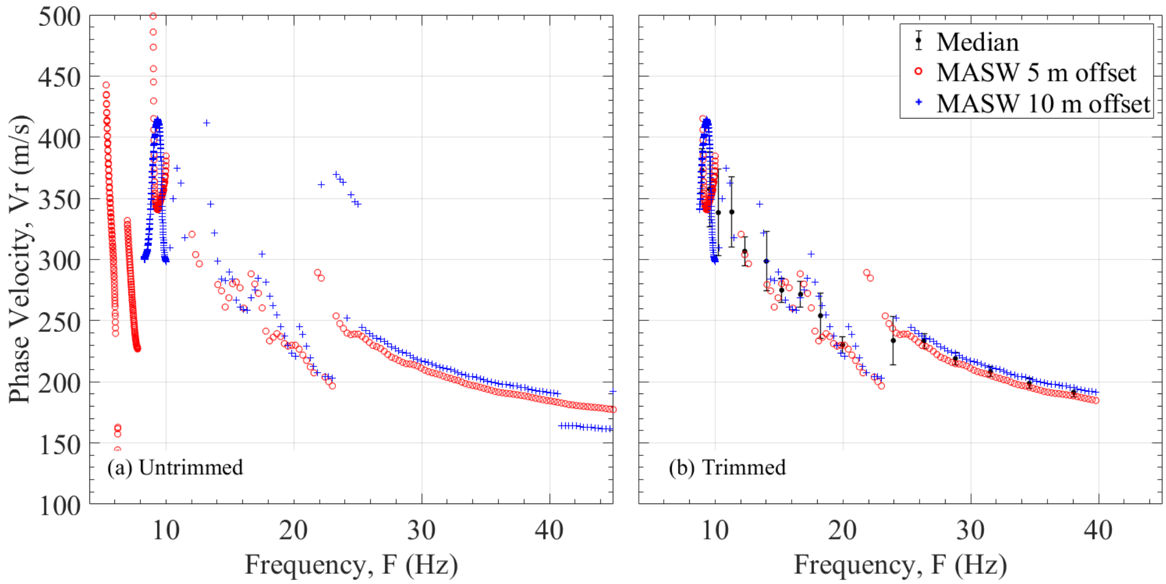

Figure 19 MASW (a) untrimmed and (b) trimmed measured dispersion data (Array 2) for Beech Street

Around a frequency of 22 hz, there appears to be a mode change. Due to this change, a multi-mode inversion was attempted. However, using a multi-mode inversion did not result in a practical solution. So only the results of the fundamental mode inversion are presented here. At the Beech Street site, 19 initial models were investigated with over 125,000 inversions performed for each model (total of over 2.3 million models assessed). At this site, a four-layer model yielded a minimum misfit of 0.673, which corresponds to a good theoretical fit to the experimental data. This data and the corresponding shear wave velocity profiles are presented in Figure 20.

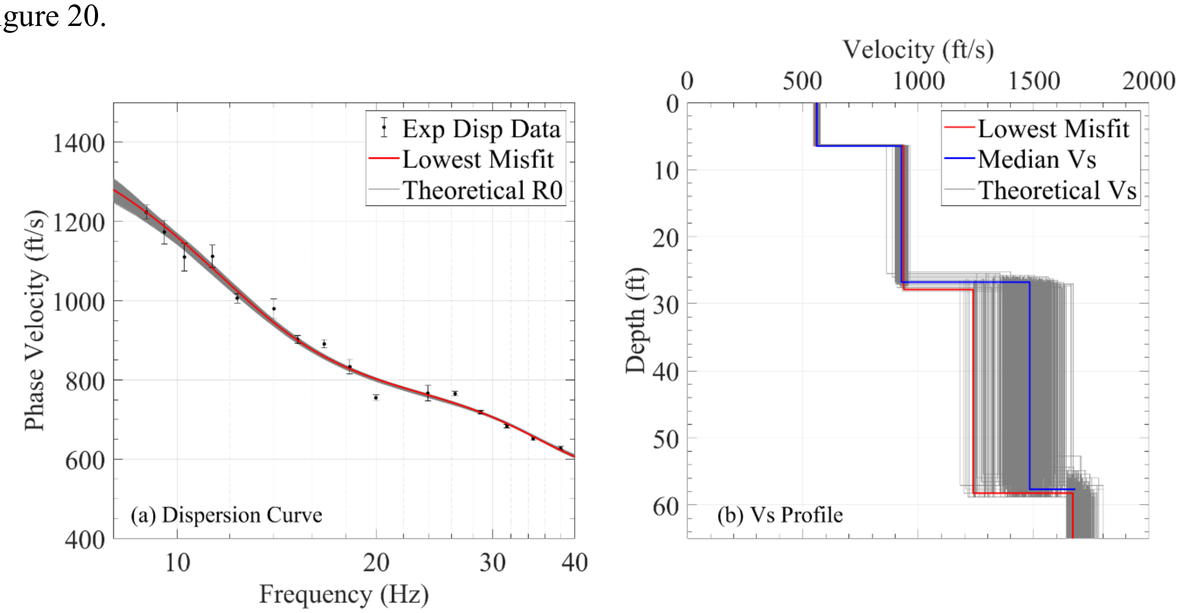

Figure 20 Dispersion curve and Vs profile with 1000 minimum misfit theoretical data for Beech Street

Boring logs at the Beech Street site indicate stiffer soil as depth increases that corresponds to increasing Vs as a function of depth. This was modeled by constraining the analysis to not allow velocity reversals. Based on the minimum misfit Vs profile a bedrock layer occurs at a depth of 29 ft with a Vs of about 1200 ft/sec. A comparison between the soil boring data, pile driving blow count, maximum case method, and Vs data are presented in Figure 21. At the Beech Street site, the depth of bedrock predicted by the Vs profile is consistent with the pile driving record and the soil boring information.

2.5 Project Site: Muddy Creek

The Muddy Creek site in Pine Bluffs, Wyoming includes two 247 ft long 4 span bridges and is part of Interstate 80 spanning muddy creek and an access road (east and westbound). Based on borehole information, the sub-surface layers consist of a medium dense, silty sand, and a dense sandy silt layer overlying very dense to weak siltstone bedrock layer. The siltstone layer belongs to the Tertiary White River Formation. Bedrock layers were identified at an approximate elevation of 5018 ft. Seismic testing was performed at an elevation near 5060 ft as presented in Figure 22 by the red line.

For this project, data collection was performed at two array locations. Array 2 was collected under the bridge, and Array 1 was collected along the Muddy Creek Drive, as shown in Figure 23.

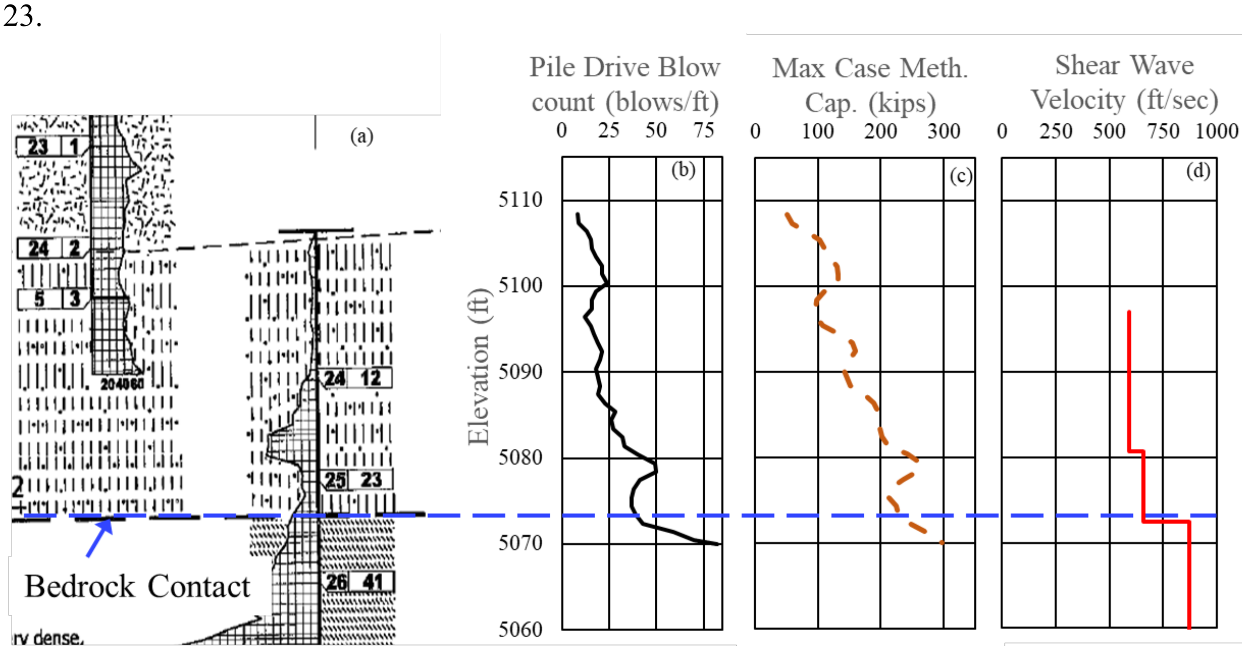

Figure 21 Bedrock depth comparison for Beech Street including a) soil boring information, b) pile driving blow count, c) max case method capacity and d) Vs profile all as a function of depth for Beech Street

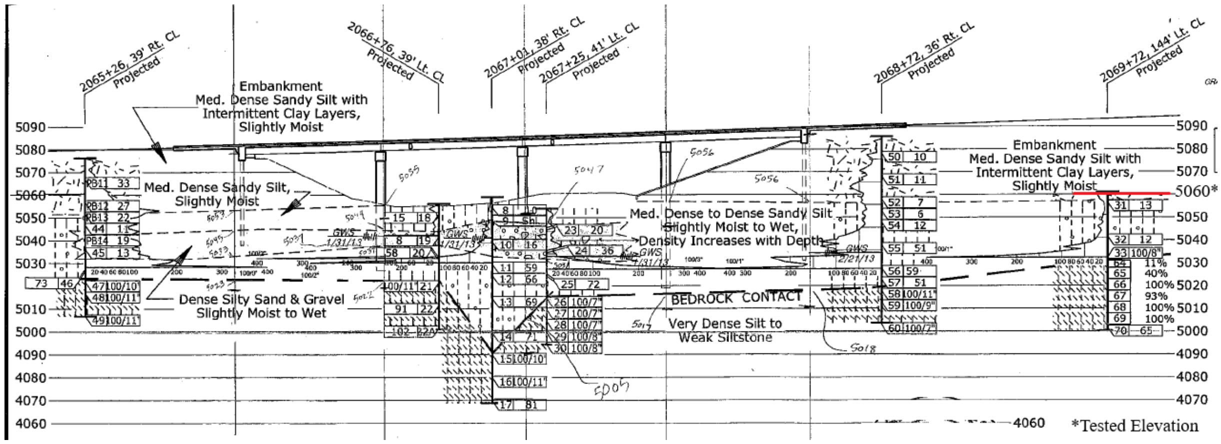

Figure 22 Existing soil profile data for Muddy Creek Westbound Lane obtained from WYDOT (Hanson, 2014b), red line indicates elevation of the seismic survey.

For this project, data collection was performed at two array locations. Array 2 was collected under the bridge, and Array 1 was collected along the Muddy Creek Drive, as shown in Figure 23. Although two arrays were used for data collection, only data for Array 1 will be presented herein.

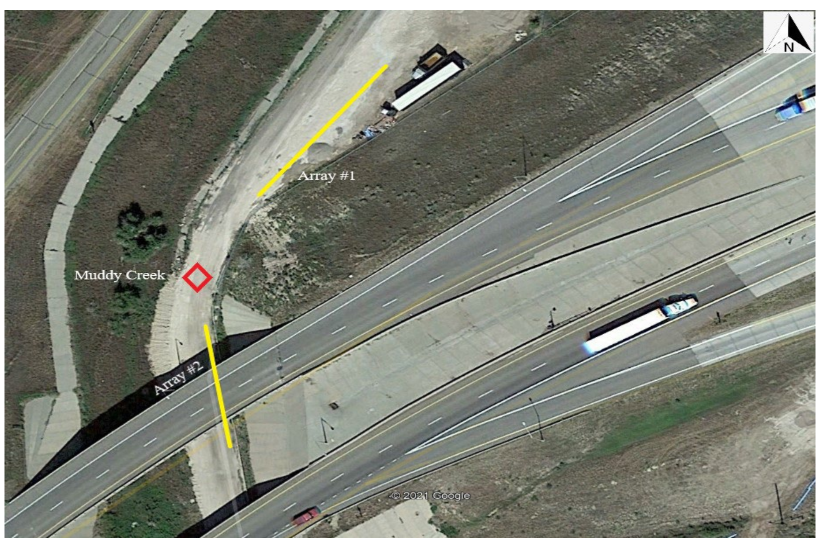

Figure 23 Survey location on Muddy Creek represented by yellow line (Google Earth Pro, 2021, with alterations)

The P-wave refraction data at Muddy Creek is presented in Figure 24. At this site there was enough room to spread out Array 2 and perform the P-wave refraction and MASW testing at a spacing of 6.6 ft. The limited data shown in the P-wave data is due to a very noisy site and lack of source strength. Even with the lack of source strength the top two Vp layers at the site are evidenced by the two distinct slopes presented in Figure 24. While it is almost certain that more than two layers exist in the subsurface the data collected using a sledgehammer source at this noisy site only provided enough signal strength to determine a two layer model using compression wave analyses. Because of this, readers should not compare model layers between the compression wave and MASW testing. Due to the poor quality of P-wave refraction data, it was not used for further analyses.

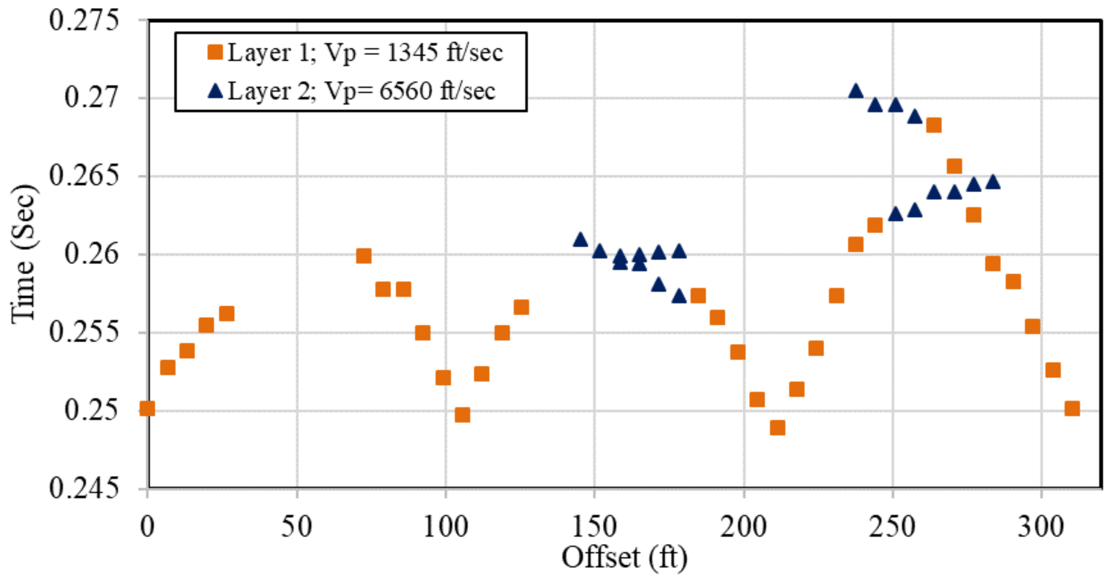

Figure 24 P-wave refraction data for Muddy Creek

The MASW experimental dispersion curve is presented in Figure 25. For the Muddy Creek site, the experimental data was cut-off at frequencies less than 10 Hz and higher than 40 Hz. The trimmed data (Figure 25b) for both offsets are in agreement at all frequencies. Phase velocities at this site range from 180 m/s to 400 m/s. The longest useable wavelength measured was 110 ft that results in a maximum depth of investigation of 55 ft. (Comina et al., 2011).

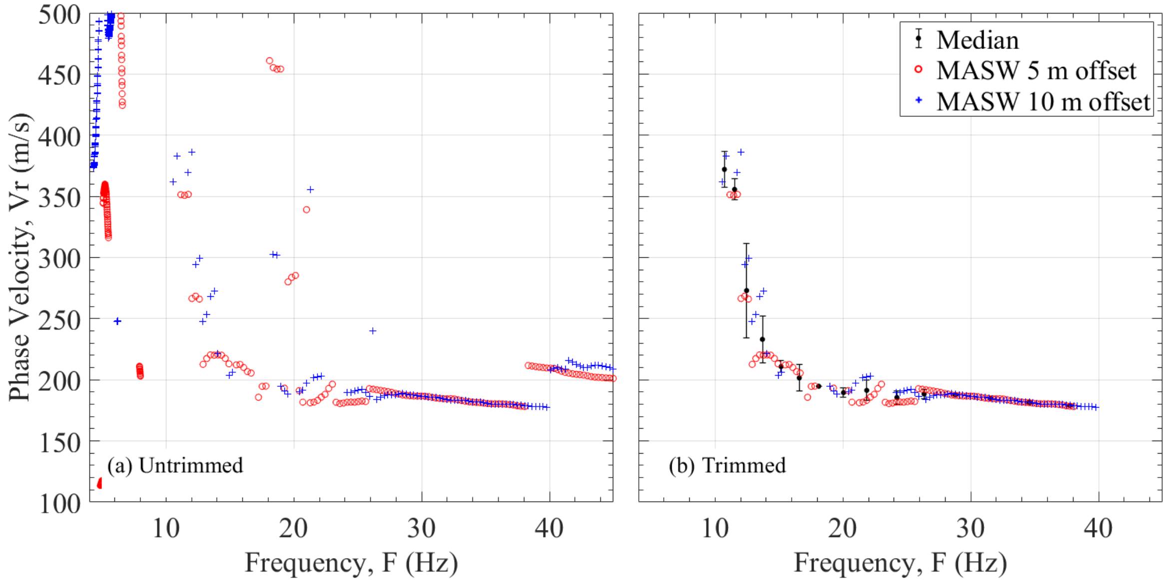

Figure 25 MASW (a) untrimmed and (b) trimmed measured dispersion data (Array 1) for Muddy Creek

The inversion was performed using free open-source software, Geopsy. Initial theoretical parameters used in the inversion were determined using the available borehole data, and layering ratio method (Cox & Teague, 2016). At the Muddy Creek site, 37 initial models were investigated with over 125,000 inversions performed for each model (total of over 4.3 million models assessed).   At this site, a four-layer model yielded a minimum misfit of 0.868, which corresponds to a good theoretical fit to the experimental data as presented in Figure 26.

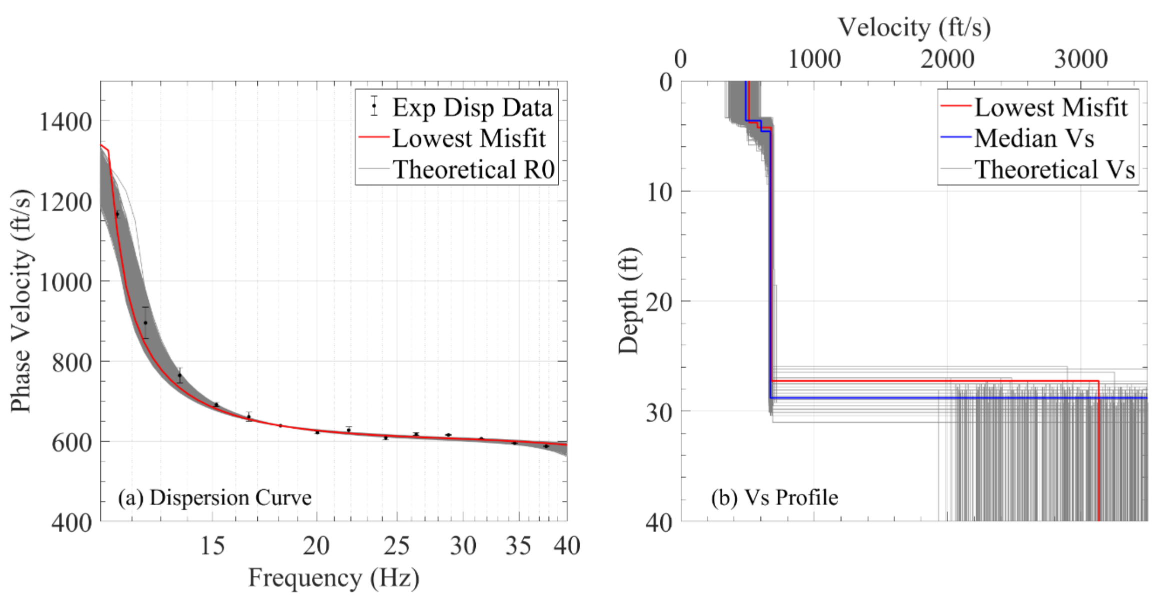

Figure 26 Dispersion curve and Vs profile with 1000 minimum misfit theoretical data for Muddy Creek

Based on the minimum misfit Vs profile a bedrock layer is expected at a depth near 28 ft below the survey and corresponds to a Vs greater than 3000 ft/sec.

A comparison between the soil boring data, pile driving blow count, maximum case method and Vs data are presented in Figure 27. At the Muddy Creek site, the depth of bedrock predicted by the Vs profile is within a few feet of the bedrock depth predicted in the soil boring data. However, the pile driving record and the max case method (which is a function of pile driving blow count) predict bedrock a few feet above soil boring data.

2.6 Project Site: Interchange Road

Interchange Road Bridge, located in Rock Springs, is a two span, 270 ft long overpass bridge. At this site, the natural ground layer, below the embankment, consists of stiff sandy clay with some interbedded silt. The bedrock layer was identified 34 to 36 feet below the ground surface at an elevation near 6200 ft. The bedrock materials were identified as moist, very dense interbedded siltstone, claystone and sandstone, and belong to the Tertiary Wasatch Formation. The soil profile is shown in Figure 28 (Scott, 2017). Data was collected at two array locations for this site, Array 1 was conducted along I-80 and Array 2 was conducted north-west of the existing bridge as presented in Figure 29. Array 1 data was very noisy, resulting in poor quality data; hence, only data collected at Array 2 will be presented herein. Array 2 was performed at an approximate elevation of 6245 ft, which is indicated by the red line in Figure 28. were collected at approximately

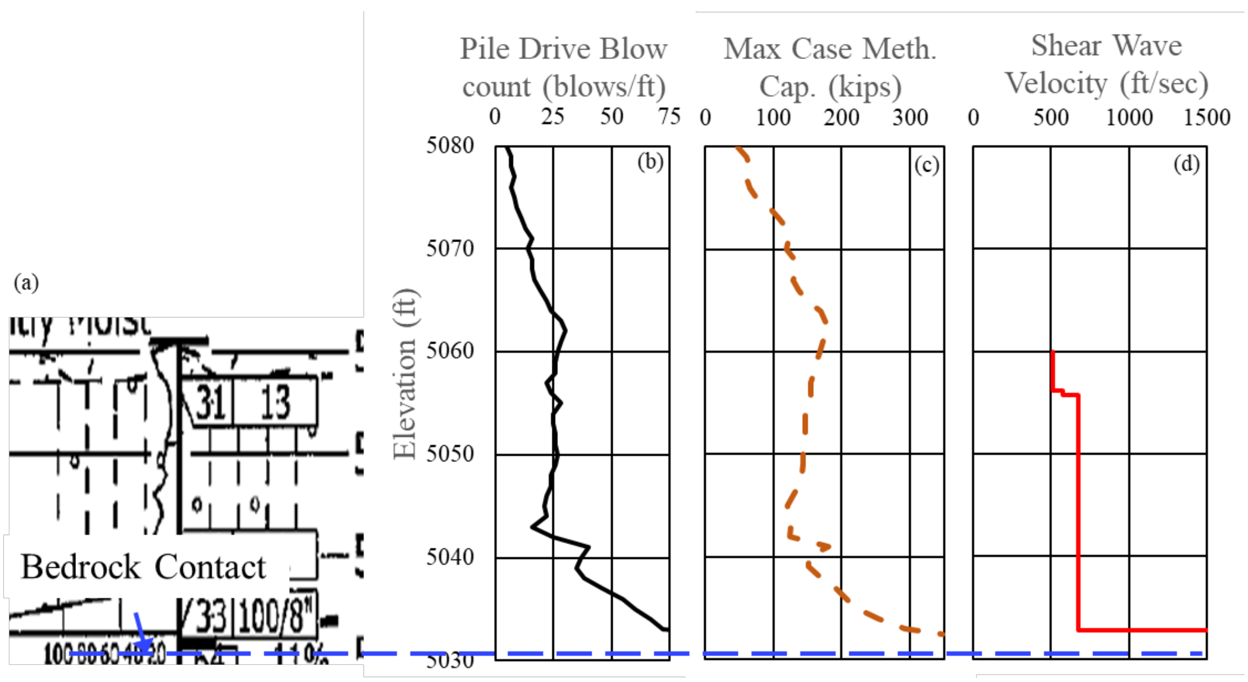

Figure 27 Bedrock depth comparison for Muddy Creek including a) soil boring information, b) pile driving blow count, c) max case method capacity and d) Vs profile all as a function of depth for Muddy Creek

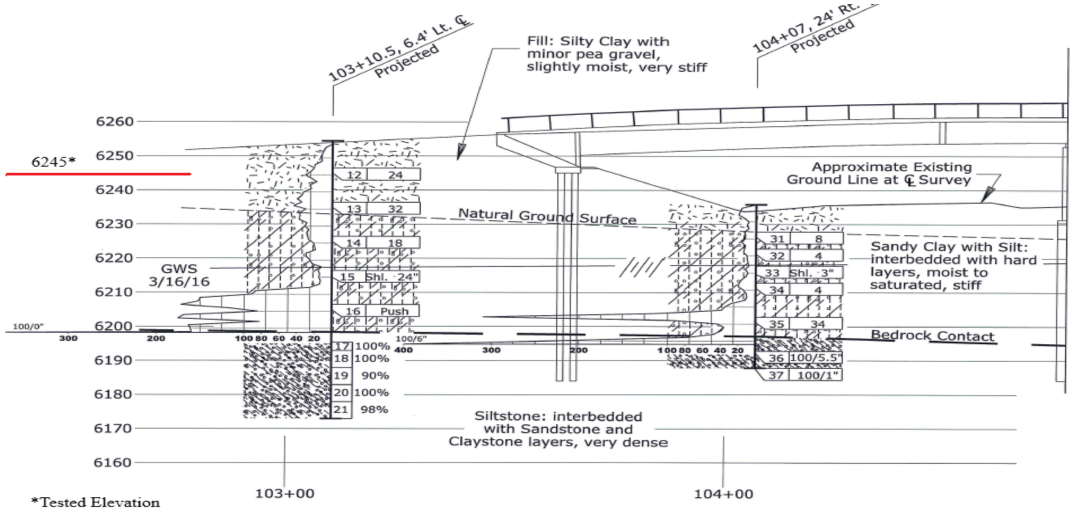

Figure 28 Subsurface cross section for Interchange Road Bridge replacement project (Scott, 2017), red line indicates elevation of the seismic survey.

Figure 29 Survey location on Interchange Road represented by yellow line (Google Earth Pro, 2021, with alterations)

For Interchange Road, the P-wave refraction analyses yielded Vp data for the top two soil layers. The topmost layer yielded a Vp = 1260 ft/sec and the second layer a Vp = 4684 ft/sec, as presented in Figure 30. While it is almost certain that more than two layers exist in the subsurface, the data collected using a sledge hammer source at this noisy site only provided enough signal strength to determine a Vp for the top two layers (i.e. a larger source paired with a longer array may produce a deeper model with more layers). Because of this, readers should not compare model layers between the compression wave and MASW testing.

The MASW data from Array 2 was very good quality data at all frequencies as presented in Figure 31. The data was trimmed at frequencies less than 5 Hz and higher than 40 Hz. Based on the experimental data the longest useable wavelength measured was 150 ft resulting in a maximum depth of investigation of 75 ft. (Comina et al., 2011). The inversion was performed using Geopsy with 16 initial models and over 125,000 inversions performed for each model a total of over 2 million models where investigated.

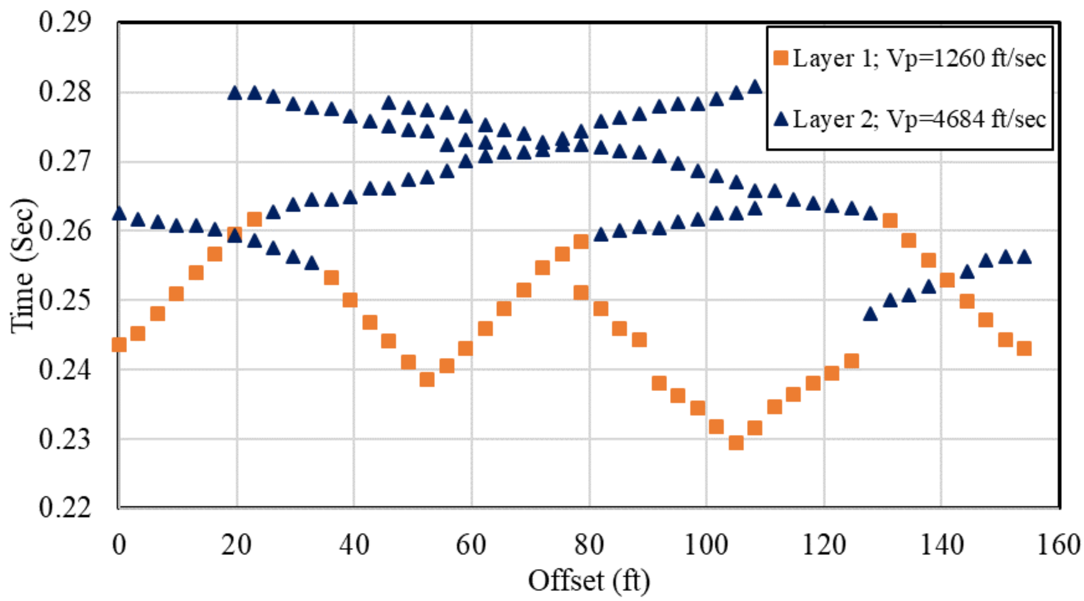

Figure 30 P-wave refraction data for Interchange Road, Rock Springs

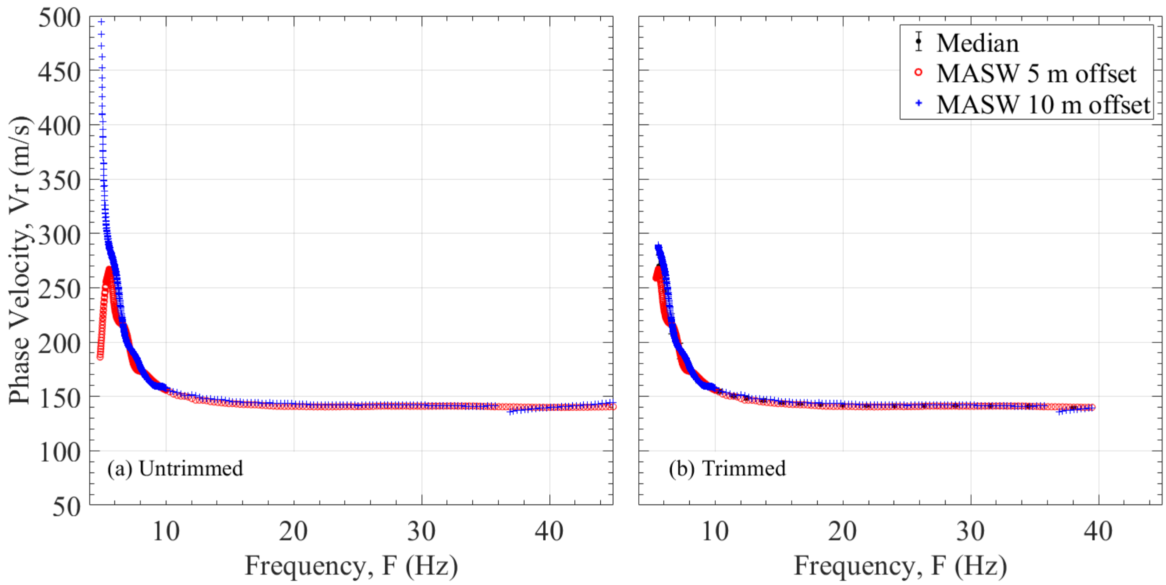

Figure 31 MASW (a) untrimmed and (b) trimmed measured dispersion data (Array 2) for Interchange Road, Rock Springs

At the Interchange Road site, a three-layer model yielded a minimum misfit of 0.553, which corresponds to a good theoretical fit to the experimental data, as presented in Figure 32. During the inversion process, a number of satisfactory solutions that fit within the uncertainty bounds provide a satisfactory fit to the experimental dispersion data 1000 of these are included in Figure 32 along with their corresponding Vs profiles.

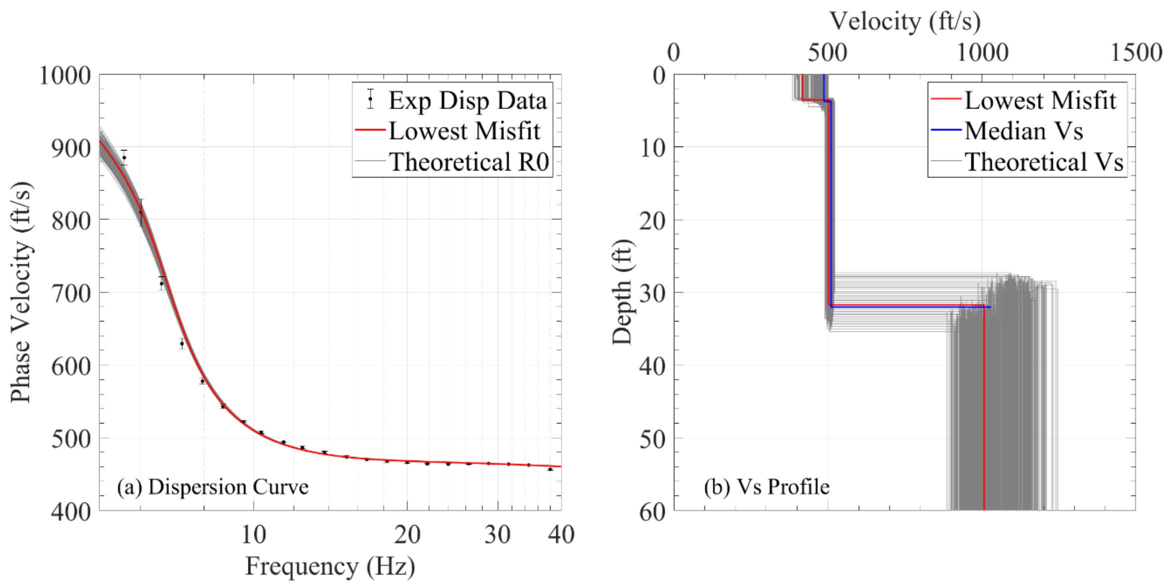

Figure 32 Dispersion curve and Vs profile with 1000 minimum misfit theoretical data for Interchange Road, Rock Springs

At the Interchange Road site, no velocity reversals were allowed in the inversion analysis. The minimum misfit Vs profile predicts a bedrock layer at a depth around 32 ft deep with a bedrock Vs = 1000 ft/sec. A comparison between the soil boring data, pile driving blow count, maximum case method and Vs data as presented in Figure 33. At the Interchange Road site, the depth of bedrock predicted by the Vs profile is a very good match to the bedrock depth predicted by the drive point data and pile drive blow count data.

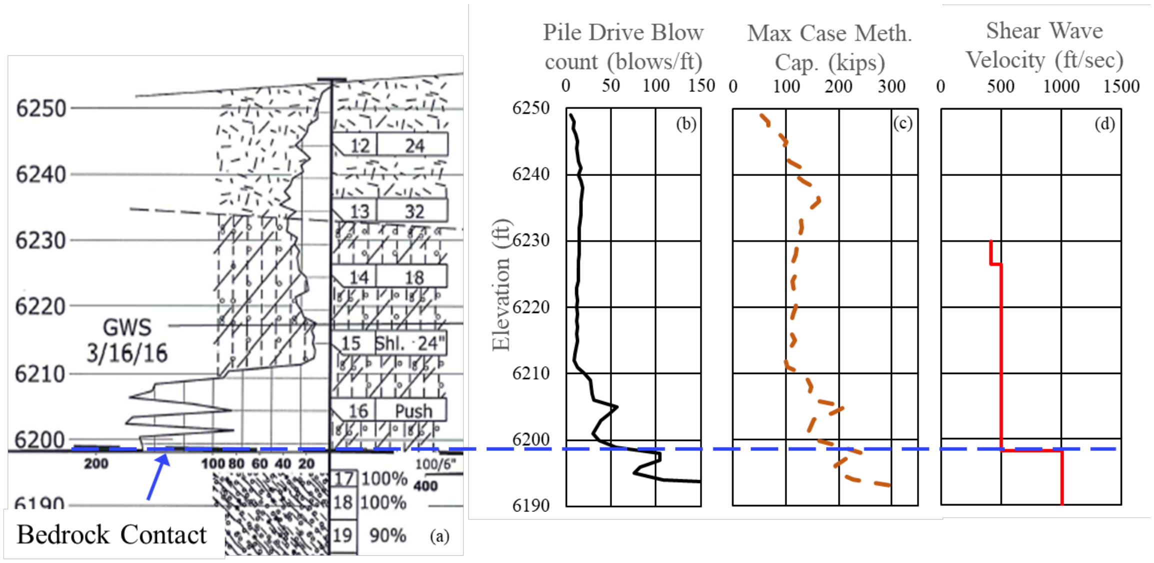

Figure 33 Bedrock depth comparison for Interchange Road including a) soil boring information, b) pile driving blow count, c) max case method capacity and d) Vs profile all as a function of depth for Interchange Road, Rock Springs

2.7 Project Site: BNSF Railroad

The bridge over the BNSF rail-line in Ranchester is a seven span, approximately 300 ft long W-beam girder railway overpass bridge. According to borehole data the near surface soils consist of medium stiff to stiff clay, underlain by flowing sand with gravel in a loose to dense state. The bedrock layer is expected at a depth near 30 ft corresponding to an elevation near 3730 ft. The bedrock layer consists of weathered to slightly weathered carbonaceous and silty shale in hard and dry to slightly moist condition and belongs to the Tertiary Lebo Member of the Fort Union Formation. The soil profile is presented in Figure 34 (Swanbom, 2019). Seismic data for this site was collected at an approximate elevation of 3762 ft. as shown by the red line in Figure 34. For this site, two arrays were used for data collection; Array 1 along the base of the embankment, and Array 2 parallel to the railway, as shown in Figure 35. Although data was collected at both arrays data between them were similar and as such only data from Array 2 is presented herein. At this site, data collection was approximately 150-200 ft away from the south Abutment. This distance is larger than the authors would have liked, but was the closest open area that was accessible for the testing.

At the BNSF Railroad site, the P-wave refraction analyses yielded the Vp of uppermost three layers, as presented in Figure 36. Layer 1 has a Vp = 1260 ft/sec, layer 2 with a Vp = 4100 ft/sec, and a third layer with a Vp of 6560 ft/sec. The P-wave refraction data was not as helpful as hoped. However, it did help with determination of the initial guesses used when performing the MASW inversion.

MASW data was measured using and active source at multiple offsets to determine an experimental dispersion curve. The initial data obtained ranged from frequencies near 40 Hz to around 10 Hz, as presented in Figure 37. The experimental data included the longest useable wavelength of 92 ft, which results in a maximum depth of investigation of 46 ft (Comina et al., 2011).

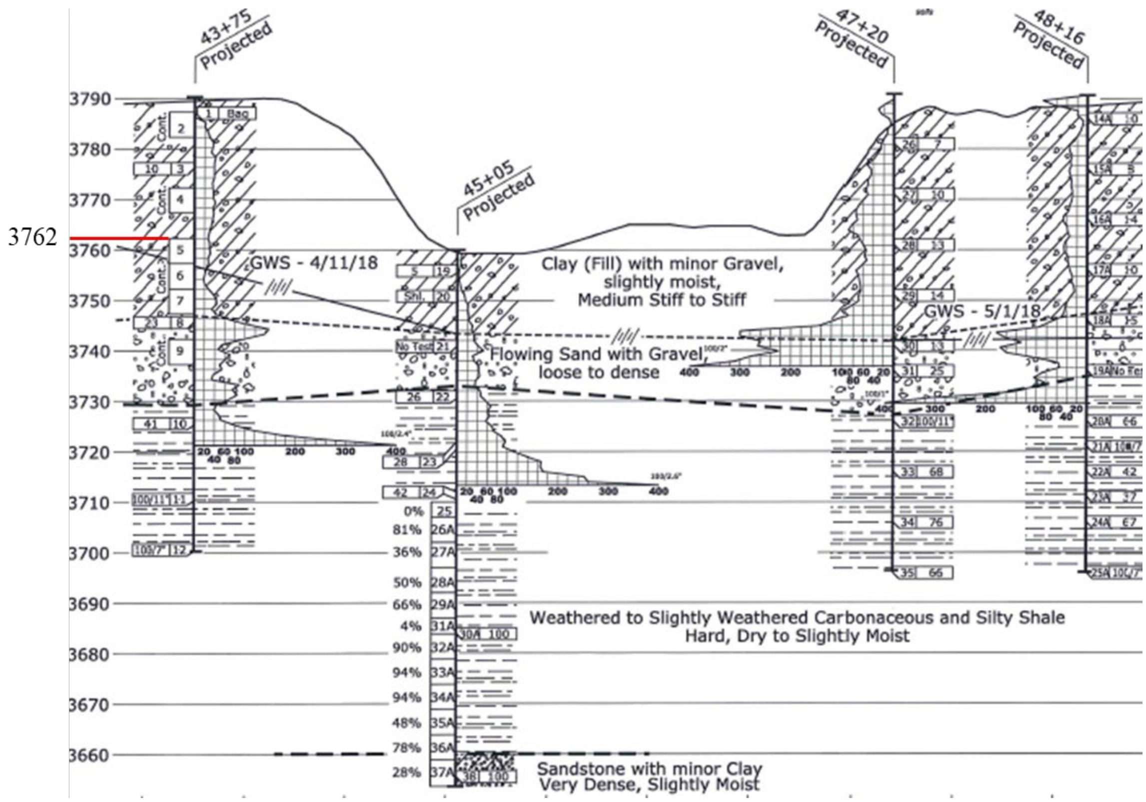

Figure 34 Existing soil profile data for the BNSF Railroad site obtained from WYDOT (Swanbom, 2019), red line indicates elevation of the seismic survey.

Figure 35 Survey location on the BNSF Railroad site represented by yellow line (Google Earth Pro, 2021, with alterations)

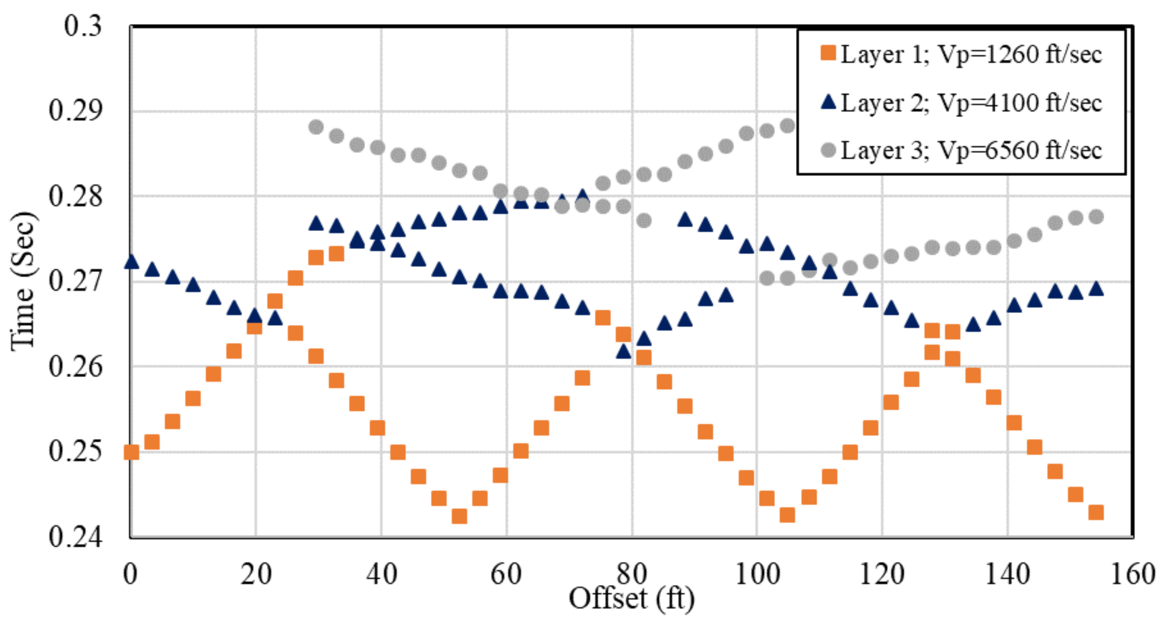

Figure 36 P-wave refraction data for the BNSF Railroad site

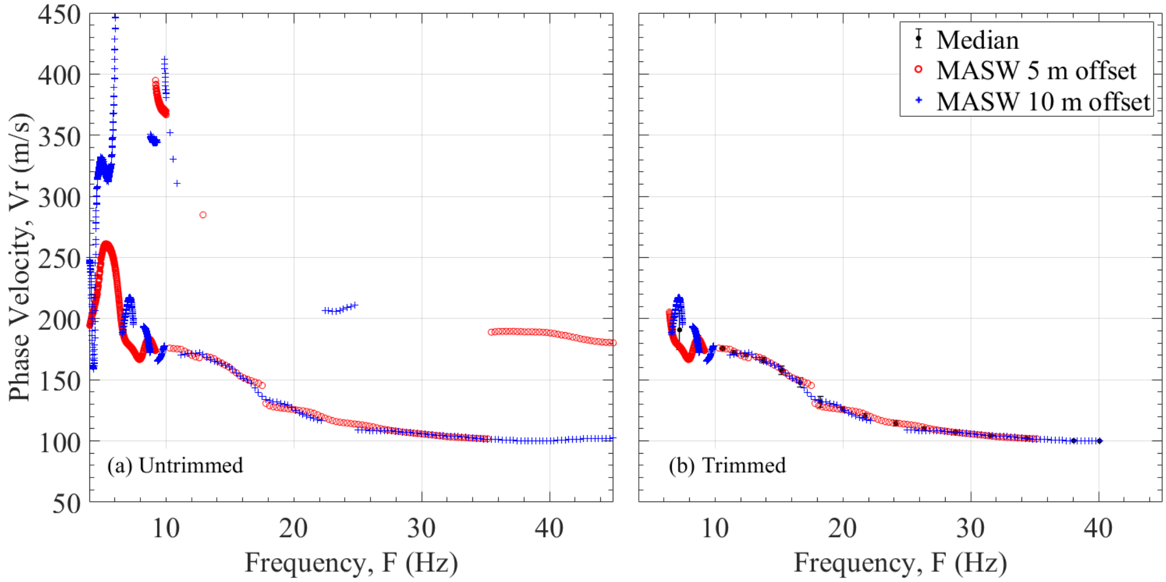

Figure 37 MASW (a) untrimmed and (b) trimmed measured dispersion data (Array 2) for BNSF Rail Road site

The inversion was performed using Geopsy with initial theoretical Vs parameters determined using the available borehole data, and the layering ratio method (Cox & Teague, 2016). At the BNSF Railroad site, 33 initial models were investigated with over 125,000 inversions performed for each model (total of over 4.1 million models assessed). Experimentally measured phase velocity data points at 7.9 Hz and 9.5 Hz were removed from the measured data to facilitate the inversion analyses. This had an effect on the determined Vs profile, but the data quality in the measured data at these frequencies appears to be multi-modal and leaving this data in resulted in poor quality inversion analyses. At this site, a four-layer model yielded a minimum misfit of 0.415. This data and the corresponding shear wave velocity profiles are presented in Figure 38.

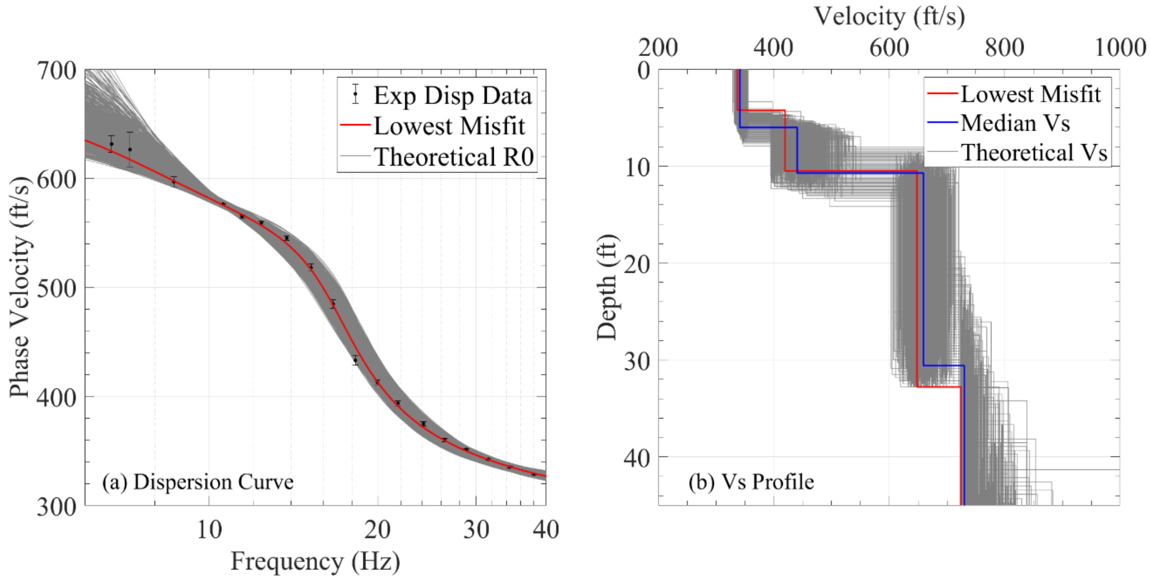

Figure 38 Dispersion curve and Vs profile with 1000 minimum misfit theoretical data for BNSF Rail Road, Ranchester

At the BNSF Railroad site, an increase in velocity with depth is expected and was modeled without allowing for velocity reversals. The inversion results are presented in Figure 38 along with the corresponding Vs profiles. The minimum misfit Vs profile includes a large velocity contrast at a depth near 10 ft, where the Vs changes from about 420 ft/s to 650 ft/s followed by a smaller increase in velocity at a depth near 32 ft. A comparison between the soil boring data, pile driving blowcount, and Vs data as presented in Figure 39. The depth of bedrock predicted using the boring data is shown by a dashed blue line across all panes in Figure 39. Below this depth there is an increase in the pile driving blow count. The depth of bedrock predicted by the Vs profile is a good match to the slight increase in the velocity contrast found during the inversion analyses.

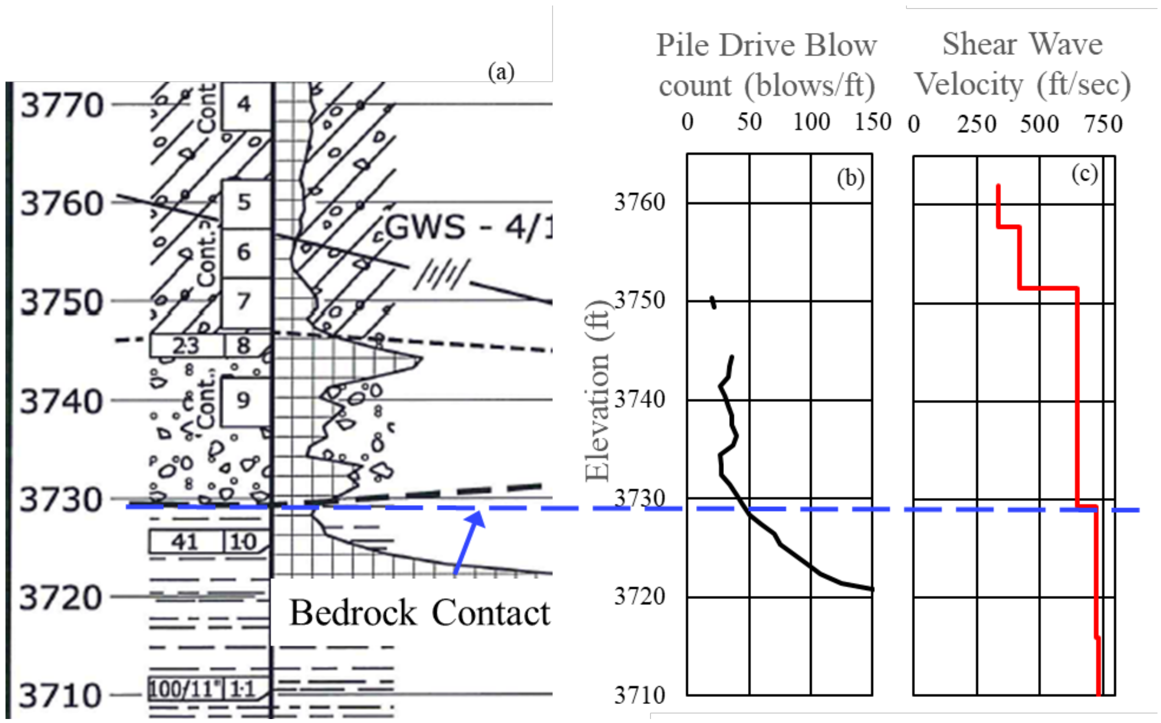

Figure 39 Bedrock depth comparison for BNSF Railroad site including a) soil boring information and b) Vs profile all as a function of depth

2.8 Project Site: Wildcat Creek

The bridge crossing Wildcat Creek, north of Gillette, Wyoming, is a four span, 130 ft long two-lane bridge. The bridge crosses Wildcat Creek, which is a seasonal drainage river with low flows. Based on borehole information, the near surface layer consists of loose, silty sand with fine clinker gravel to a depth near 12 ft.

Beneath the silty sand layer lies stiff slightly moist claystone underlain by dry coal stringers that belong to the Tertiary Wasatch Formation. Figure 40 shows the existing subsurface profile (Sullivan, 2018) along with the elevation of the seismic surveys which were conducted near an elevation of 4075 ft. For this project, two arrays were used to collect data parallel to the road near the bridge.

One on the North (Array 1) and one on the South (Array 2) as shown in Figure 41. Only data from Array 1 will be presented herein. For Wildcat Creek, the P-wave refraction analyses yielded the Vp of the top two layers as presented in Figure 42. The Vp of the surface layer is 1365 ft/sec and the Vp of the layer just beneath the surface layer is 5467 ft/sec. The Vp data was not used for any further analyses.

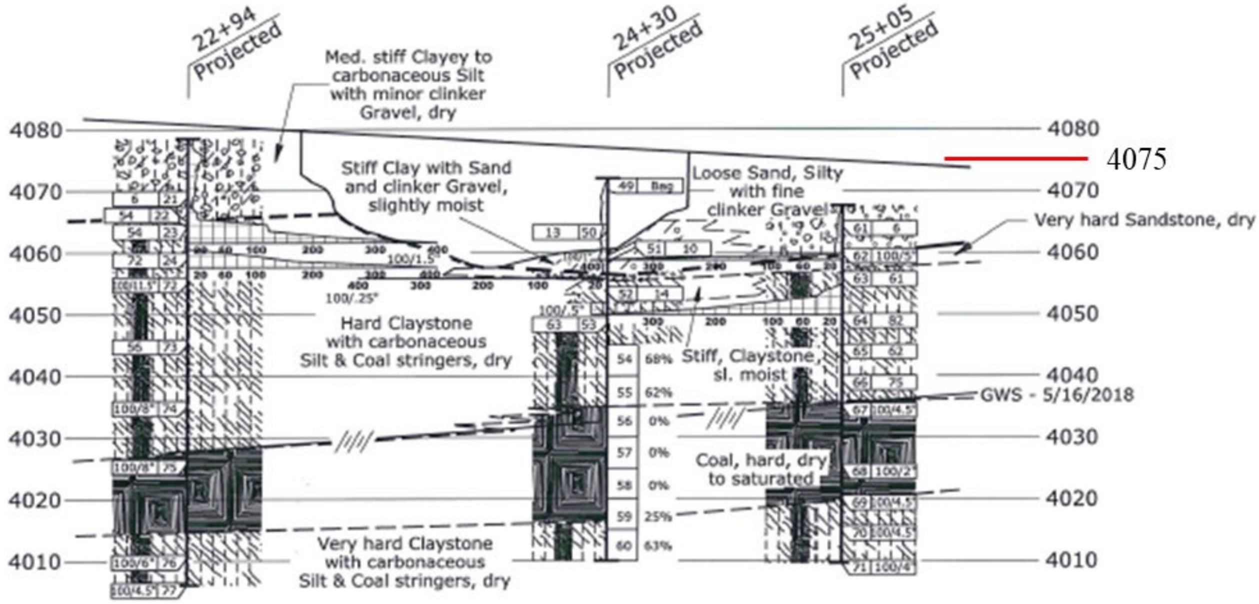

Figure 40 Existing soil profile data for Wildcat Creek obtained from WYDOT (Sullivan, 2018), red line indicates elevation of the seismic survey.

Figure 41 Survey location on Wild Cat Creek represented by yellow line (Google Earth Pro, 2021 with alterations)

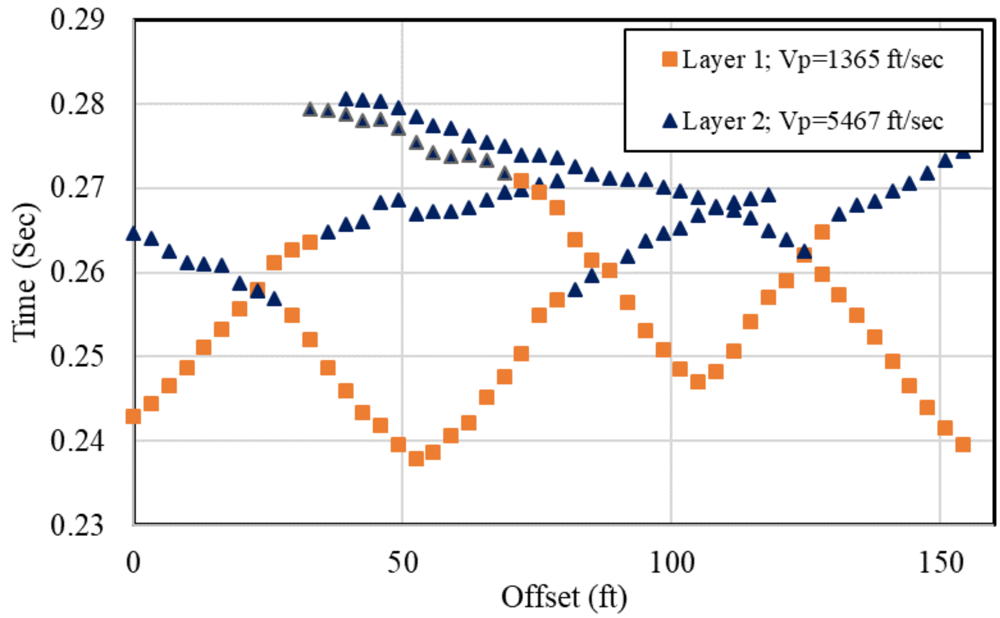

Figure 42 P-wave refraction data for Wildcat Creek, including all offsets

The MASW data presented in Figure 43 is very smooth at most frequencies except near 12 Hz, where a slight hump in the fundamental mode is observed. Raw data was trimmed at frequencies less than 7 Hz and greater than 40 Hz. Based on the experimental data the longest useable wavelength measured was 150 ft, which yields a maximum depth of investigation of 75 ft (Comina et al., 2011).

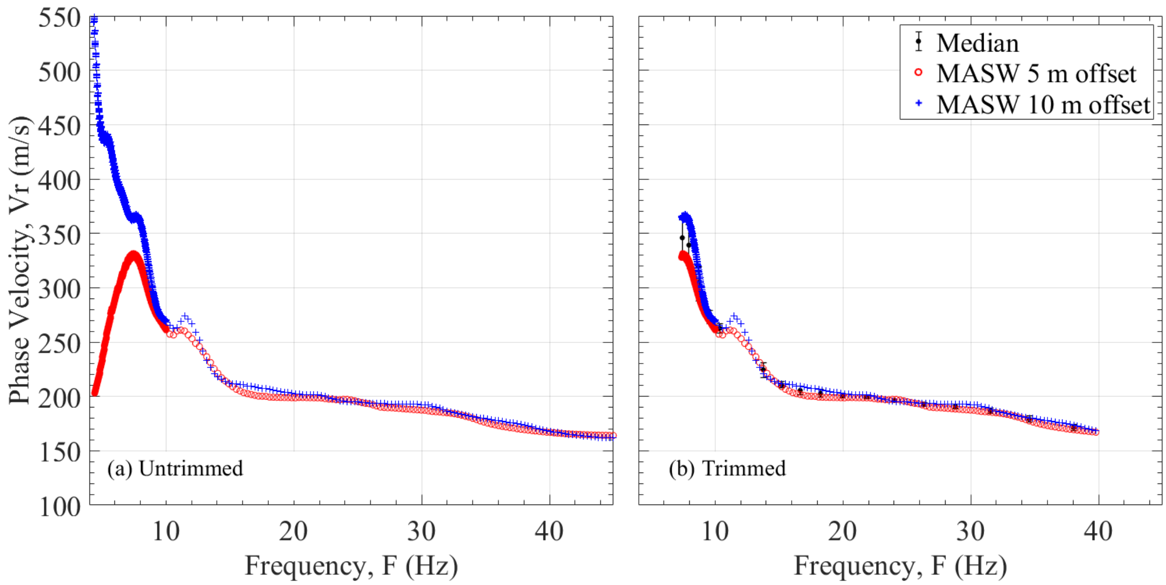

Figure 43 MASW (a) untrimmed and (b) trimmed measured dispersion data (Array 1) for Wildcat Creek

The inversion was performed using free, open-source software Geopsy. Initial theoretical Vs parameters were determined using the available borehole data, and the layering ratio method (Cox & Teague, 2016). At the Wildcat Creek site, 13 initial models were investigated with over 125,000 inversions performed for each model (total of over 1.6 million models assessed). Target dispersion data at 10.45 Hz and 12.46 Hz were removed so that the hump in the raw data was not modeled. The authors attempted to model the hump in the phase velocity around a frequency of 12 Hz; however, those models yielded unrealistic velocity layers and values. At this site, a four-layer model yielded a minimum misfit of 0.492, which corresponds to a good theoretical fit to the experimental data. This data and the corresponding shear wave velocity profiles are presented in Figure 44.

At the Wildcat Creek site, an increase in velocity with depth is expected based on borehole data. To model this appropriately, no velocity reversals were allowed in the inversion analysis. Figure 44 presents the 1000 theoretical Vs profiles, the minimum misfit Vs profile, and the median Vs profile from the 1000 Vs profiles. Based on the minimum misfit Vs profile, the bedrock layer is at a depth near 34 ft, which corresponds to an elevation of 4041 ft. At this elevation, the Vs increases from 800 ft/sec to greater than 1400 ft/sec. A comparison between the soil boring information, pile-driving record and the measured Vs profile are presented in Figure 45. The bedrock layer from the boring data is around an elevation near 4037 that is within five feet of that predicted by the Vs profile.

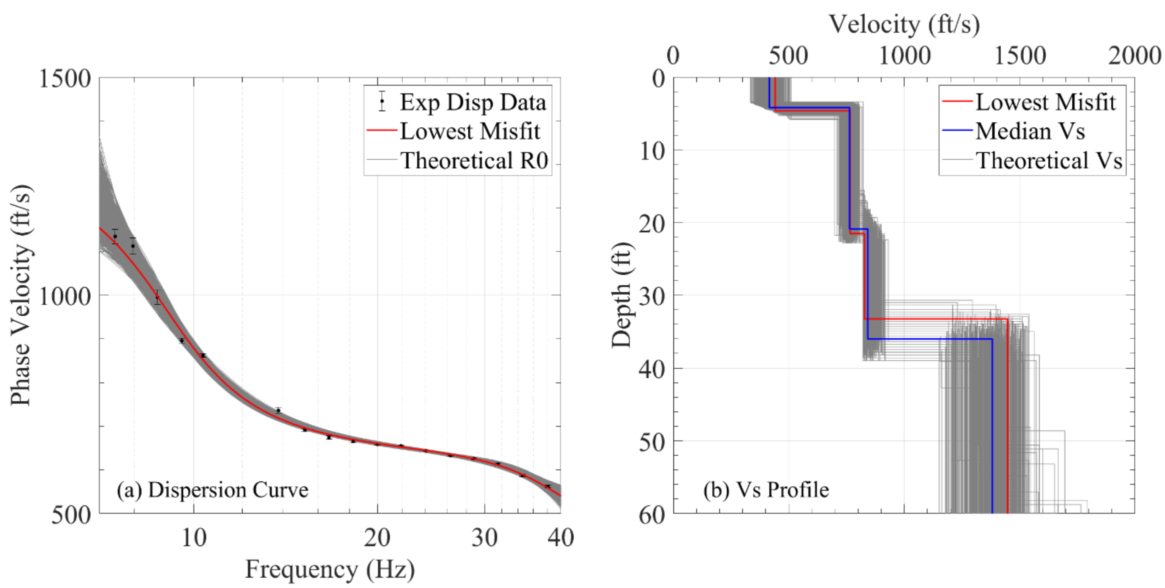

Figure 44 Dispersion curve and Vs profile with 1000 minimum misfit theoretical data for Wildcat Creek

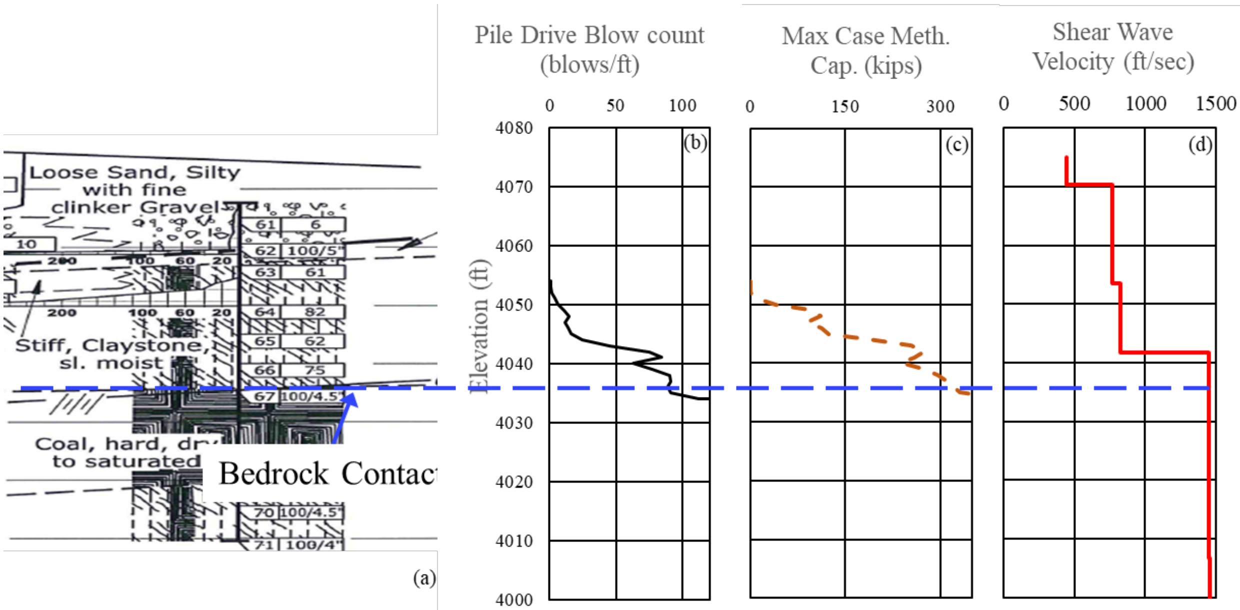

Figure 45 Bedrock depth comparison for Wildcat Creek including a) soil boring information, b) pile driving blow count, c) max case method capacity and d) Vs profile all as a function of depth

2.9 Project Site: Cedar Street

Cedar Street Interchange, in Rawlins, Wyoming, includes two three span bridges, approximately 130 ft long over highway 287 along Interstate 80 (east and westbound). Based on borehole data, a dry medium dense silty sand layer at the surface is underlain by the bedrock layer that consists of tuffaceous sandstone and belongs to the Tertiary Miocene Formation. The soil profile is presented in Figure 46. Due to limited space, only a single array was used for this site.   Seismic data for this site was collected at an elevation near 6700 ft, as shown by the red line in Figure 46.

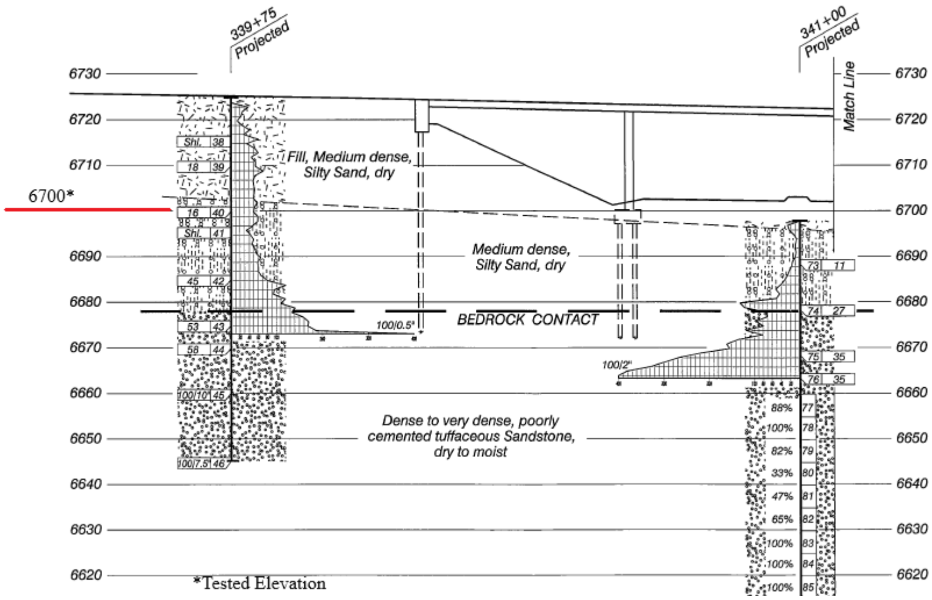

Figure 46 Existing soil profile data for Cedar Street Eastbound Lane obtained from WYDOT (Johnson, 2009), red line indicates elevation of the seismic survey.

The array was located at an elevation near 6700 ft on the south side of Interstate 80 parallel to the freeway at the base of the embankment, as presented in Figure 47. For Cedar Street, the P-wave refraction analyses yielded only a single layer model, as presented in Figure 48.

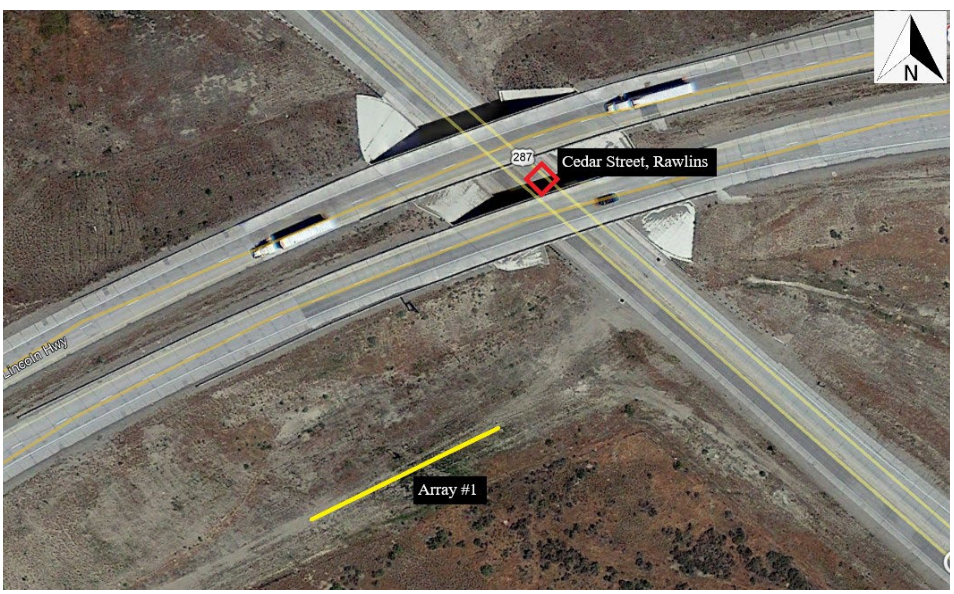

Figure 47 Survey location on Cedar Street represented by yellow line (Google Earth Pro, 2021, with alterations)

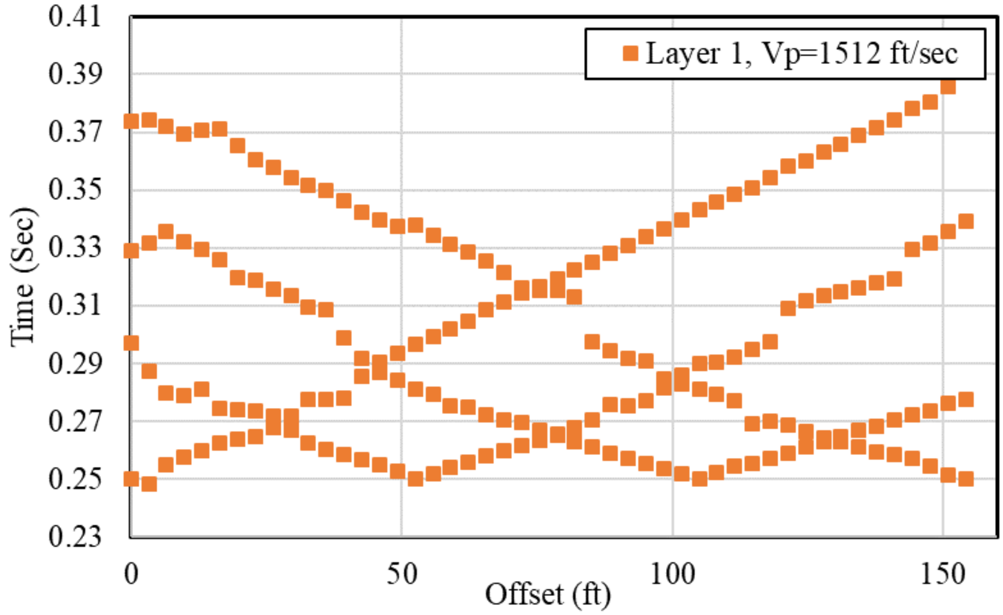

Figure 48 P-wave refraction data for Cedar Street

The P-wave refraction data were collected at shot points of 0 ft 52.5 ft, 105 ft, and the other end of the array 154 ft and yielded a compression wave velocity of the surface layer of 1512 ft/sec. This data was not used for any other analyses at this site. MASW data was measured using and impulse source at multiple offsets to determine an experimental dispersion curve. The initial data obtained shows a curve from 45 Hz to 6 Hz as presented in Figure 49.

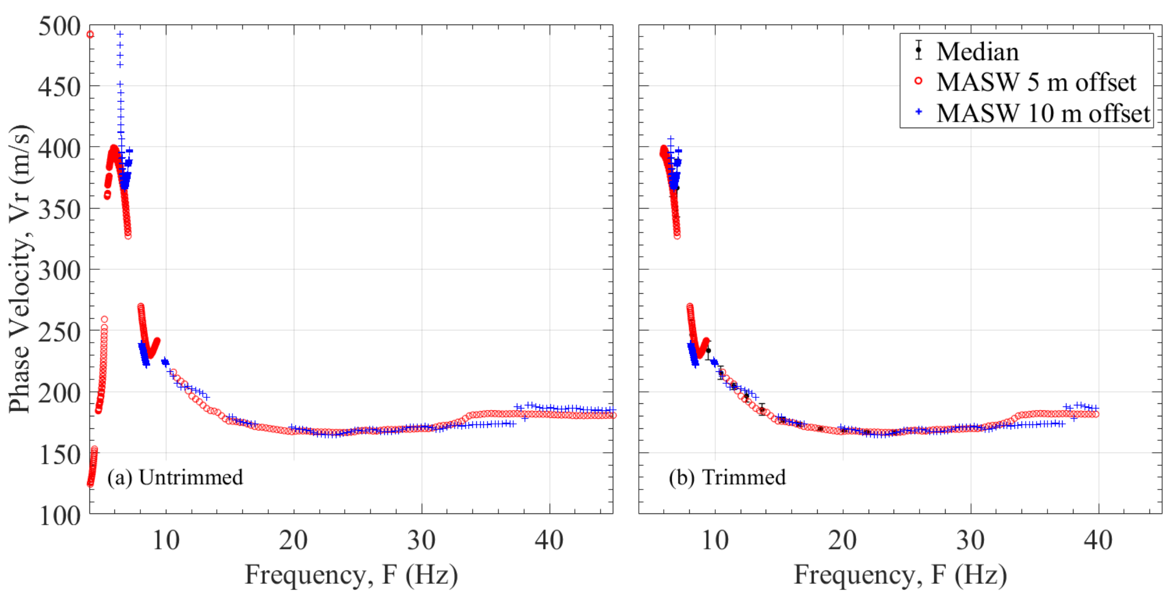

Figure 49 MASW (a) untrimmed and (b) trimmed measured dispersion data (Array 1) for Cedar Street

Based on the experimental data the longest useable wavelength measured was 203 ft which results in a maximum depth of investigation of 101 ft (Comina et al., 2011). The experimentally measured dispersion was resampled at pre-determined frequencies prior to inversion using the free open-source software Geopsy. Initial theoretical Vs parameters were determined using the available borehole data, P-wave refraction, and layering ratio method (Cox & Teague, 2016). At the Cedar Street site, 45 initial models were investigated with over 125,000 inversions performed for each model (total of over 5.6 million models assessed). The phase velocity around 8 Hz was removed to facilitate the inversion, without yielding unrealistic results. The Cedar Street site also includes a slightly decreasing phase velocity at low frequencies prior to a large increase. This is indicative of a harder crust at the surface with a softer layer underneath. However, when this was modeled with a combination of realistic velocities and a few different depths, no realistic model was predicted. In order to account for this the authors removed the higher frequencies from the inversion and focused on the phase velocity data at frequencies less than 25 Hz. This likely had little effect on the Vs determined at deeper layers because the frequencies above 25 Hz affect the near surface layers. Although this is not ideal, the deeper layers are still likely to be accurate.   Using the data at frequencies, less than 25 Hz a four-layered model yielded a minimum misfit of 0.788, which corresponds to a good theoretical fit to the experimental data. This data and the corresponding shear wave velocity profiles are presented in Figure 50.

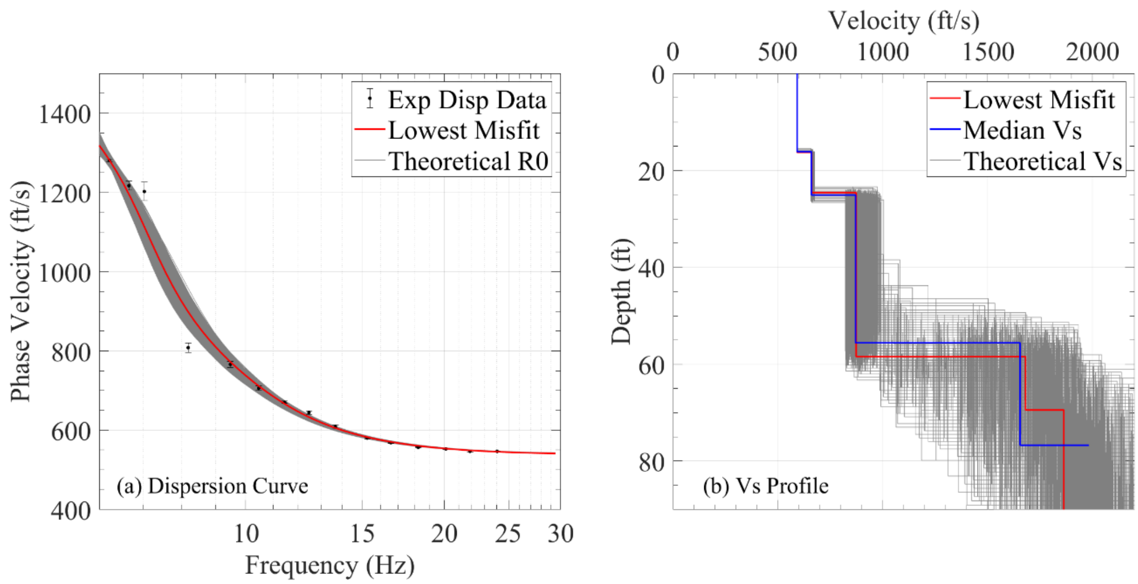

Figure 50 Dispersion curve and Vs profile with 1000 minimum misfit theoretical data for Cedar Street

A comparison between the soil boring data, pile driving and Vs data is presented in Figure 51. At the Cedar Street site, the depth of bedrock predicted by the Vs profile is within about 3 ft of the bedrock contact predicted by the soil boring.

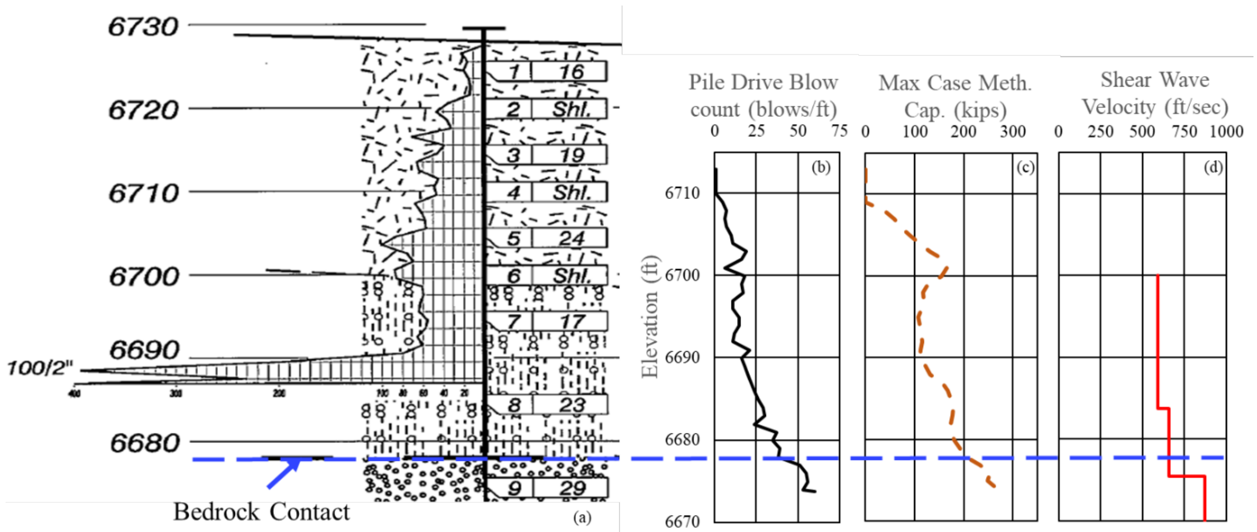

Figure 51 Bedrock depth comparison for Cedar Street including a) soil boring information, b) pile driving blow count, c) max case method capacity and d) Vs profile all as a function of depth

2.10 Project Site: Owl Creek

Owl Creek Bridge is a three span 140 ft long two-lane bridge over a small creek in Hot Spring County. Based on the existing borehole information, the subsurface consists of a loose to dense sand and gravel layer from the surface down to a weathered shale, which belongs to the Cretaceous Age Cody Shale Formation. The soil profile is presented in Figure 52. The seismic survey was performed at an elevation of approximately 5189 ft as shown in Figure 52. Due to site constraints, two arrays parallel to the bridge were conducted, as presented in Figure 53. The two arrays were similar in quality and only data from Array 2 will be presented herein.

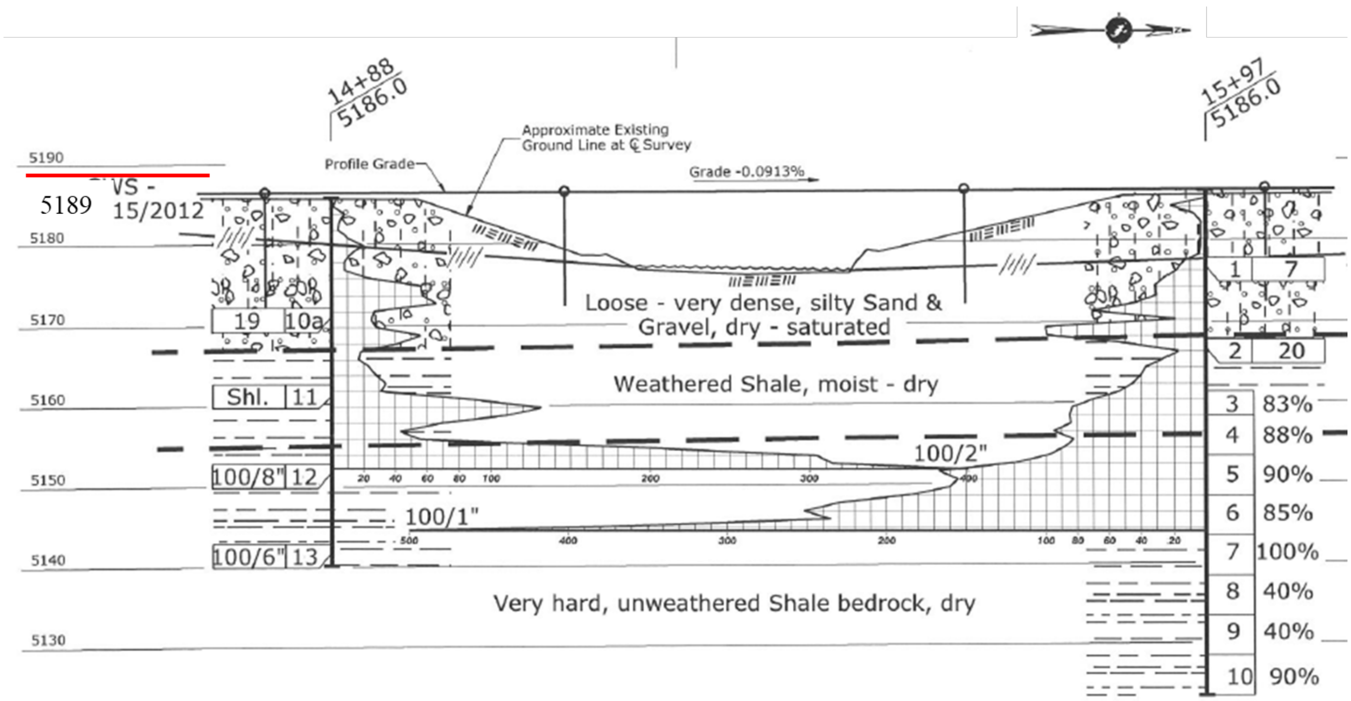

Figure 52 Existing soil profile data for Owl Creek obtained from WYDOT (Smith, 2013), red line indicates elevation of the seismic survey.

Figure 53 Survey location on Owl Creek represented by yellow line (Google Earth Pro, 2010, with alterations)

For Owl Creek, the P-wave refraction analyses yielded the Vp of the top two layers, as presented in Figure 54. The surface-most layer indicated a Vp of 2050 ft/sec, and the layer just beneath a Vp of 6560 ft/sec. These velocities are quite high, and were not used in any further analyses.

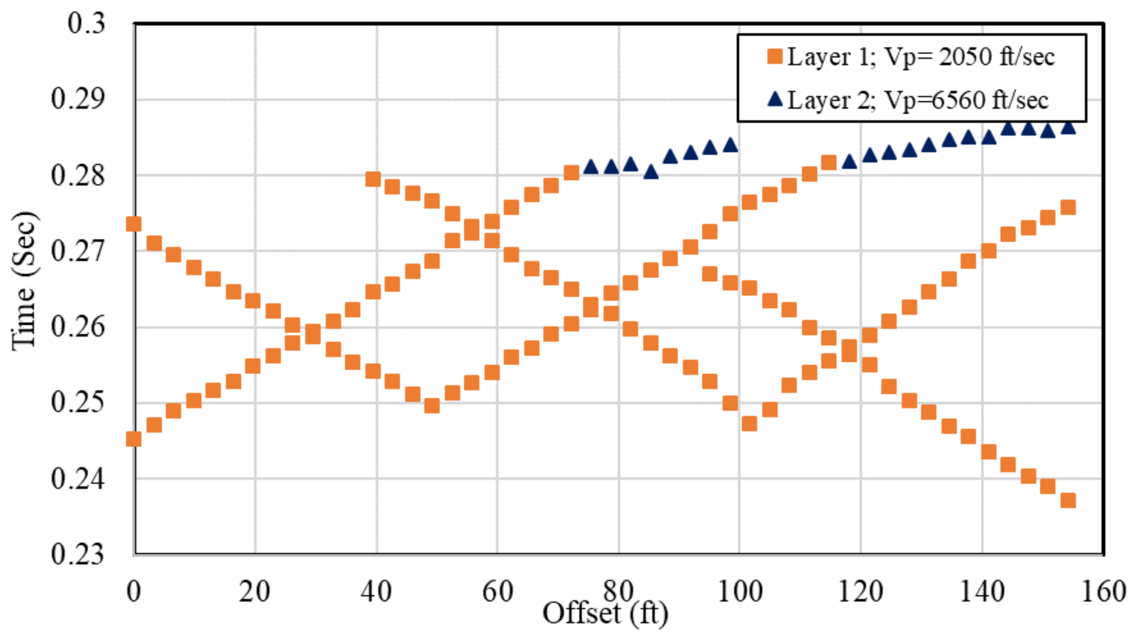

Figure 54 P-wave refraction data for Owl Creek

The MASW data was measured using and active source at multiple offsets to determine an experimental dispersion curve. Both offsets within the data agreed with one-another and the initial data obtained showed a very smooth curve from high to low frequencies, as presented in Figure 55.

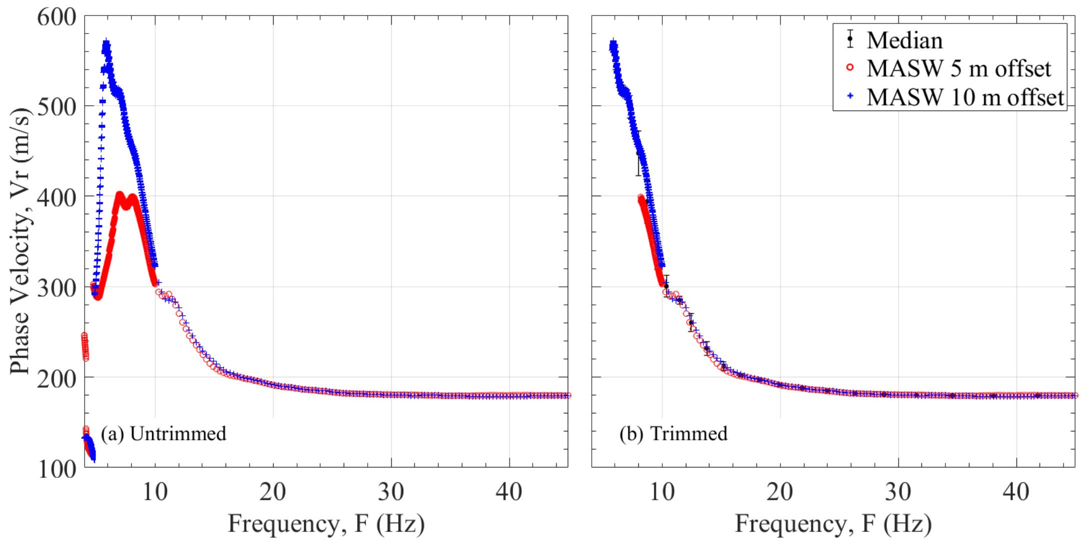

Figure 55 MASW (a) untrimmed and (b) trimmed measured dispersion data (Array 2) for Owl Creek

Based on the experimental data the longest useable wavelength measured was 225 ft which results in a maximum depth of investigation of 112 ft. (Comina et al., 2011). At the Owl Creek site, 37 initial models were investigated with over 125,000 inversions performed for each model (total of over 4.6 million models assessed). At this site, a five-layer model yielded a minimum misfit of 0.949, which corresponds to a good theoretical fit to the experimental data. This data and the corresponding shear wave velocity profiles are presented in Figure 56.

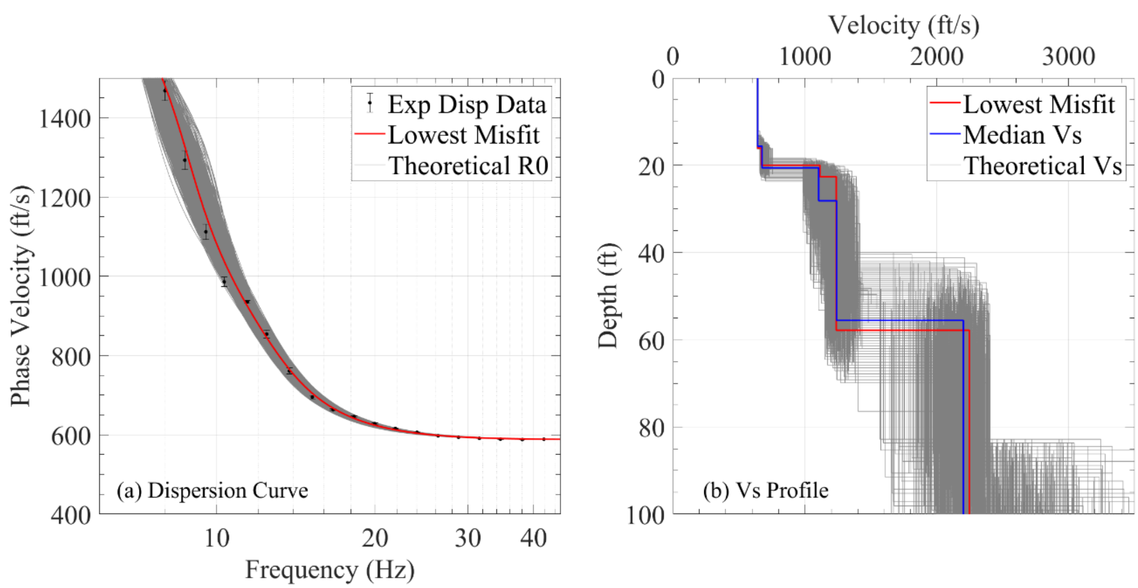

Figure 56 Dispersion curve and Vs profile with 1000 minimum misfit theoretical data for Owl Creek

During the inversion process, a number of satisfactory solutions that fit within the uncertainty bounds provide a satisfactory fit to the experimental dispersion data. In order to characterize some uncertainty, 1000 Vs profiles that produce the lowest misfit dispersion estimates are also presented in Figure 56. At the Owl Creek site an increase in velocity with depth is expected based on borehole date. To appropriately model this no velocity reversals were allowed in the inversion analysis. Based on the minimum misfit Vs profile a bedrock depth near 21 ft deep is expected where the Vs increases from about 600 ft/sec to about 1200 ft/sec and a larger increase in velocity is predicted at a depth of 55 ft from the surface with a Vs greater than 1200 ft/sec.

A comparison between the soil boring data and the predicted Vs profile is presented in Figure 57. At an elevation around 5169 there is an increase in the velocity that corresponds to the bedrock depth. At a much greater depth around elevation 5131 ft there is another increase in velocity that may correspond to hard in-tact bedrock or some other dense layer.

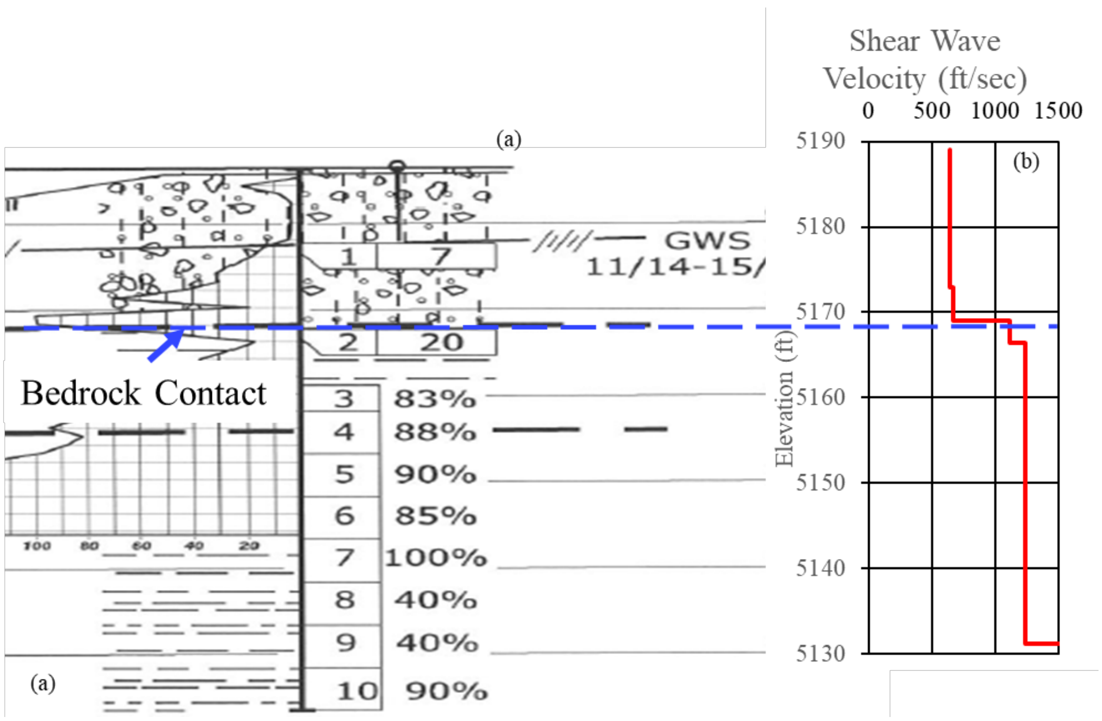

Figure 57 Bedrock depth comparison for Owl Creek including a) soil boring information and b) Vs profile as a function of depth

2.11 Conclusion

At most sites the Vs profile provided a very good estimate of bedrock depth, or an increase in Vs at a depth within five feet of the bedrock layer. It should be mentioned that although the comparisons between the drill logs, pile driving records, or Vs profiles were shown in side-by-side comparisons this data was not collected at the exact same location. In fact, at almost every site; offsets, and interpolation had to be used to make these comparisons. However, the authors believe that these comparisons still provide a valuable addition to the report and allow bedrock depth comparisons that could not otherwise be performed.

The P-wave refraction data collected for this report did not produce results as expected. At nearly every site, there was a lack of data, or the noise at the site over-powered the compression wave, making it hard to determine a first arrival. This could be overcome in future experiments by using a larger source. However, the authors caution that even a larger source may not result in better data, and other measures may need to be employed. These could include, working at night, or at times when traffic noise can be minimized, longer arrays, better geophones, better coupling and other factors. The authors were hopeful that this testing would produce P-wave and velocity models that would allow us to detect the boundary between weathered and more in-tact bedrock, however, this was not the case. It is suspected that detecting this layer boundary may not be possible with current P-wave and MASW testing and inversion techniques.

CHAPTER 3 DISCUSSION AND CONCLUSION

P-wave refraction and MASW testing was performed at nine bridge sites in Wyoming. At each site, the seismic testing was compared to drive point data and pile driving logs at the sites where piles were used. The P-wave refraction data did not produce Vp at the depths hoped for in this study, and much of the data was not used for anything more than initial guesses in the MASW inversion analyses. It was hoped that the P-wave data would allow the researchers to measure the Vp above and below the weathered zone within the bedrock. However, this was not the case, and at almost every site only the Vp of the top two layers was measured except the BNSF Railway site where three layer Vp were measured. At each site, the first-arrivals picked at large distances (second or third layers down from the surface) required the authors to use their best judgement. This is not unusual when analyzing compression wave data, however the authors recommend caution when using the Vp of the second or third layers determined at each site.

The MASW seismic data was analyzed using the Frequency Domain Beamformer method (FDBF, Zywicki 1999) as well as the layering ratio approach as suggested by Cox and Teague (2016). Inversion analyses employed the program Geopsy which uses the neighborhood algorithm to search for acceptable solutions for the measured surface wave data (Wathelet, 2008).

Table 2 presents bedrock depths associated with each site determined from boring logs as well as from MASW testing. At six of the nine sites, the bedrock depth determination is within a foot or two. At two of the nine sites, bedrock depth was predicted within five to seven feet of one another, and at Lodgepole Creek the bedrock depth prediction between the drive point and MASW testing did not match. Overall, the MASW predicted bedrock depth at most of the sites within a foot or two. It was hoped that this study would show that MASW testing would allow practitioners at WYDOT to use compression wave and MASW testing and perform fewer traditional borings. However, an accurate determination of bedrock depth is of paramount importance when bidding pile driving projects, and the MASW testing did not provide results accurate enough to negate the need for extra boreholes. Especially when considering pile driving contracts and how contractors are compensated. Unfortunately, this study showed that the accuracy needed to determine very accurate bedrock depths, for pile driving, was not possible using MASW testing.

Table 2. Comparison of Bedrock elevation and depth for each site

Site[1,2]	Elevation of Survey (ft)	"Bedrock" elevation (ft)	Depth of Bedrock Beneath Survey		Consistent?
			Borehole data (ft)	MASW Inversion (ft)	
Lodgepole Creek	5037	4969	68	39	No
Parson Street	5095	5039	57	50	Close
Beech Street	5097	5073	24	27	Yes
Muddy Creek	5060	5030	30	28	Yes
Rock Springs	6230	6200	30	32	Yes
Ranchester	3761	3730	31	32	Yes
Wildcat Creek	4075	4036	39	34	Close
Cedar Street[A]	6700	6678	22	25	Yes
Owl Creek[B]	5189	5169	20	20	Yes

1 - All values in this table are interpreted from soil profile data and MASW Vs data. They are approximate

2 - All survey elevation data are approximate

A - Cedar Street Vs data includes increase of Vs at depth of 22 ft. but boring data isn't sufficient to confirm increse in stiffness at 60 ft.

B - Owl creek data includes Vs increase around 20 ft which matches boring, however deeper increase in Vs is not verified by boring data

A second goal of this project was to check for correlations between Vs and the other data including; uniaxial compression strength, rock quality designation, and percentage recovery from the rock coring. These comparisons are shown in Figures 58, 59, and 60 respectively. A correlation between any of the three variables would result in a linear trend in data. However, there is no linear trend, or a trend of any kind between the Vs and the variable chosen. This could be due to the fact that the MASW testing results in an "average" shear wave velocity of all layers above the deepest layer, or that MASW testing is performed over a large area and averages the velocity from that area.

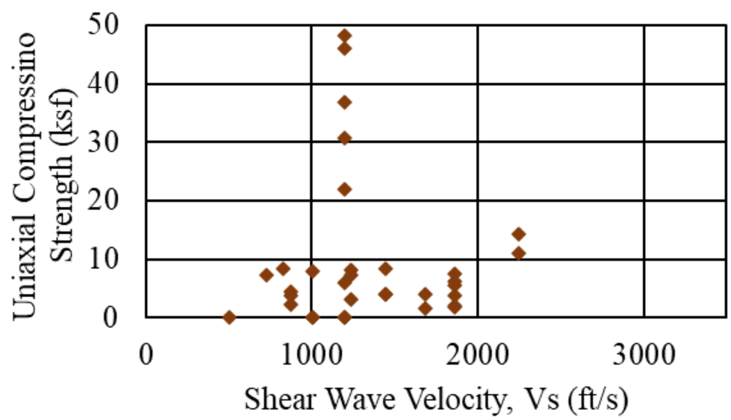

Figure 58 Correlation between uniaxial compression strength and shear wave velocity

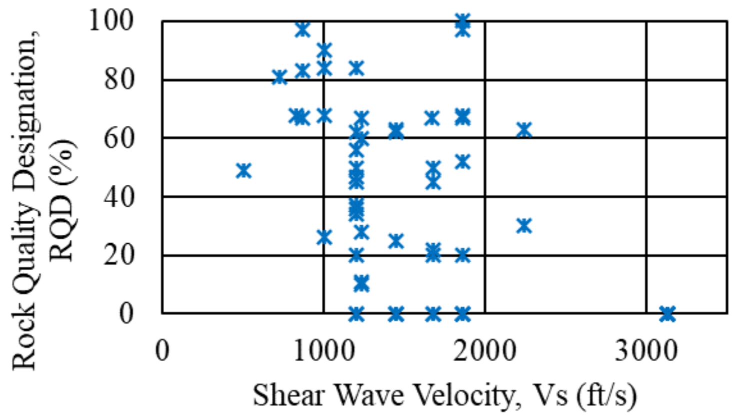

Figure 59 Correlation between rock quality designation and shear wave velocity

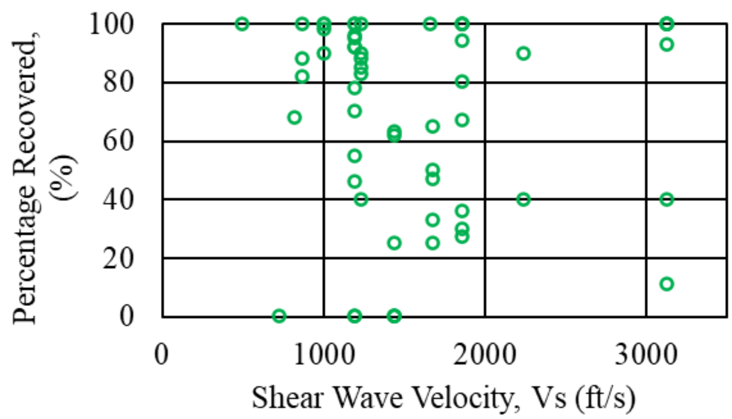

Figure 60 Correlation between percentages recovered from coring and shear wave velocity

Whatever the reason, the lack of any type of correlation means that there was no reason to perform any type of statistically analyses to determine any type of confidence for any of the correlations.

A third goal of the project was to predict the boundary between the weathered bedrock layer and the more competent rock below the weathered zone. This soil in this zone has been termed intermediate geomaterial, with the soil sometimes behaving like a rock and at other times like a soil. Regardless, none of the Vs profiles at any of the nine sites have any correlation to this zone. Although the bedrock depth was predicted relatively close at six of the nine sites, even these six sites showed no increase in Vs that would correlate to a non-weathered bedrock layer. This could be due to the averaging effect of MASW testing, or due to the fact that weathered rock still has enough structure intact to transmit waveforms through at a velocity very similar to the in-tact bedrock.

Despite challenges associated with determination of bedrock depth, the MASW technique did provide a measure of stiffness for each site tested. In areas with moderate to high seismic risk (i.e. the western part of Wyoming) these tests provide valuable stiffness data that can be used for advanced modeling and design of bridges.

Future work could include more inversion modeling that may reduce or increase variability in the model, or result in advanced inversion techniques. Rock samples taken from the coring could also be used to perform shear wave velocity measurements in the lab that may produce more accurate correlations. It may also benefit the correlations to perform some advanced downhole or seismic cross hole testing. This can be especially helpful at sites with dipping layers. For any future researchers attempting to perform compression wave testing along roadways researchers should include a larger source for P-wave testing. The authors recommend a large active repetitive source (like a vibroseis) which may allow for deeper, more reliable data. It would also be advisable when performing MASW testing to collect love wave data, and perform a joint inversion that includes both love and surface wave data.

REFERENCES

Comina, C., Foti, S., Boiero, D., & Socco, L. V. (2011). Reliability of VS,30 Evaluation from Surface-Wave Tests. *Journal of Geotechnical and Geoenvironmental Engineering*, *137*(6), 579–586. https://doi.org/10.1061/(ASCE)GT.1943-5606.0000452

Cox, B. R., & Teague, D. P. (2016). Layering ratios: a systematic approach to the inversion of surface wave data in the absence of *a priori* information. *Geophysical Journal International*, *207*(1), 422–438. https://doi.org/10.1093/gji/ggw282

Ear to the Ground (2016). "Peering Into The Earth: Geophysical Techniques Can Find Rock For Forest Roads". <https://washingtondnr.wordpress.com/2016/08/07/peering-into-the-earth-geophysical-techniques-can-find-rock-for-forest-roads/>, 9/27/19.

El-Naqa, A. (1996). "Assessment of geomechanical characterization of a rock mass using a seismic geophysical technique." Geotechnical and Geological Engineering, 1996, 14, pp. 291-305.

Google Earth Pro (2021). Wyoming, USA, multiple locations within the state. <https://earth.google.com/web/@0,0,0a,22251752.77375655d,35y,0h,0t,0r> (accessed, 2021)

Green, J. (2014). Beech *Street Overpass Bridge Replacement Structure BAP (WBL) Station 2118+73.34, and Structure BAO (EBL), Station 2118+15.49, Milepost 402.14.* Wyoming Department of Transportation, Project I806198/I806199, Unpublished internal document.

Griffiths, S. C., Cox, B. R., Rathje, E. M., & Teague, D. P. (2016). Surface-Wave Dispersion Approach for Evaluating Statistical Models That Account for Shear-Wave Velocity Uncertainty. Journal of Geotechnical and Geoenvironmental Engineering, 10.1061/(ASCE)GT.1943-5606.0001552.

Hanson, B. (2014a). *Parson Street Bridge Replacement Structure BAN (WBL) Station 2082+50.86, and Structure BAM EBL, Station 208+94.15 (R.M.401.46).*Wyoming Department of Transportation, Project I806198, Unpublished internal document.

Hanson, B. (2014b). *Muddy Creek Bridge Replacement Structure BAJ (EBL) Station 2066+49.50, and Structure BAK (WBL), Station 2067+7.50 (R.M. 401.16).* Wyoming Department of Transportation, Project I806198, Unpublished internal document.

Hood, Kirk. Interview. Conducted by Shawn Griffiths. September 27, 2019

Johnson, R. (2009). *Cedar Street Interchange, Interstate 80, M.P. 215.57*. Wyoming Department of Transportation, Project 0804234/80-4(234), Unpublished internal document..

Love, J.D., and Christiansen, A.C. (1985). "Geologic Map of Wyoming". Department of the Interior, U.S. Geologic Survey in cooperation with the Geological Survey of Wyoming.

McCann., D.M., Culshaw, M.G., and Northmore, K.J. (1990). “Rock Mass Assessment from Seismic Measurements.” From Bell, G.G., Clushaw, M.G., Cripps, J.C. and Coffey, J.R. (eds). Feild Testing in Engineering Geology. Geological Society Engineering Geology special Publication No 6, pp 257-266

Nazarian, S. (1984). “In Situ Determination of Elastic Moduli of Soil Deposits and Pavement Systems by Spectal-Analysis-of-Sruface-Waves Method.” Ph.D. dissertation. The University of Texas at Austin, 1984.

Nazarian, S., Stokoe, K.H., Briggs, R.C., and Rogers, R., (1988). “Determination of Pavement Layer Thicknesses and Moduli by SASW Method.” Transportation Research Record No. 1196. Official Journal of the Transportation Research Board. pp. 133-150.

Park, C. B., Miller, R. D., & Xia, J. (1999). Multichannel analysis of surface waves. In *GEOPHYSICS* (Vol. 64, Issue 3).

Poudal, R.D. (2021). "Characterization of Sub-Surface Vs Profile Using Seismic Methods In Wyoming for Transportation Infrastructure" University of Wyoming, M.S. Thesis.

Rivers, B., and Nichols, S. (2019). “EDC-5: Advanced Geotechnical Methods in Exploration (A-GaME).” Center for Accelerating Innovation, U.S. Department of Transportation Federal Highway Administration. < https://www.fhwa.dot.gov/innovation/everydaycounts/edc_5/geotech_methods.cfm>, 9/30/19.

Scott, J. (2017). *Bridge over I-80, Blair Interchange and Interchange Road*. Wyoming Department of Transportation, Project B123014 and I802220.

Scott, J. (2019). *Supplemental Drilling for the Bridge Replacement over Lodgepole Creek*. https://doi.org/February 14, 2019. Wyoming Department of Transportation, Project 1104015, Unpublished internal document.

Smith, B. (2013). *Foundation Investigation for Bridge Replacement over Owl Creek, Station 15+40, M.P. 10.68, Structure CPV.* Wyoming Department of Transportation, Project 090013, Unpublished internal document..

Swanbom, L. (2019). *Bridge over BNSF Railway, (Structure No. CDR), RM 89.38*. Wyoming Department of Transportation, Project N372041, Unpublished internal document..

Wathelet, M. (2008). An improved neighborhood algorithm: parameter conditions and dynamic scaling. *GEOPHYSICAL RESEARCH LETTERS*, *35*, 9301. https://doi.org/10.1029/2008GL033256

Wathelet, M., Jongmans, D., & Ohrnberger, M. (2004). Surface-wave inversion using a direct search algorithm and its application to ambient vibration measurements. *Near Surface Geophysics*, *2*(4), 211–221. https://doi.org/10.3997/1873-0604.2004018

Wood, C. M., Asce, S. M., Cox, B. R., & Asce, A. M. (2012). *A Comparison of MASW Dispersion Uncertainty and Bias for Impact and Harmonic Sources*.

Yasar, E., and Erdogan, Y. (2004). "Correlating sound velocity with density, compressive strength and Young' modulus of carbonate rocks." International Journal of Rock Mechanics and Mining Sciences, 41, pp 871-875.

Zywicki, D. J. (1999). Advanced signal processing methods applied to engineering analysis of seismic surface waves [Georgia Institute of Technology]. In *ProQuest Dissertations and Theses*. http://libproxy.uwyo.edu/login/?url=https://www.proquest.com/dissertations-theses/advanced-signal-processing-methods-applied/docview/304501274/se-2?accountid=14793