SURFACE GEOPHYSICS AS TOOLS FOR CHARACTERIZING EXISTING BRIDGE FOUNDATION AND SCOUR CONDITIONS

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ABSTRACT

Assessing foundation and scour conditions at existing bridges frequently requires geotechnical characterization of the subsurface. Characterization may be needed either to verify subsurface conditions as reported in the original bridge design investigations or to identify and characterize changes in subsurface conditions due to scour events. Sometimes, previous subsurface characterization for scour at older bridges has not been performed, cannot be located or was incomplete or inadequate for current design criteria. Surface geophysical methods provide economical means to profile relevant subsurface information, including depth to and competency of more dense or cohesive alluvial horizons and bedrock. Results from such surveys may be sufficient to verify previous investigations and existing conditions. Further investigation using invasive methods such as drilling, often into difficult access areas, may then be better focused or not be needed. This paper presents three case studies using surface geophysical methods to assess scour and foundation conditions at existing bridges in Arizona. One bridge, with five spans and driven pile foundations that recently suffered loss of an approach due to flooding, crosses a wide ephemeral streambed in the Basin and Range Physiographic Province. One old interstate bridge with two piers on footings and three spans that originally served the pre-existing rural highway, crosses an indistinct wash in the Colorado Plateau Physiographic Province. One more modern interstate bridge with three piers on spread footings, crosses an incised ephemeral stream in the Transition Zone between Basin and Range and Colorado Plateau. Geophysical methods utilized included seismic refraction for compression wave profiles of alluvium and bedrock, refraction microtremor for shear wave profiles in saturated alluvium and alluvium over bedrock profiles, and resistivity to indicate the presence of clays in coarse saturated alluvium. In these cases previous characterization of subsurface conditions was substantially confirmed and geophysical information on subsurface profiles and material strengths was obtained.

INTRODUCTION

Assessing and characterizing geotechnical conditions at existing roadway and highway bridges for scour conditions or general foundation adequacy can become complex and costly when difficult access, difficult ground conditions or regulatory constraints limit traditional exploratory methods. Even when traditional methods can be applied, results based on penetration testing and / or recovered samples may be of limited usefulness. When ground surface is available (not completely inundated), surface geophysical methods provide means, at a minimum, for subsurface preliminary or screening characterization under these conditions. Surface geophysics can provide information concerning subsurface geometry and relevant material properties. Case studies presented in this paper demonstrate the usefulness of three surface methods, seismic refraction, seismic refraction microtremor and four-point resistivity, to assist in characterization of the subsurface at existing bridges. The seismic refraction method can provide two-dimensional profile interpretations of subsurface horizon interface depths and compression wave (p-wave) velocities within those horizons (Rucker, 2000). Limits of the method include, requiring that p-wave velocities increase with progressively deeper horizons (no velocity reversals), p-wave velocities are profoundly effected by fluid saturation (water table), and the need for relative quiet in the field to collect data. The refraction microtremor (Remi) method (Louie, 2001; Optim, 2004a) can provide just one-dimensional interpretations of subsurface horizon interface depths and shear wave (s-wave) velocities within these horizons by utilizing surface waves (concepts of SASW, Spectral Analysis of Surface Waves). Remi makes interpretation of velocity reversal conditions possible, and can be performed in areas with relatively high ambient ground vibrations and noise. S-wave velocities and interface depths are also not impacted or only minimally impacted by fluid saturation. Electrical resistivity measurements are profoundly effected by the presence or absence of conductive fluids and clays. Clays tend to be very conductive and are consistent with low resistivities. Water is a poorly to moderately conductive, depending on the salts or other total dissolved solids contents, and in saturation is consistent with moderate resistivities. Most non-metallic soil or rock particles are very poor conductors or insulators, and without clays or considerable moisture, are consistent with high to very high resistivities.

CASE 1 – BRIDGE WITH POSSIBLE DAMAGE BY EXTREME FLOOD

In January 2005, a major flood in Beaver Dam Wash caused significant erosion to channel banks, the approach and south abutment of the Beaver Dam Wash Bridge on Mohave County Highway 91 in northwestern Arizona (AMEC, 2005a). As a result, the width of the channel at the bridge was widened by approximately 180 feet (55 m) at the south abutment. There were concerns related to the structural integrity of the bridge and the owner requested a thorough evaluation of the bridge. The existing structure, constructed in the 1950's, is a 2-lane bridge constructed of reinforced concrete and is approximately 330 feet (100 m) in length (Figure 1). The five-span structure is supported on driven H-piles at both abutment and pier locations. Geotechnical data contained within the as-built drawings was limited to basic soil descriptions and approximate depths at each boring location. At the time of the original investigation, the upper roughly 2 to 10 feet (0.6 to 3 m) of river channel deposit consisted of loose sand, overlaying 10 to 18 feet (3 to 5.5 m) of sand and gravel, with occasional sand and boulders at depth. Plans indicated that each boring was terminated 2 to 3 feet (0.6 to 0.9 m) into a clayey cobble and boulder deposit. Both abutments and piers were constructed on driven vertical and battered H-piles driven to depths ranging from 40 to 47 feet (12 to 14 m). At the abutments, sheet pile walls surrounding the abutments extended to 25 feet (7.6 m) below constructed grade. It is anticipated that the river channel elevations have varied somewhat since the surveys were performed in 1951 and, therefore, may not reflect existing conditions. Following the 2005 flood event, initial investigation at the bridge consisted of two borings advanced to depths of 22 and 27 feet (6.7 to 8.2 m). The soils encountered in the borings were typically sands and silty sands with some to considerable gravel, with relative density of encountered soils typically moderately dense to very dense in zones. The borings were terminated when auger and sample refusal occurred, and estimated depths of scour were highly varied between the borings. One boring was completed as a piezometer, and groundwater was recorded a depth of 1 to 2 feet (0.3 to 0.6 m).

***Figure 1**. Beaver Dam Wash Bridge (Piers 2, 3 and 4) in Northwest Mohave County, Arizona after winter 2005 flood. Surface seismic lines were completed between each set of piers and abutments. Surface resistivity was completed at a previously installed piezometer on a gravel bar in the northern portion of the stream channel bed.*

Broader coverage of the site subsurface conditions was then accomplished using surface seismic methods. Six combination seismic refraction (p-wave) and refraction microtremor or Remi (s-wave) surveys, were performed between piers and abutments under the bridge (with one survey upstream), with the intent of determining the extent of low-seismic velocity scourable materials within the channel. The seismic lines were completed utilizing a Geometrics S-12 signal enhancement engineering seismograph and an array with 10-foot spacings between 4.5 Hz geophones (Figure 2). Useful compression wave (p-wave) depths and velocities could not be attained because of the shallow depth to groundwater. P-wave depth and velocity interpretations were consistent with the top of the water table at a depth of 1 to 2 feet (0.3 to 0.6 m) and not with the subsurface profile. However, one-dimensional vertical shear wave (s-wave) profiles at each seismic line were obtained by utilizing traffic over the bridge and jogging alongside the geophone array to generate energy for Remi analysis. An electrical resistivity sounding utilizing a standard four-point Wenner array with electrode spacings of 5, 10, 20 and 30 feet (1.5, 3, 6 and 9 m) was completed at the previously installed PVC-cased piezometer near the north end of the bridge.

***Figure 2**. seismograph with 4.5 Hz geophone array deployed along active stream bank under bridge. P-wave data was collected using the sledgehammer energy source with the instrument set to 0.063 msec sample intervals and 256 msec sample period. S-wave data was collected using jogging alongside the array and traffic noise on the bridge with the instrument set to 1 msec sample intervals and a 12 second sample period.*

In general, a relatively low seismic velocity channel alluvium horizon with typical depths of 17 to 21 feet (5.2 to 6.4 m) was interpreted at the line locations across the site (Figure 3). S-wave velocities for this horizon were typically in the range of 360 to 370 feet per second (f/s) (110 to 113 meters/sec [m/s]) in the upper 10 to 13 feet (3 to 4 m) and increased to 630 to 830 f/s (192 to 253 m/s) to depths of 17 to 21 feet (5.2 to 6.4 m). These s-wave velocities were consistent with riverbed sands and loose to perhaps medium dense sand, gravel and cobble deposits. Given sufficient stream power, these deposits could be considered non-resistant, readily excavatable, and susceptible to scour, especially within about 10 to 13 feet (3 to 4 m) of the surface. Beneath the near-surface channel alluvium, at depths greater than about 17 to 21 feet (5.2 to 6.4 m), interpreted s-wave velocities increased to between 2,400 and 3,200 f/s (730 to 975 m/s). Results for this zone were consistent with a medium dense to dense clayey sand, gravel and cobble deposit. This deposit was anticipated to be moderately to strongly resistant to scour. Resistivity was used to determine the presence or absence of clays. Results of the resistivity sounding were interpreted as two layers; an upper horizon extending to a depth of about 19 feet 5.8 m) with a resistivity of about 10,000 ohm-cm (100 ohm-m), and an underlying lower horizon with a resistivity of about 2,500 to 3,300 ohm-cm (25 to 33 ohm-m). A saturated coarse cohesionless granular material without clay is consistent with the higher resistivity, and a moist to saturated clayey coarse granular material is consistent with the lower resistivity.

Sand, gravel and cobble (SGC) materials with s-wave velocities less than about 1,500 f/s (460 m/s) can typically be anticipated to be cohesionless in nature and to be relatively erodable, depending on largest particle size. Comparing erodability with excavatability, materials with s-wave velocities less than about 1,500 f/s (460 m/s) can be excavated with a rubber-tired backhoe. SGC materials with s-wave velocities greater than about 2,000 f/s (610 m/s) up to about 3,500 f/s (1,070 m/s) can be anticipated to exhibit cohesion from clays in the SGC matrix (Rucker, 2004). Such higher velocity SGC materials tend to be much stronger and much more erosion resistant. Subsurface materials with s-wave velocities greater than about 3,500 f/s (1,070 m/s) up to 6,000 f/s (1,830 m/s) or more can be anticipated to be very strongly cemented materials or bedrock. Based on the seismic Remi survey interpretations, review of the as-built construction drawings and refusal during geotechnical drilling of the site, bridge pile foundation and erodability parameters for the clayey sand, gravel and cobble deposit could be based on a "weak rock."

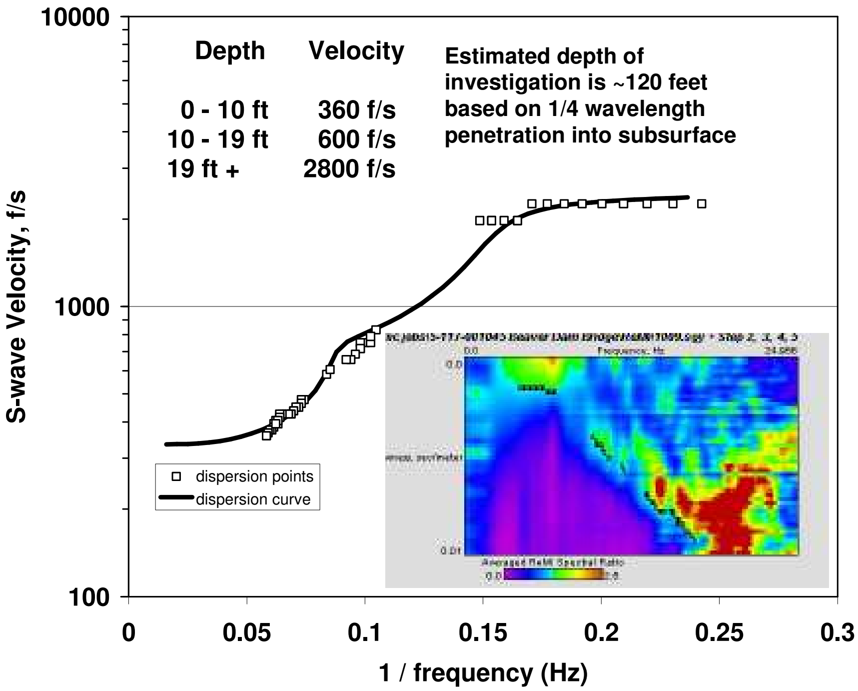

***Figure 3**. Typical interpreted Remi result at bridge in Figure 1. Dispersion point data was obtained from the insert; frequency increases from left to right and seismic velocity increases from bottom to top. A dispersion curve generated from the depth and s-wave velocity profile matches the point data well.*

SEISMIC VELOCITY, MATERIAL STRENGTH, AND EXCAVATABILITY AND ERODABILITY

Seismic propagation velocities, including p- and s-waves, are a function of the low-strain (dynamic) modulus or moduli of the material mass through which the seismic waves propagate. Since s-waves are minimally affected by fluid saturation, they are especially useful in characterizing material mass strength in the presence of fluid saturation, a common condition at bridge sites. Stronger (higher modulus) material masses are more resistant to excavation or erosion than weaker material masses. Estimated ranges of hydraulic stream power needed to initiate head cutting erosion in various strength cemented soil to soft or weak clastic rock geo-material masses are summarized in Table 1. Details presented in Table 1 may vary somewhat for other geo-material types. In Table 1, material mass strength is quantified through seismic velocity or the Kirsten Excavatability Index (Kirsten, 1982, 1986). Annandale (1995) reviews determination of stream power as a function of hydraulic flow conditions. For purposes of comparison and correlation, ranges of hydraulic stream power are compared to excavation equipment needed to perform effective excavation as estimated (quantified) by seismic p-wave velocities. Annandale (1995) presents relations of stream power to initiation of head cutting erosion through an erodability index based on the Kirsten excavatability index for geo-materials. Procedures to determine erodability index for geo-materials, initially for dam spillway design applications, have been established and published by the National Resource Conservation Service (NRCS, 2001) utilizing the Kirsten concepts and procedures.

TABLE 1
Approximate Erodability & Excavatability of Materials
Limestone & Cemented Soils (caliche)

Seismic Velocity f/s (m/s) (Rucker and Fergason, 2006)	**Trackhoe / Dozer Type & Power (Cat, 1984, 1993)**	**Erodability / Excavatability Index (Kirsten 1982, 1986; NRCS, 2001)**	**Erosion Threshold Stream Power, KW/m^2 (Annandale, 1995)**
s-wave < 750 f/s (230 m/s) p-wave < 1,500 f/s (460 m/s)	Hand spade	< 0.01	Very erodable
s-wave 750 – 1,500 (230 – 460) p-wave 1,500 – 3,000 (460 – 910)	Hand pick & spade	0.01 – 0.099	Very erodable – 0.2
s-wave 1,500 - ~1,800 (460 – 550) p-wave 3,000 - ~3,500 (910 – 1,070)	Cat 325BL 168 hp 125 KW Cat D6D 136 hp 101 KW	0.1 – 0.99	0.2 – 1.0
s-wave ~1,800 – 2,000 (550 – 610) p-wave ~3,500 – 4,000 (1,070 – 1,220)	Cat 330BL 222 hp 165 KW Cat D7G 200 hp 149 KW	1.0 – 9.99	1.0 – 5.0
s-wave ~2,100 – 3,000 (640 – 910) p-wave ~4,200 – 5,900 (1,280 – 1,800)	Cat 345BL 321 hp 239 KW Cat D8L 335 hp 249 KW	10 – 99	5.0 – 30
s-wave 3,000 – 3,600 (910 – 1,100) p-wave 5,900 – 7,200 (1,800 – 2,200)	Cat 375 428 hp 319 KW Cat D9L 460 hp 342 KW	100 – 999	30 – 200

***Table Notes**: Bulldozer and backhoe power ranges are presented by Kirsten (1982, 1988) as a measure of equivalent performance for excavation. All velocities are approximate and represent a typical range. S-wave velocities are assumed to be about half of p-wave velocities consistent with a Poisson's ratio of 0.33. Seismic velocity ranges for backhoes and trackhoes in cemented soils with typical p-wave velocity less than 6,000 f/s (1,830 m/s) are from Rucker and Fergason (2006). See the Caterpillar Performance Handbook (Caterpillar, 1984, 1993 or current edition) for details on use of seismic information for rippability. Different model configurations include variations in weight and horsepower.*

Correlations of seismic velocity with other geotechnical parameters useful for foundation design or assessment have also been performed (Rucker and Fergason, 2006) as presented in Figure 4. Because of the potential for velocity reversals that could hide underlying lower velocity (lower strength) horizons from p-wave seismic refraction, such correlations need to be based on s-wave velocity profiles where significant velocity reversals can be identified and quantified.

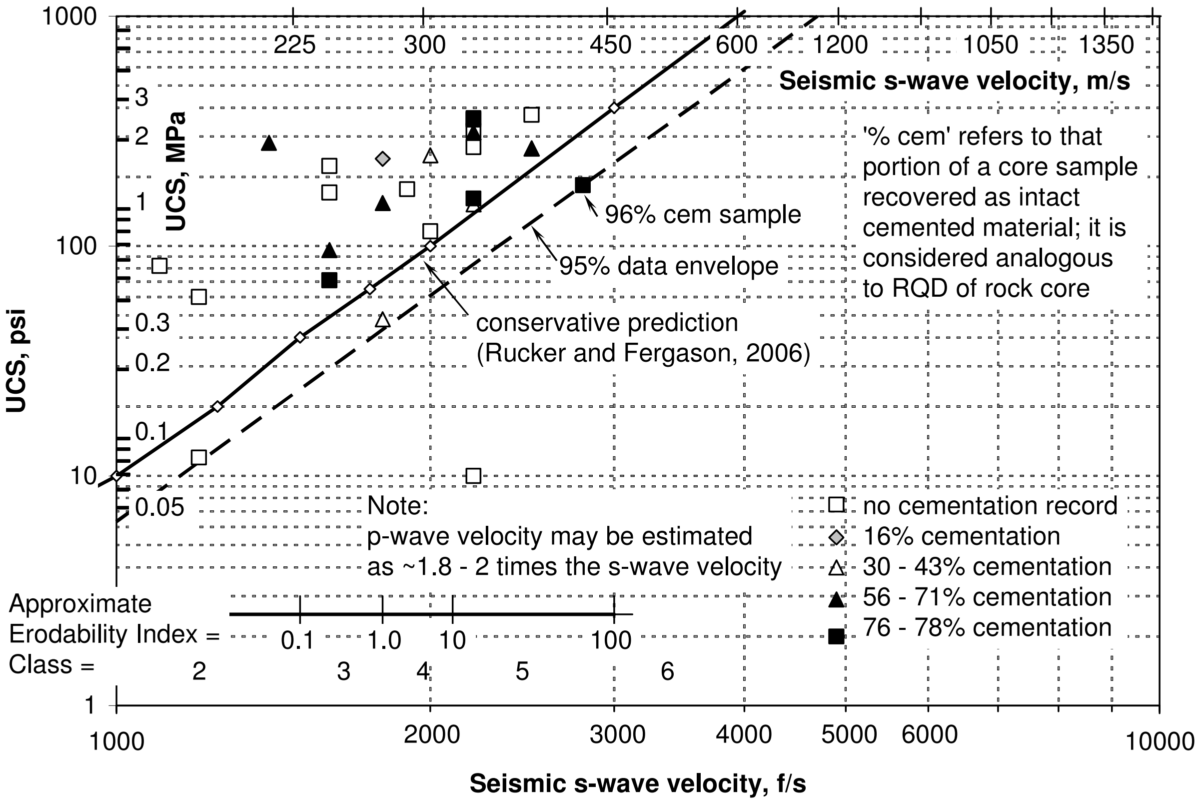

***Figure 4**. Unconfined compressive strength (UCS) values from core samples in cemented soils and rock-like materials in the Salt River Valley in central Arizona with overlapping s-wave velocities (Rucker and Fergason, 2006). The solid line shows a conservative relation between s-wave velocity and UCS; 95% of the data lie at or above the s-wave velocity - UCS trend at the dashed line.*

CASE 2 – RE-ASSESSING CONDITIONS AT AN INTERSTATE BRIDGE OVER AN INCISED DRAINAGE

Surface seismic was utilized in a geotechnical re-evaluation of pier foundation conditions (AMEC, 2005b) of the existing northbound Little Squaw Creek Bridge (figure 5) on Interstate I-17 south of Rock Springs in central Arizona. The purpose of this investigation was to re-evaluate pier foundation conditions compared to the original design investigation, evaluate potential scour depths at the bridge, and provide particle size distributions of streambed materials for use in hydraulic and bridge foundation analysis as part of a scour retrofit for the bridge. Three combination seismic refraction and Remi lines were performed in order to determine the depth to bedrock and characterize the resistance to erosion of the bedrock at Piers 1 and 2 straddling the active stream channel, and characterize the extent of low seismic velocity / scourable materials adjacent to Piers 1 and 2 and within the existing stream channel between these piers. The seismic lines were completed utilizing the Geometrics S-12 seismograph and geophone arrays with geophone spacings of 5 feet (1.5 m). Energy sources were sledgehammer for p-wave data and jogging and ambient highway traffic for Remi data. Surface particle scan lines for coarse particle size distribution and bulk samples for particle size analysis were also obtained.

Piers 1 and 2 straddled the channel of Little Squaw Creek. Pier 1 was on the south side of the stream channel about 7 to 10 feet (2.1 to 3 m) from a steep outcrop of Precambrian Schist forming the south boundary of the creek channel. An active channel in the stream center was about 20 feet (6 m) wide and had a small amount of water flowing in it. This active channel consisting of gravels, cobbles and small boulders was considerably coarser than the adjacent channel floodplain bank deposits that consisted primarily of sand, gravel and cobbles. To the north of Pier 2 towards Pier 3, fill was observed to form the north boundary of the stream channel. Piers 3 and 4 were not of concern in this investigation.

Figure 5. *Little Squaw Creek Bridge, constructed in the 1960's, is a 2-lane, 5-span, 315-foot (96 m) long bridge constructed of reinforced concrete. The structure is supported on spread footings at both the south abutment and pier locations. Seismic lines were completed at or between Piers 1 and 2 in or near the active channel. Equipment was hiked into the small canyon from the vehicle parked by the highway.*

In general, a relatively low seismic velocity channel alluvium horizon with typical interpreted depths of 1 to 3 feet (0.3 to 0.9 m) was interpreted at the line locations across the site. P-wave velocities were typically in the range of 600 to 1,300 f/s (180 to 400 m/s), and s-wave velocities were typically in the range of 300 to 500 f/s (90 to 150 m/s). These p-wave and s-wave velocities were consistent with riverbed sands and loose to perhaps medium dense sand, gravel and cobble deposits that are considered non-resistant, readily excavatable and susceptible to scour.

Figure 6. *Starting location of the seismic line adjacent to Pier 2. Results of this line are presented in Figure 7. Note very coarse particles in the streambed. A scan line using a 100-foot (30 m) cloth tape was performed at this location to provide a very coarse particle size distribution.* D_{50} *and* D_{95} *for the surface particles were 0.5 and 11.2 inches (12.5 and 284 mm), respectively. The largest particle counted had a b-axis dimension of 17.3 inches (440 mm).*

Beneath the near-surface channel alluvium in the floodplain at Piers 1 and 2, interpreted p-wave velocities increased to about 2,000 to 3000 f/s (610 to 910 m/s) and s-wave velocities increased to about 1,200 f/s (370 m/s). Results for this zone were consistent with a medium dense to dense sand, gravel and cobble (SGC) deposit that, depending on available stream power, could be moderately resistant to scour due to the large particle sizes in the deposit.

Underlying the SGC deposit at Pier 1 and the surface deposit in the active channel, bedrock with high velocities was interpreted. P-wave velocities ranged from 5,700 to 9,600 f/s (1,740 to 2,930 m/s) and were underlain by more competent bedrock beginning at depths of about 6 to 10 feet 1.8 to 3 m) with p-wave velocities of 11,000 to 15,000 f/s (3,400 to 4,600 m/s). Corresponding s-wave velocities ranged from 2,600 to 3,500 f/s (790 to 1,070 m/s). The relatively low s-wave velocities compared to the very high p-wave velocities indicated that fractures and joints in the underlying rock mass may have been saturated with groundwater. Such saturation could be anticipated to increase p-wave velocities over unsaturated fractured, jointed rock, but would have little effect on s-wave velocities.

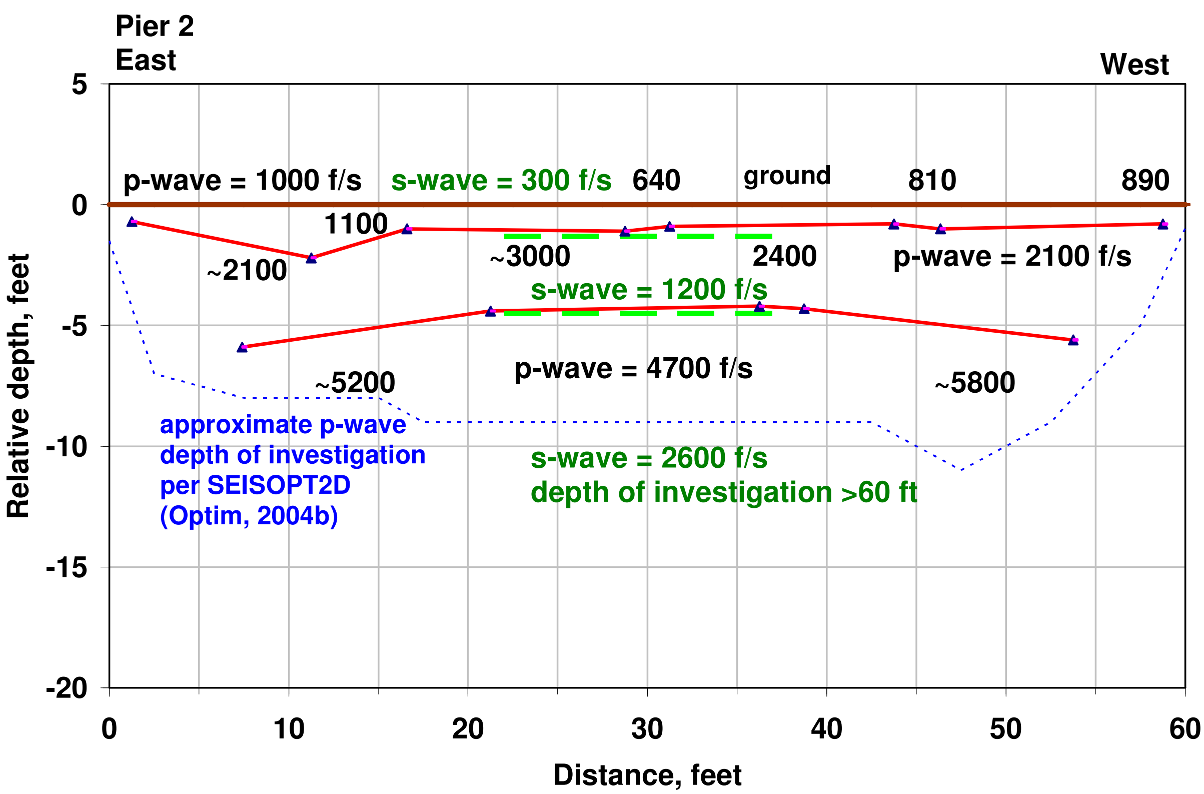

***Figure 7**. Seismic profile at Pier 2. Depths and distances are in feet, and p-wave and s-wave velocities are in feet/second. The horizon at 1 to 5 foot (0.6 to 1.5 m) depth (p-wave 2,100 to 3,000 f/s, s-wave 1,200 f/s; 640, 910 and 370 m/s, respectively) is consistent with SGC above water table. Below 5 foot (1.5 m) depth (p-wave 4,700 to 5,800 f/s, s-wave 2,600 f/s; 1,400, 1,800 and 790 m/s, respectively) is consistent with fractured, weathered bedrock or clayey SGC.*

Underlying the SGC deposit at Pier 2, rock or rock-like material with p-wave velocities of 4,700 to 5,800 f/s (1,400 to 1,800 m/s) was interpreted. Such p-wave velocities could indicate relatively competent material or a saturated SGC material below the groundwater table. Corresponding s-wave velocities underlying the SGC deposit increased to about 2,600 f/s (790 m/s). The increase in s-wave velocity indicated that relatively competent material was present underlying the SGC and that the p-wave velocity increase was not due solely to groundwater.

Based on sketches of existing pier foundation conditions and the results of the seismic investigation, it appeared that the spread footing of Pier 1 was founded on rock (p-wave velocity ~11,000 f/s or 3,400 m/s, s-wave velocity ~2,600 f/s or 790 m/s) and the spread footing of Pier 2 was founded on rock or rock-like material (p-wave velocity ~5,000 f/s or 1,500 m/s, s-wave velocity ~2,600 f/s 790 m/s). Shallow rock (p-wave velocity ~7,300 to 9,600 f/s or 2,200 to 2,900 m/s, s-wave velocity ~3,500 f/s or 1,070 m/s) was present under the thin gravel cobble material in the active channel. Based on the geophysical interpretations, it was estimated that the upper roughly 4 to 6 feet (1.2 to 1.8 m) of material at Piers 1 and 2 were cohesionless, primarily coarse granular materials that may be subject to scour if sufficient hydraulic energy is applied. Rock or rock-like materials underlying the coarse cohesionless SGC horizon, and upon which Piers 1 and 2 are founded, are anticipated to be much more resistant to scour erosion.

CASE 3 – OLDER BRIDGE AT MINOR EPHEMERAL DRAINAGE ON INTERSTATE HIGHWAY

Surface seismic was utilized for a geotechnical evaluation of pier foundation conditions eastbound Babbitt Tanks Wash Bridge (Figure 8) in the vicinity of Meteor Crater on Interstate I-40 east of Flagstaff in northern Arizona. The purpose of this investigation was to evaluate pier foundation conditions and evaluate potential scour depths at the bridge across an indistinct ephemeral drainage, and provide particle size distributions of streambed materials for use in hydraulic and bridge foundation analysis as part of a scour assessment for the bridge (AMEC, 2006). The existing bridge, constructed in the 1940's, is a 2-lane, 3-span, 80 foot (24 m) long reinforced concrete structure supported on spread footings at both the abutments and two pier locations. No geotechnical data concerning foundation conditions was available beyond a "probable solid rock line" on the plans.

***Figure 8**. East-bound Babbitt Tanks Wash Bridge is in foreground looking north. Piers 1 and 2 are located in or adjacent to the active channel of Babbitt Tanks Wash. Based on the 1944 plans, pier foundations are three-foot (0.9 m) wide strip footings that are 18 inches (0.46 m) thick. Plans showed footing bottoms set approximately six inches (0.15 m) below the "Probable Solid Rock Line."*

The site is located within Paleozoic Age sedimentary formations consisting of Kaibab limestone in the Colorado Plateau Physiographic Province. Bedrock exposures commonly outcrop among a mostly thin mantle of residual, colluvium and alluvium through the area. The active channel upstream of the bridge consisted primarily of sands and finer materials interspersed between areas of exposed rock outcrops, with some gravel and cobbles exposed at the surface. Downstream from the bridge in the median area, bedrock was exposed in the active channel between the eastbound and westbound bridges.

Figure 9. *Surface seismic setup under bridge. Unlike the typical channel upstream, considerable coarser material was exposed at the surface under the bridge. The tops of the pier footings were generally exposed under the bridge but were covered at ends of the piers. The channel bottom appeared to be several inches higher than the tops of the footings; the ground surface grade and sloped to the footings.*

Two combination seismic refraction and refraction microtremor lines were performed at selected locations near Piers 1 (Figure 9) and 2 in the active channel under the bridge. The seismic lines were performed in order to determine the depth to bedrock and characterize the resistance to erosion of the bedrock, and characterize the extent of low seismic velocity / scourable materials adjacent to the piers and within the existing stream channel between the piers. The seismic lines were completed utilizing the Geometrics S-12 seismograph and 4.5 Hz geophone arrays with geophone spacings of 5 feet (1.5 m). Energy sources were sledgehammer for p-wave data and jogging and ambient highway traffic for Remi data. Surface particle scan lines for coarse particle size distribution and bulk samples for particle size analysis were also obtained.

Based on as-built plans of existing pier foundation conditions and the seismic results, it appeared that the spread footings of Piers 1 and 2 were founded on relatively soft rock with p-wave velocities ranging from 3,300 to 5,000 f/s (1,000 to 1,500 m/s) and s-wave velocities of about 2,100 to 3,100 f/s (640 to 940 m/s). Results near Pier 1 are presented in Figure 10. It can be anticipated that a rubber-tired backhoe with excavation equipment less than 100 hp or 75 KW (such as Case 590 Super M at 99 hp or a Cat 416c at 78 hp) will encounter refusal in geo-materials with p-wave velocities of about 3,000 to 3,300 f/s or 910 to 1,000 m/s (Stacy and Noble, 1975; Rucker and Fergason, 2006). As summarized in Table 1, geo-materials with progressively higher p-wave velocities require more powerful equipment for effective excavation. At a p-wave velocity of 5,000 f/s (1,500 m/s), it is anticipated that heavy equipment with available power greater than 300 hp or 220 KW (such as a Cat D8L at 335 hp or Cat 345BL at 321 hp) would be needed for effective excavation. Thus, excavation to 'refusal' for bottom of footings would be consistent with placing the footings on soft rock. Soft rock consisting of limestone can be considered to be similar in behavior to strongly cemented soils or caliche as described in Rucker and Fergason (2006).

SUMMARY

These three case studies illustrate kinds of subsurface profile and material strength parameters that can be obtained using surface geophysics. Field geophysical operations in all three cases required one man-day or less effort, not including mobilization to the (frequently remote) sites. As experience develops in the characterization and application of quantitative strength-based erodability parameters, surface methods, especially (modulus characterizing) seismic methods, will be become ever more effective tools for bridge scour screening and characterization evaluations.

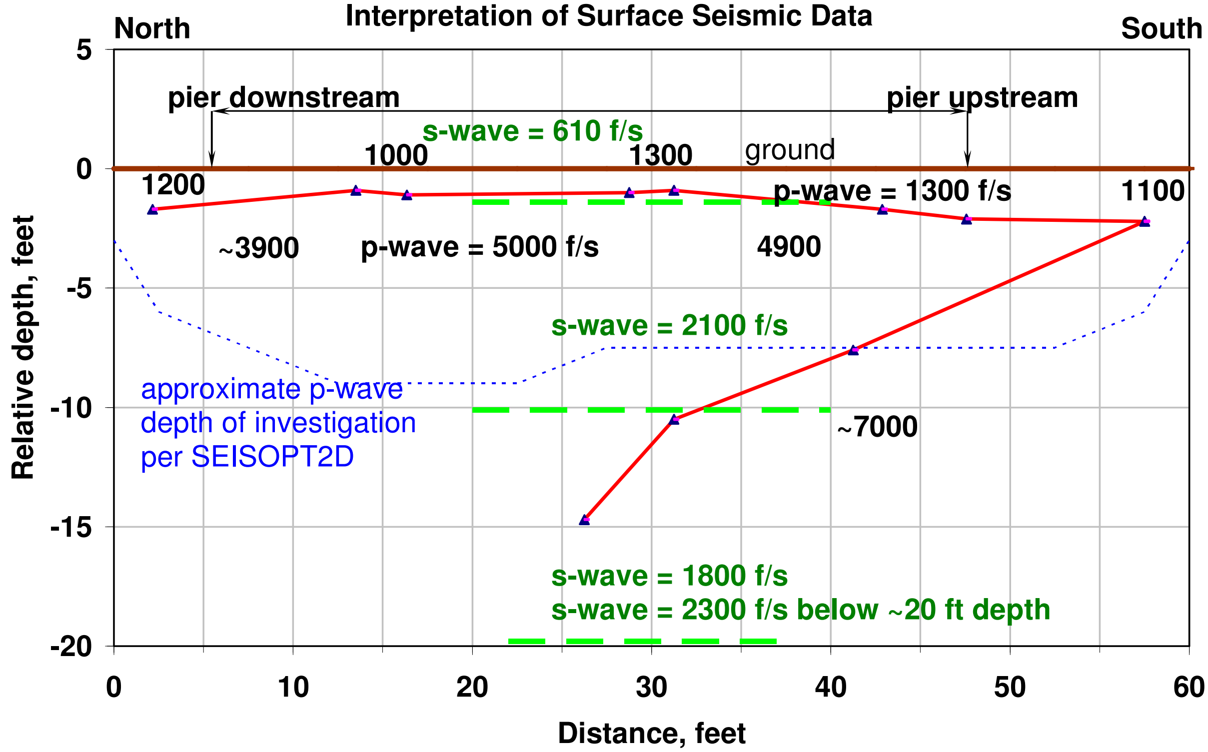

Figure 10. *Interpreted seismic profile at line shown in Figure 9 with depths and distances in feet and p- and s-wave velocities in feet/second. P-wave velocities in the scour resistant horizon originally characterized as "probable solid rock" varied across the profile and were lower towards the downstream end of the pier. The interpreted s-wave velocity, being limited to a one-dimensional profile, reflected the more erodable lower p-wave velocity in that horizon. A somewhat lower s-wave velocity horizon was interpreted beginning at a depth of about 10 feet (3 m).*

REFERENCES

AMEC, 2005a, Geotechnical Investigation Report, Beaver Dam Wash Bridge, Mohave County Highway 91, Mohave County, Arizona, AMEC Job No. 4-117-001045, June 17, 2005

-----2005b, Geotechnical Support Services, Little Squaw Creek Bridge Scour Retrofit, I-17, MP 239.2, Rock Springs, Arizona, AMEC Job No. 5-117-001055, June 20, 2005

-----2006, Scour Potential Evaluation, Babbitt Tanks Wash Bridge, Interstate (I-40), MP 225, Near Flagstaff, Arizona, Coconino County, Arizona, AMEC Job No. 6-117-001012, April 27, 2006

Annandale, G.W., 1995, Erodability: Journal of Hydraulic Research, **33**(4), 471-494.

Caterpillar Tractor Company, (Cat), 1984, Caterpillar Performance Handbook, Edition 15, Peoria, Illinois, October.

---------------1993, Caterpillar Performance Handbook, Edition 24, Peoria, Illinois, October.

Kirsten, H.A.D., 1982, A classification system for excavation in natural materials: Civil Engineer in South Africa, **24**(7), 293-308.

-----------1988, Case histories of groundmass characterization for excavatability, *in* Kirkaldie, L, Rock Classification Systems for Engineering Purposes, ASTM STP 984: American Society for Testing and Materials, Philadelphia, Pennsylvania, 102-120.

Louie, J.L., 2001, Faster, better: Shear-wave velocity to 100 meters depth from refraction microtremor arrays: Bulletin of the Seismological Society of America, **91**, 347-364.

NRCS, 2001, Field procedures guide for the headcut erodability index, Chapter 52, Part 628, National Engineering Handbook: U.S. Department of Agriculture Natural Resources Conservation Service, 210-VI-NEH, rev. 1, March.

Optim L.L.C., 2004a, ReMi Version 3.0 software package: Optim Software and Data Solutions, UNR-MS-174, 1664 N. Virginia St., Reno, Nevada, 89557.

-----------2004b, Seisopt@2D v4.0 software package: Optim Software and Data Solutions, UNR-MS-174, 1664 N. Virginia St., Reno, Nevada, 89557.

Rucker, M.L., 2000, Applying the seismic refraction technique to exploration for transportation facilities; 1st International Conference on the Application of Geophysical Methodologies to Transportation Facilities and Infrastructure: St. Louis, Missouri, FHWA, December 11-15.

-----------2004, Percolation theory approach to quantify geo-material density-modulus relationships; 9th ASCE Specialty Conf. on Probabilistic Mechanics and Structural Reliability: Albuquerque, New Mexico, July 26-28.

Rucker, M.L. and Fergason, K.C., 2006, Characterizing unsaturated cemented soil profiles for strength, excavatability and erodability using surface seismic methods, *in* Miller, G.A., Zapata, C.E. Houston, S.L. and D.G. Fredlund, Unsaturated Soils 2006, Geotechnical Special Publication No. 147: ASCE, Reston, Virginia, 589-600.

Stacy, T.R. and Noble, G.W., 1975, Seismic evaluation of soil trenchability; Proceedings of the Sixth Regional Conference for Africa on Soil Mechanics and Foundation Engineering: **2**, 185-186.