FINAL GEOTECHNICAL EXPLORATION REPORT
WASH BRIDGE
WALNUT GROVE ROAD, MP 7.7 TO 7.9
ADOT TRACS NO. T0414 01D
FEDERAL AID NO. YYV-0(212)T
ADOT CONTRACT NO. 2022-006.01
YAVAPAI COUNTY, ARIZONA

Prepared for:

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Prepared by:

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Ethos Project No. 2023099
August 21, 2024

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JESSE R
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ARIZONA, U.S.A.

August 21, 2024
Ethos Project No.: 2023099

Ben Ansley, PE
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SUBJECT: Final Geotechnical Exploration Report
Wash Bridge
Walnut Grove Road, Milepost 7.7 to 7.9
ADOT TRACS No. T0414 01D
Federal Aid No. YYV-0(212)T
ADOT Contract No. 2022-006.01
Yavapai County, Arizona

Dear Ben:

Ethos Engineering, LLC (Ethos) is pleased to present the results of a geotechnical investigation performed for the Wash Bridge project in Yavapai County, Arizona. Our scope of services was performed in general accordance with our proposal dated November 3, 2023 (Revision 2). The results of our field investigation and geotechnical engineering recommendations for support of the planned improvements are presented herein.

We appreciate the opportunity to be of service to AECOM Technical Services, Inc. (AECOM) on this project. If you have any questions regarding this report, please do not hesitate to contact us.

Sincerely,
Ethos Engineering, LLC

Registered Professional Engineer (Civil)
CERTIFICATE NO.
47791
JESSE R
HUSTON
Date Signed 8/21/2024
ARIZONA, U.S.A.

Jesse Huston, P.E.
Principal/Senior Geotechnical Engineer

Reviewed By:

Registered Professional Engineer (Civil)
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1.0 PROJECT DESCRIPTION

The project will include replacement of the existing Wash Bridge along Walnut Grove Road (also known as Wagoner Road) between milepost (MP) 7.7 and MP 7.9 in Yavapai County, Arizona. The existing narrow bridge was constructed in 1936 and is now considered structurally deficient.

The existing bridge will be replaced with a reinforced concrete box culvert (RCBC) constructed under the existing bridge. Once the RCBC is constructed, the existing bridge will be removed in phases and the adjacent roadway approaches will be constructed. We understand the existing bridge foundations will be protected in place to facilitate construction of the new RCBC. The RCBC will include approximately 6 feet of soil cover above the RCBC and will support the new roadway. We understand the new pavement section at the approaches will be reconstructed with asphalt concrete and aggregate base.

2.0 FIELD EXPLORATION

2.1 Field Coordination and Permitting

Prior to our field exploration, Ethos prepared a Field Investigation Plan (FIP) to document the planned field exploration, and for use in environmental clearance and permitting. Ethos obtained Yavapai County right-of-way use permit #ROW24-000282 dated June 5, 2024 for the field exploration. Ethos staked the planned boring locations in the field and coordinated utility clearance of each location with Arizona 811. A traffic control plan was prepared for this work by Bullway Barricades Co. LLC as a subcontractor to Ethos.

2.2 Subsurface Exploration – Borings

The subsurface exploration with borings was performed on June 6, 2024 and included one boring advanced to a depth of 30 feet. The boring, identified as B-1, was performed in the southbound shoulder just north of the existing bridge. The boring location is shown on the Site Map Showing Test Locations included as Figure 1 attached to this report. A log of the boring is presented in Appendix A1.

Drilling was performed by ACS Services LLC (ACS) using a truck-mounted CME-75 drill rig equipped with hollow-stem auger. A representative bulk sample of the subgrade soils were obtained from the drill cuttings at the boring location. Drive sampling was performed using standard penetration test (SPT) split spoon samplers or open-end drive samplers (2.42-inch-diameter brass rings) at maximum 5-foot intervals in each boring using a calibrated automatic hydraulic-actuated 140-pound hammer, free falling 30 inches. The hammer efficiency is noted in the heading of each boring log. The SPT and ring samplers were driven 18 and 12 inches, respectively. Unless noted otherwise on the boring log, the sample penetration resistance was recorded as number of blows per six inches of penetration. The penetration results are presented on the boring log adjacent to each sample.

The recovered soil samples were removed from the sampler, sealed to reduce moisture loss, and stored for subsequent review and laboratory testing. Upon completion, the boring was backfilled to the surface with drill cuttings. Groundwater was not encountered in the boring.

Encountered soils were visually inspected, labeled and classified in the field, and logged in general accordance with ASTM D2488, the Unified Soil Classification System (USCS), Arizona Department of Transportation (ADOT), and Ethos guidelines. Field direction, and logging of borings were performed by Ethos personnel.

2.3 Subsurface Exploration – Geophysical Evaluation

A geophysical evaluation was performed to supplement the boring data and better evaluate the depth to rock at the bridge site. One geophysical test line was performed in the wash and abutting the west side of the existing bridge. The geophysical survey was performed by Advantage Geophysics, Inc. (Advantage), on June 6, 2024, as a subconsultant to Ethos. The geophysical evaluation included a two-dimensional seismic p-wave refraction study. The geophysical test line location, identified as SL-1, is shown on the Site Map Showing Test Locations included as Figure 1 attached to this report. The results of the geophysical evaluation along with additional details of the work are presented in the Geophysical Evaluation report included in Appendix A2.

3.0 LABORATORY TESTING

Selected laboratory tests were assigned by Ethos and performed by ACS on representative samples recovered from the borings to support our field classification and to provide information regarding engineering characteristics and properties of the subsurface materials. Table 3.1 lists the laboratory tests assigned for the project.

Table 3.1: Laboratory Testing Program

Item/Description	Number of Tests
Grain Size Analysis (Total - Coarse and Fine) - ASTM C136 & C117	2
Atterberg Limits (Plasticity Index) - ASTM D4318	2
Moisture Content - ASTM D2216	2
In-Place Dry Density - ASTM D2937	2
Moisture-Density (standard Proctor) - ASTM D698, Method A	1
pH and Resistivity - AZ Method 236e	1
Sulfates and Chlorides - AZ Method 733b	1
Direct Shear - ASTM D3080	1

4.0 SITE CONDITIONS AND GEOTECHNICAL PROFILE

4.1 Site Conditions

The Wash Bridge site is located along Walnut Grove Road at the crossing with a generally east-west oriented, unnamed ephemeral wash that drains surface water from the hills to the west of the site and into the Hassayampa River located just east of the site. Walnut Grove Road is two-lane undivided rural roadway with an asphalt pavement surface.

The existing bridge is single-lane slab bridge. The roadway surface elevation at the wash crossing is approximately 3,825 feet, with the underlying wash surface elevation around 3,810 feet. Vegetation within the area includes moderate desert shrubs and trees along the existing wash.

4.2 Geologic Setting

The project site is in the Basin and Range Geologic Province. Published statewide geologic mapping indicates the surficial geologic units consist of Pliocene- to middle Miocene-aged deposits (Richard et al 2000). These deposits are described as moderately to strongly consolidated conglomerate and sandstone deposited in basins during and after late Tertiary faulting. These deposits include lesser amounts of mudstone, siltstone, limestone, and gypsum. These deposits are generally light gray or tan. They commonly form high rounded hills and ridges in modern basins, and locally form prominent bluffs.

4.3 Generalized Subsurface Profile

Based on the conditions encountered in the boring, the results of the geophysical survey, and the site geology, a generalized subsurface profile was developed for the Wash Bridge site and is presented in Table 4.1. Refer to the boring log and geophysical results in Appendices A1 and A2, and laboratory testing in Appendix B, for additional details. The grain sizes of sand and gravel particles indicated on the boring log are representative of the predominant grain sizes based on laboratory testing and visual inspection.

Table 4.1: Generalized Subsurface Profile at Wash Bridge

Stratum	Approximate Bottom Elevations (feet)	Approximate Bottom Depth (feet)	USCS Material Types	Relative Firmness / Density
1	3,790	35	Silty Sand and variable amounts of gravel (SM)	Medium Dense
2	3,775 (maximum depth explored)	50+	Sandstone and/or Conglomerate (estimated)	Hard/Very Dense

Notes:

(1) Elevations and depths referenced to Walnut Grove Road surface elevation at 3,825 feet.

(2) Stratum 1 & 2 transition elevation/depth estimated from geophysical results.

4.4 Site Seismicity

The project seismic AASHTO Load and Resistance Factor Design (LRFD) criteria were determined in accordance with Section 3.10 of the AASHTO LRFD Bridge Design Specifications [American Association of State Highway and Transportation Officials (AASHTO)] (AASHTO, 2012). The horizontal design acceleration is defined as having a 7% chance of exceedance during a 75-year recurrence interval. Based on the conditions encountered in the boring and results of the geophysical evaluation, a Site Class D is applicable for the Wash Bridge.

The probabilistic horizontal spectral acceleration values for the designated return period and corresponding peak horizontal ground acceleration (PGA) were obtained from the U.S. Geological Survey seismic hazards program website (USGS, 2009). The resulting seismic design values are presented in Table 4.2.

Table 4.2: Summary of Seismic Parameters

Seismic Design Parameter	Value
Latitude	34.31893º
Longitude	-112.57148º
Site Class	D
Peak Ground Acceleration (PGA)	0.083g
Short Period Acceleration (S_S)	0.192g
Long Period Acceleration (S_1)	0.056g
Site Coefficient, F_{PGA}	1.6
Site Coefficient, F_a	1.6
Site Coefficient, F_v	2.4
Spectral Acceleration, A_s	0.132g
Spectral Acceleration, S_{DS}	0.307g
Spectral Acceleration, S_{D1}	0.134g
Seismic Zone	1

4.5 Moisture and Groundwater Conditions

Groundwater was not encountered in the boring during the field investigation. The moisture condition of the soils encountered in the boring were described as slightly moist to moist.

A review of groundwater data in the Arizona Department of Water Resources (ADWR) Groundwater Site Inventory (2023) database does not include readings from index well locations in the vicinity of the site. Several nearby non-index ADWR wells indicate historic groundwater readings with depths on the order of 2 to 25 feet. The potential for an elevated groundwater surface and/or surface flows within the wash during times of precipitation were considered for design and should be considered during construction of the project.

5.0 ENGINEERING ANALYSES AND RECOMMENDATIONS

5.1 General

The following sections of this report present our recommendations regarding foundation design for the planned RCBC and related geotechnical engineering considerations. The planned structure may be supported on shallow spread footings provided scour protection is incorporated into the design (as necessary, determined by AECOM). These recommendations are based on our understanding of the project, the results of the field exploration and laboratory testing performed for the project, and engineering analyses.

5.2 Spread Footings

Spread footings will be utilized to support the RCBC planned to replace the existing bridge. We understand scour protection in the form of cut-off wall(s) will be incorporated into the design as determined by AECOM. The strength and service limit state design analyses for spread footings were completed per the methods presented in Sections 10.5 and 10.6, respectively, of AASHTO (2012), and ADOT Geotechnical Design Policy SF-1 (2010a). Based on information provided by AECOM, we understand the RCBC will have an approximate length of 32 feet, approximate width of 25 feet, and approximate invert elevation of 3,811 (+/- 2 feet).

A spread footing bearing resistance chart was developed for the planned RCBC and is presented in Appendix C. Development of the bearing resistance chart is discussed in the following sections.

5.2.1 Strength Limit State

The factored net bearing resistance, q_{Rn}, at the strength limit state was determined using the net nominal bearing resistance (ultimate bearing capacity), q_{nn}, calculated per Section 10.6.3.1.2a and bearing resistance factor, φ_b, from Section 10.5.5.2.2 of AASHTO (2012) for spread footings on level ground. The parameters presented below in Table 5.1 were used for the nominal resistance and strength limit state analyses.

Table 5.1: Strength Limit State Design Parameters

Parameter	Symbol	RCBC
		Value
Soil Angle of Internal Friction [degrees]	ϕ_f	30
Soil Total Unit Weight [pcf]	γ_{total}	115
Cohesion [psf]	c	0
Maximum Footing Length [feet]	L	32
Footing Bearing Depth [feet]	D_f	1
Effective Footing Width [feet]	B_f	20 to 30
Bearing Resistance Factor [dim]	φ_b	0.45

Notes: pcf – pounds per cubic foot; psf – pounds per square foot; dim – dimensionless.

5.2.2 Service Limit State

Per ADOT Geotechnical Design Policy SF-1 (2010a), the modified Schmertmann method presented in Section 8.5 of the Federal Highway Administration Soils and Foundation Reference Manual (Samtani and Nowatzki 2006) was used to calculate settlements at the service limit state. The parameters are based on the measured soil densities, N values at and below the bearing elevations, and elastic modulus (E_s) to N value correlations from Kulhawy and Mayne (1990). The bearing resistance charts present the family of service limit state curves developed per ADOT Geotechnical Design Policy SF-1 (2010a) for design settlements of 0.25, 0.5, 0.75, 1.0, 1.5 and 2.0 inches.

5.2.3 Sliding Resistance

The factored sliding resistance, R_R, for limit state design should be determined using the nominal sliding resistance between soil and foundation, R_τ, and nominal passive resistance, R_{ep}, per Section 10.6.3.4, and corresponding resistance factors, φ_τ and φ_{ep}, from Section 10.5.5.2.2 of AASHTO LRFD (2012). We recommend the parameters presented in Table 5.2 be used for analyzing sliding resistance.

Passive lateral soil resistance should be neglected in the upper 3 feet of finished grade due to the potential for disturbance. Below a depth of 3 feet, the nominal passive resistance can be estimated assuming a hydrostatic pressure distribution of 360 psf per foot.

Table 5.2: Parameters for Sliding Resistance of Spread Footings

Parameter	Symbol	Value
Factored Sliding Resistance		
Resistance Factor for Shear between Soil and Foundation	φ_τ	0.80
Resistance Factor for Passive Resistance	φ_{ep}	0.50
Nominal Sliding Resistance		
Soil Angle of Internal Friction	ϕ_f	30 degrees
Soil Total Unit Weight	γ	115 pcf
Cohesion	c	0
Passive Earth Pressure Coefficient	K_p	3.0

5.2.4 Eccentricity

The eccentricity in the L (long) dimension of an abutment or wall footing is typically negligible, such that L = L'. The effective footing length (B') in the B (short) dimension is calculated as B' = B – $2e_B$, where e_B is the B dimension eccentricity determined by the structural engineer. The maximum allowable eccentricity at the strength limit state should be calculated in accordance with ADOT Geotechnical Design Policy SF-2 (ADOT 2010b).

5.3 Lateral Earth Pressures

Structures retaining soils should be designed for the lateral earth pressure imposed by the soils. The magnitude of the lateral earth pressure is a function of the backfill material, imposed surcharge loads, drainage accommodations and the rigidity of the retaining structure. The recommended lateral earth pressure values presented herein assume the backfill will be structure backfill comprised of granular soils which meet the requirements of Section 203 of the ADOT Standard Specifications (ADOT 2021). The limits of structure backfill should extend a minimum of 3 feet laterally from the back edge of all structure walls.

Walls which are free to deflect a minimum of 0.2 percent of the wall height should be designed for the full active earth pressure condition and an active equivalent fluid unit weight on the order of 35 psf per foot of wall height. Walls which are restrained from lateral movement should be designed for the at-rest condition using an equivalent fluid unit weight of 55 psf per foot of wall height.

The lateral earth pressures presented herein assume a horizontal backfill surface and do not include hydrostatic pressure or surcharge loadings which should be incorporated into the structural design in addition to the earth pressure loading. Vertical surcharge loads (e.g., traffic loading) should be added to the above earth pressures after multiplying them by an earth pressure coefficient of 0.30 for active conditions, and 0.46 for at-rest conditions. These values are based on an internal friction angle of 33 degrees for the structure backfill.

5.4 Slopes

5.4.1 Permanent Slopes

Permanent cut and fill slopes should have configurations no steeper than 3:1 (H:V). These values represent the maximum allowable slope configurations. Flatter slopes will promote re-vegetation and can accept landscaping.

5.4.2 Temporary Slopes

Temporary slopes should be excavated in accordance with OSHA (2020). In accordance with Subpart P, Appendix A, the existing embankment soils and native soils to an approximate depth of 20 feet are considered to be Type C soils. For excavations less than 20 feet in such soils, Subpart P, Appendix B indicates a maximum allowable unshored slope of 1.5H:1V for Type C soils. Flatter slopes may be required where either sandy soils are encountered or where the soils become excessively wet, and soft.

Should steeper slopes be required due to the proximity of existing structures or other contractor needs, the stability of the slopes should be verified by a registered geotechnical engineer (State of Arizona) who is proficient in slope stability analyses. Based on the overall site conditions, it does appear that steeper slopes would be feasible in cohesive and/or cemented soil layers across the project, pending further analysis of specific locations and excavations.

The perimeter of all excavations should be protected against water runoff and infiltration near the edges to maintain stability. Heavy equipment and spoil piles should not be allowed within 10 feet of the edge of the excavation. The perimeter of all excavations should be protected against water runoff and infiltration near the edges to maintain stability.

5.5 Pavements

The subgrade materials encountered in the boring consist of silty sand with gravel. In general, these soils will provide good support for pavements at the site. Subgrade preparation beneath pavements should be completed as outlined in the Earthwork section of this report. We understand the new pavement section at the approaches will include 3 inches of asphalt concrete over 8 inches of aggregate base to meet the minimum structural section required by Yavapai County for a Minor Collector Roadway. No specific pavement thickness design was performed.

5.6 Surface Drainage

Long-term performance of pavements and structures will require that the subgrade soils and backfill be protected against excessive water infiltration and/or saturation. Surface drainage should be established away from foundations and pavements to minimize moisture infiltration into the subgrade. Structural fill and backfill should be well compacted to reduce possible moisture infiltration through loose soil intervals.

5.7 Preliminary Soil Corrosion or Degradation Potential

5.7.1 Metal in Contact with Soil

The corrosion potential of near-surface soils on corrugated metal pipes was characterized using laboratory pH and electrical resistivity testing, performed on one sample in accordance with Arizona Test Method 236. The results indicate a laboratory pH value of 8.5 resistivity value of 4,190 ohm-centimeters (ohm-cm). It is recommended that the type and/or coating of metal in direct contact with soil be selected in accordance with ADOT Pipe Selection Guidelines (ADOT, 1996). Pipe locations where the pH is greater than 9.0 and/or the resistivity is less than 2,000 ohm-cm require the use of special pipes and/or pipe coatings. None of the samples tested had a pH value greater than 9.0 or resistivity value less than or greater than 2,000 ohm-cm. Therefore, specialized piping or other corrosion mitigation measures such as corrosion monitoring don't appear necessary for metallic pipes. The individual test results are included in Appendix B.

5.7.2 Concrete in Contact with Soil

One sample from the current investigation was tested for soluble sulfates and chlorides (Arizona Test Method 733 and Arizona Test Method 736) to support design of concrete structures. The test results are included in Appendix B and summarized in Table B-1.

The tested total soluble sulfate content was 6 parts per million (ppm). The sulfate test measures the water-leachable or "available" sulfate content. These results were compared to Table 19.3.1.1, "Exposure Categories and Classes," in Section 19.3.1, of the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete* (ACI 2019). The sample falls within Exposure Class S0 for water-soluble sulfate (SO_4^{2-}) in soil by percent mass (SO_4<0.1% or 1,000 ppm) and are categorized with a severity level of "not applicable" in terms of sulfate exposure. Based on ACI Table 19.3.2.1, "Requirements for Concrete by Exposure Class," in Section 19.3.2 (ACI 2019), there is no restriction on Portland cement type for concrete structures in contact with these materials.

The tested chloride content was 19 ppm. Regarding chloride attack, Section 19.3.2 (ACI 2019) indicates that when concrete is exposed to external sources of chlorides, concrete should be proportioned to satisfy the requirements for the applicable exposure class in Table 19.3.1.1 (ACI 2019). A majority of the samples for within Exposure Class C1. Table 19.3.2.1 (ACI 2019) should be referred to for requirements for concrete by exposure class. For Exposure Class C1, the minimum compressive strength of concrete specified is 2,500 psi and the maximum water-soluble chloride ion content in concrete, by percent weight of cement, is 0.30% for non-prestressed concrete and 0.06% for prestressed concrete.

5.7.3 Further Evaluation

The results presented in this section are general in nature and may not be representative of site conditions. We recommend that the results of our laboratory testing be reviewed by a person or firm experienced in corrosion protection designs for the actual construction at the site, and/or by the appropriate pipe or material manufacturer. A qualified corrosion engineer should be consulted if corrosion of underground utilities is a concern or if a detailed evaluation is necessary.

5.8 Earthwork

The following earthwork recommendations are intended to provide support for the proposed new pavements, RCBC, slopes, and associated embankments. The recommendations presented in this report are contingent upon performing the earthwork recommended herein. The grading activities at the site should be performed under observation and testing directed by a geotechnical engineer.

5.8.1 Site Preparation

Completely remove all vegetation (including roots) and other organics, debris, any unstable (soft, loose, disturbed, water softened, etc.) soils, any uncontrolled fill, structural elements not intended to remain, and other deleterious materials from proposed pavement, embankment and structure areas prior to construction. This site grading should extend laterally a minimum of 2 feet beyond pavement, embankment and structure areas unless noted otherwise. All areas of excavation should be observed and approved by a representative of the geotechnical engineer after clearing and before any filling operations begin at the site.

5.8.2 Subgrade Preparation

For all areas, prior to placement of fill, aggregate base and/or concrete, the exposed subgrade should be scarified to a minimum depth of 6 inches, adjusted to a moisture content within the range of plus or minus 2 percent of optimum, and compacted to at least 95% of maximum dry density as determined by the applicable ADOT test methods.

5.8.3 Fill Materials and Placement

In general, the existing silty sand soils are considered suitable for reuse as embankment fill throughout the project. With the exception of aggregate base, all import soils (if any) should be reviewed and approved by the Engineer prior to being hauled to the site.

Earthwork construction should be in accordance with Section 203 of the ADOT Standard Specifications (ADOT, 2021) and the project's Special Provisions. Fill material should be placed in loose lifts no thicker than 12 inches where heavy compaction equipment is used, provided compaction can be achieved throughout the lift thickness. Where hand operated compactors are used, loose lifts should not exceed 6 inches in thickness. Fill lifts should be of uniform thickness when compacted. All fill should be compacted to a minimum of 95% of the maximum dry density at within plus or minus 2% of the optimum moisture content as determined per ASTM D698.

Cobbles may be encountered during earthwork construction. Cobbles larger than 6 inches should not be placed within 3 feet of finished subgrade elevation. At no time should cobbles be nested together. There should be a sufficient amount of finer material to fully encapsulate cobbles.

5.8.4 Structure Backfill

The limits of structure backfill placement are assumed to be the entire limits of excavations for the RCBC and wing walls. In all cases the structure backfill should extend a minimum of 3 feet laterally from the back edge of all walls as shown on ADOT Drawing No. SD 5.02.

The structure backfill material should meet the requirements of Section 203 of the ADOT Standard Specifications (ADOT 2021) and those shown on ADOT Drawing No. SD 7.01. All structure backfill should be moisture conditioned to within 2 percent of the optimum moisture content and compacted to a minimum of 100 percent of the maximum ASTM D698 Standard Proctor density.

5.8.5 Earthwork Factors

Development of earthwork factors for near-surface materials was based on the evaluation and analysis of data from density tests on in-situ soil samples and laboratory moisture-density relationships. The recommended earthwork factor for on-site materials was calculated using the following equation:

$$\% \text{ Shrink/Swell} = \left[1 - \frac{\gamma_{ex}}{\gamma_{fill}}\right] 100$$

Where:

γ_{ex} = in situ dry density of material to be excavated
γ_{fil} = 95 percent of the maximum dry density

The moisture-density relationships (Proctor) were developed using ASTM D698, Method A on single sample. The results indicated the maximum dry density of the site soils was 135.0 pcf (with rock correction). In-situ dry density was determined on a near surface soil sample. The results indicated an in-situ dry density of 111.0 pcf.

Using 95 percent of the average maximum dry density and the average tested in situ dry density, the following calculation was made:

γ_{fill} = 0.95*(135.0) = 128.3 pcf
γ_{ex} = 111.0 pcf
% Shrink/Swell = $\left[1 - \frac{111.0}{128.3}\right] 100$ = 8.7%.

Positive results indicate shrink and negative results indicate swell. Based on the positive result, an average value of 8.7 percent shrink is anticipated. For estimating, we recommend a shrinkage value of 10 percent be utilized for the project.

5.8.6 Ground Compaction Factor

The ground compaction factor is an estimate of the ground height loss, including clearing and grubbing, that will result from compaction of the surface to 95% relative compaction. The impact of clearing and grubbing is a function of the existing ground cover. The existing ground cover included moderate vegetation in existing wash.

The impact from compaction is related to the fill placement height. In general, the proposed finished grades are anticipated to be near the current roadway grade. However, the finished width of the new wash crossing will be slightly wider than the existing narrow bridge, which may require embankment fills on the order of 10 to 15 feet in height adjacent to the existing bridge.

Based on these factors, the recommended ground compaction factor for the project is 0.20 feet. Settlement of the planned fill embankments and of the near-surface native soils is anticipated in response to embankment construction; however, most settlement is expected to occur during construction of the embankments.

5.8.7 Excavation

Based on the conditions encountered in the borings, excavations are anticipated to be achievable using conventional earthmoving equipment. Although the bedrock was not encountered in the boring or interpreted within 10 feet of the wash bottom along the seismic test line, the area is mapped with shallow bedrock at the surface and as such, a bedrock high could be encountered during excavation for the new RCBC. If encountered, excavations into these materials will require increased excavation effort.

6.0 CLOSURE

The geotechnical services were performed in a manner consistent with that level of care and skill ordinarily exercised by other members of the geotechnical profession practicing in the same locality, under similar conditions and at the date the services were provided. Our conclusions, opinions and recommendations are based on the completed test boring, visual observations and the review of plans prepared by others. It is possible that conditions could vary beyond the data evaluated. Ethos makes no guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and their representatives, and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on site and off site), or other factors may change over time, and additional work may be required with the passage of time. Any party other than the Client who wishes to use this report shall notify Ethos of such intended use. Based on the intended use of the report, Ethos may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Ethos from any liability resulting from the use of this report by any unauthorized party.

7.0 REFERENCES

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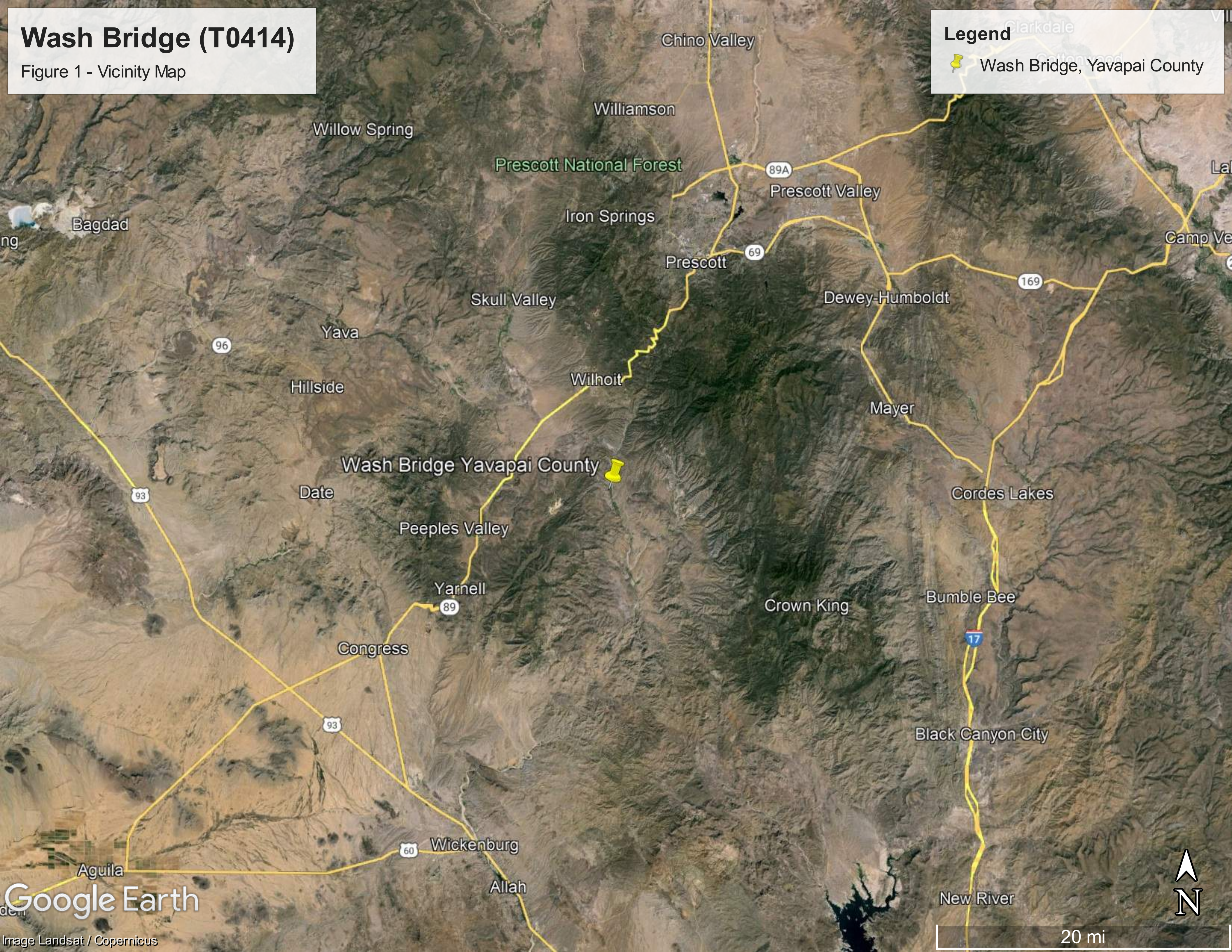
FIGURES

Wash Bridge (T0414)

Figure 1 - Vicinity Map



Wash Bridge (T0414)

Figure 2 - Site Map Showing Test Locations

APPENDIX A1

Field Exploration Results – Boring Logs

SOILS SAMPLING & BORING LOG INFORMATION

The material and in-situ moisture descriptions of soils presented on the boring logs are based on visual observation and classification in accordance with the Unified Soil Classification System (USCS), presented on the next page. The field logs were modified, where appropriate, based on laboratory testing of selected samples.

The relative density and firmness described on the test boring logs are generally based on standard penetration test (SPT) blows per foot (N) for mostly cohesionless and cohesive soils. 2-inch outside diameter (O.D.) SPT samplers are advanced up to 18 inches into undisturbed soils beyond the base of either a hollow stem auger or drill casing. The samplers are driven with a 140-pound hammer and a 30-inch drop. SPT values are recorded on the boring logs for each 6-inch increment of penetration with sampler refusal based on a penetration of less than 6 inches and a blowcount of 50.

Relative Density

Relative density for mostly cohesionless, uncemented sands and sand and gravel mixtures is described based on the following SPT blowcounts:

N	Relative Density
0-4	Very Loose
5-10	Loose
11-30	Medium Dense
31-50	Dense
>50	Very Dense

Relative Firmness

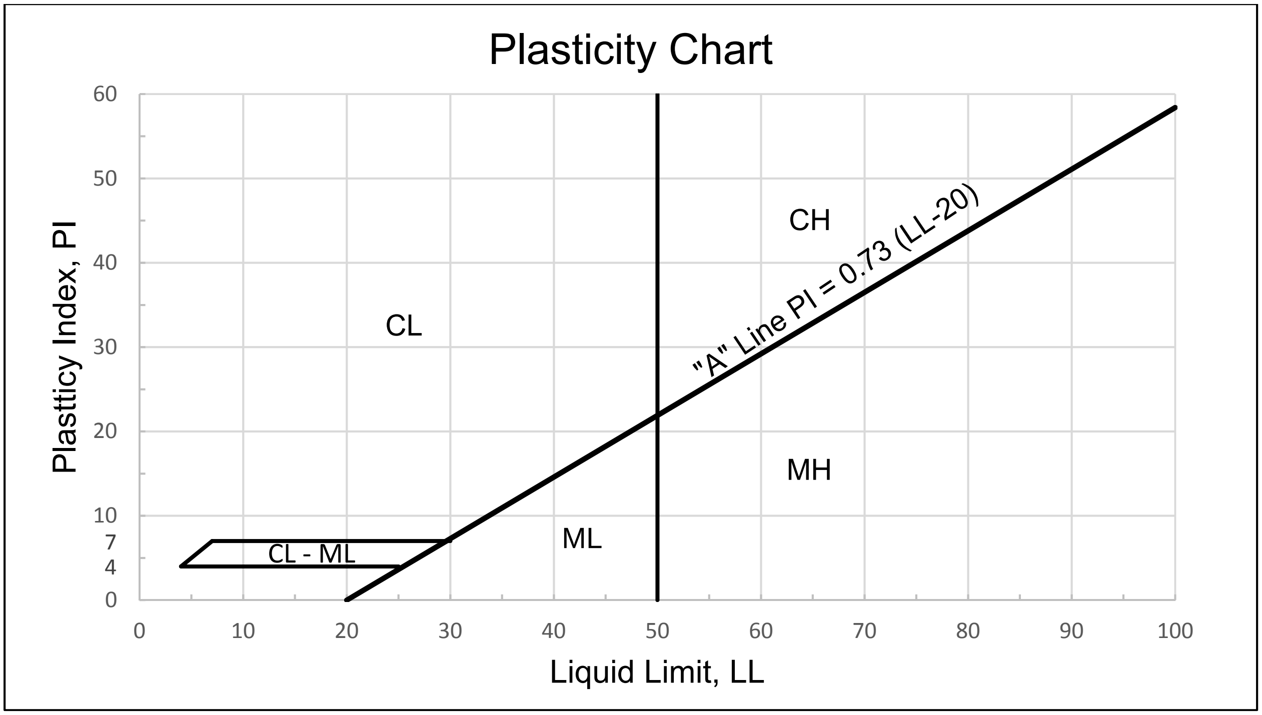
Relative Firmness for cohesive and/or cemented soils including silts, clays and silty to clayey sandy and gravelly soils is described based on the following SPT blowcounts:

N	Relative Firmness
0-4	Very Soft
5-8	Soft
9-15	Moderately Firm
16-30	Firm
31-49	Very Firm
50+	Hard

Undisturbed samples of firmer soils, typically present in the southwest, are obtained with 3-inch O.D. samplers lined with 2.42-inch inside diameter (I.D.) brass rings. The samplers are advanced up to 12 inches into undisturbed soils beyond the base of either a hollow stem auger or drill casing. The samplers are driven with a 140-pound hammer and a 30-inch drop. The N value blowcounts are recorded on the boring logs for each 6-inch increment of penetration with sampler refusal based on a penetration of less than 12 inches and a blowcount of 100.

Unified Soil Classification System
(ASTM D2487)

<table>
<tr><td colspan="4">Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests</td><td>Group Symbol</td><td>Group Description</td></tr>
<tr><td rowspan="8">Coarse-Grained Soils (More than 50% Retained on No. 200 Sieve).</td><td rowspan="4">Gravels More than 50% of Coarse Fraction Retained on No. 4 Sieve</td><td colspan="2" rowspan="2">Clean Gravels Less than 5% Fines</td><td>GW</td><td>Well Graded Gravels, Gravel-Sand Mixtures or Sand-Gravel-Cobble Mixtures.</td></tr>
<tr><td>GP</td><td>Poorly Graded Gravels, Gravel-Sand Mixtures or Sand-Gravel-Cobble Mixtures.</td></tr>
<tr><td rowspan="2">Gravels with More than 12% Fines</td><td>Fines Classify as ML or MH</td><td>GM</td><td>Silty Gravels, Gravel-Sand-Silt Mixtures</td></tr>
<tr><td>Fines Classify as CL or CH</td><td>GC</td><td>Clayey Gravels, Gravel-Sand-Clay Mixtures</td></tr>
<tr><td rowspan="4">Sands 50% or More of Coarse Fraction Passes No. 4 Sieve</td><td colspan="2" rowspan="2">Clean Sands Less than 5% Fines</td><td>SW</td><td>Well Graded Sands, Gravelly Sands.</td></tr>
<tr><td>SP</td><td>Poorly Graded Sands, Gravelly Sands.</td></tr>
<tr><td rowspan="2">Sands with More than 12% Fines</td><td>Fines Classify as ML or MH</td><td>SM</td><td>Silty Sands, Sand-Silt Mixtures</td></tr>
<tr><td>Fines Classify as CL or CH</td><td>SC</td><td>Clayey Sands, Sand-Clay Mixtures</td></tr>
<tr><td rowspan="4">Fine-Grained Soils (50% or More Passes No. 200 Sieve).</td><td rowspan="2">Silts and Clays (Liquid Limit less than 50)</td><td colspan="2">PI > 7 and Plots on Above “A” Line</td><td>CL</td><td>Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays</td></tr>
<tr><td colspan="2">PI <4 or Plots Below “A” Line</td><td>ML</td><td>Inorganic Silts, Clayey Silts with Low Plasticity</td></tr>
<tr><td rowspan="2">Silts and Clays (Liquid Limit 50 or More)</td><td colspan="2">PI Plots on Above “A” Line</td><td>CH</td><td>Inorganic Clays of High Plasticity, Fat Clays, Silty and Sandy Clays of High Plasticity</td></tr>
<tr><td colspan="2">PI Plots Below “A” Line</td><td>MH</td><td>Inorganic Silts of High Plasticity, Silty Soils, Elastic Silts</td></tr>
</table>



Angularity	
Angular	
Subangular	
Subrounded	
Rounded	

Soil Particle Definitions	
Material	Particle Size Range
Boulders	Greater than 300 mm (12 in.)
Cobbles	300 mm to 75 mm (12 in. to 3
Coarse Gravel	75 mm to 19 mm (3 in. to ¾ in.)
Fine Gravel	19mm (3/4 in.) to No. 4 sieve
Coarse Sand	No. 4 Sieve to No. 10 Sieve
Medium Sand	No. 10 Sieve to No. 40 Sieve
Fine Sand	No. 40 Sieve to No. 200 Sieve
Fines (Silt or Clay)	Less than No. 200 Sieve

Plasticity	
PI = 0	Non-Plastic
1 ≤ PI ≤ 7	Low
8 ≤ PI ≤ 25	Medium
PI ≥ 25	High

Moisture
Slightly Moist
Moist
Wet
(Saturated)

ETHOS ENGINEERING, LLC.

Wash Bridge

Soil Boring: B-1

Client Name:	AECOM Technical Services	Project Address:	Yavapai County	Project No:	2023099
Date Started:	06/06/2024	Date Completed:	06/06/2024	Hole Depth:	30'
Station/Offset:	- / - -	Surface Elevation:	3824'	Lat/Long:	34.3191, -112.57148
Contractor:	ACS	Operator:	Noel	Method:	Auger
Rig Type:	CME-75 / Blue 237727	Hammer Type:	Auto	Hammer Efficiency:	82%
Logged By:	K. Baker	Reviewed By:	J. Huston	Water Level At Time Of Drilling:	N/A

Elevation (ft)	Depth (ft)	Graphic Log	Soil Description and Remarks	Samples: Bulk	Samples: Driven	Samples: Blow Counts/6"	Lab Results: Moisture Content (%)	Lab Results: Dry Density (PCF)	Lab Results: Atterberg Limits (LL-PL-PI)	Lab Results: Fines (%)
	0–11.0		**SILTY SAND WITH GRAVEL** (SM), considerable predominantly fine subrounded gravel, fine to coarse sand, nonplastic, brown, slightly moist, loose to medium dense	Bulk (1–5 ft)					NP	21
	2				●	7-12	4.1	111		
3820	5				●	4-3				
3815	9				☒	4-6-9 (15)				
	11.0		**SILTY SAND** (SM), trace to some fine to coarse subrounded gravel, predominantly fine sand, nonplastic, brown, slightly moist, medium dense							
3810	14				☒	5-6-7 (13)	4.4		NP	22
3805	19				☒	13-12-10 (22)				
3800	24				☒	6-8-9 (17)				
3795	29				☒	7-8-11 (19)				
	30.5									

Drilling stopped at 29'. Sampler stopped at 30'6". Backfilled with cuttings.

APPENDIX A2

Field Exploration Results – Geophysics Report

Advantage Geophysics, Inc.
Gilbert, Arizona
San Diego, California
Phone: 602.688.9146
www.advantagegeophysics.com

June 12, 2024
Report No. 1

Mr. Jesse Huston
Ethos Engineering
9180 South Kyrene Road, #104
Tempe, AZ 85284

Subject: **Geophysical Evaluation, Seismic Refraction**
ADOT Contract No. 2022-006.01, Task TBD
ADOT TRACS No. T0414
Wash Bridge (#08229)
Yavapai County, Arizona
Project No. 2024021

Dear Mr. Huston:

In accordance with your authorization, Advantage Geophysics, Inc. (AGI), has performed a geophysical evaluation pertaining to the Yavapai County Wash Bridge (#08229) site near Kirkland Junction, Arizona (Figure 1). The purpose of our evaluation is to develop a subsurface velocity profile of the location evaluated, and to evaluate the apparent rippability and velocities of the subsurface materials at your specified location along the wash perpendicular to Wagoner Road. Based on our discussions with you, a bridge replacement with concrete box culverts is planned near the geophysical evaluation location. This data letter report presents our methodology, equipment used, analysis, and findings. Our seismic refraction field services were conducted on May 31, 2024.

SCOPE OF SERVICES

Our scope of services for the project included:

- Performance of a single seismic refraction traverses, SL-1 t at the project site (Figure 2).
- Compilation, processing, and analysis of the collected seismic data.
- Preparation of this illustrated data letter report presenting our geophysical evaluation results.

SITE AND PROJECT DESCRIPTION

The project site is located about 1 mile northwest of the historic Hassayampa River Bridge near Kirkland Junction, Arizona (Figures 1 and 2). Seismic refraction traverse SL-1 was conducted at your specified location west of the existing single lane Wash Bridge (#08229) within an existing relatively narrow wash drainage and approximately perpendicular to the Wagoner Road alignment (Figure 2). Figure 3 depicts the general site conditions in the evaluated area.

Based on our discussions with you, it is our understanding that the depth of evaluation requested is up to approximately 30 to 40 feet below existing ground surface. Our seismic evaluations were designed to develop information up to approximately 45 feet below ground surface (bgs) at the SL-1 location with the actual depths resolved dependent on subsurface site conditions at each location and on other variables.

EVALUATION METHODOLOGY AND ANALYSIS

A seismic P-wave (compression wave) refraction study was conducted at the project site to develop subsurface velocity profiles of the areas studied, and to assess the apparent rippability of the subsurface materials. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones and recorded with a 24- channel Geometrics StrataView seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

A single seismic traverse at SL-1 was conducted in the study area (Figure 2). The general locations and length of the seismic line were determined by surface conditions, site access, existing surface obstructions, your requested line location, and your requested depth of evaluation. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint. In general, classical seismic refraction theory requires that subsurface velocities increase with depth (generalized reciprocal method [GRM] and time-intercept modeling). In classical analysis methods a layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity such as those caused by core stones, intrusions, or boulders can also result in the misinterpretation of the subsurface conditions. However, in general the application of seismic tomography data collection and analysis methods performed for this project are not subject to these limitations.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree "hardness." Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2021), as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that soil and rock characteristics, such as degree of soil cementation, presence of cobbles and boulders, and rock fracture spacing and orientation, play a significant role in determining rippability or rock quality. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

For trenching operations, the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of cobbles and boulders, which can be troublesome in narrow trenching operations, should be anticipated.

It should be noted that the rippability cutoffs presented in the table are slightly more conservative than those published in the Caterpillar Performance Handbook (Caterpillar, 2021). Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids. Ripping is still more art than science, and much will depend on operator skill and experience.

DATA PROCESSING

As previously indicated, a single seismic traverse was conducted as part of our study. The collected data were processed and analyzed using Rayfract® Version 4.06 (Intelligent

Resources Inc., 2024). Rayfract® uses first arrival picks and elevation data to produce subsurface velocity models by the Wavepath Eikonal Traveltime (WET) method (Schuster, 1993). The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of subsurface conditions.

RESULTS

The results from our tomographic analysis are presented in Figure 4. As depicted, the tomography model reveals distinct relatively low velocity materials in the near-surface and generally relatively higher velocity materials at depth. The relatively lower velocity materials are possibly alluvium and weathered conglomerate. The relatively higher velocity materials at depth might represent weathered to unweathered conglomerate or other sedimentary formational material or bedrock. Also evident in the tomography models are substantial and distinct lateral variations in velocity which may be related to variations in depths to caliche cemented soils, variations in depths to cobble and boulder bearing alluvium deposits, possible intrusions, bedrock fracturing, bedrock pinnacles, differential weathering, varying depths to bedrock, and/or a combination of these factors and features regarding the subsurface materials.

Based on the seismic refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area. Furthermore, blasting may be required depending on the excavation depth, location, equipment used, and desired rate of production. In addition, oversized materials should be expected. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment, and production rate.

LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluations will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Advantage Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use of or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have questions related to this report, please contact the undersigned at your convenience.

Respectfully submitted,
Advantage Geophysics, Inc.

REGISTERED GEOLOGIST
CERTIFICATE NO.
52449
MARK D.
EDWARDS
Date Signed 6/12/24
ARIZONA, U.S.A.
Expires 06/30/26

Mark Edwards, RG (Arizona)
Owner/Principal Geologist/Geophysicist
medwards@advantagegeophysics.com
602-688-9146 (office)

Frederico Tiago Diogo

Frederico Diogo
Senior Project Geophysicist
fdiogo@advantagegeophysics.com

MDE:FD:mde

Attachments:	Figure 1 –	Site Location Map
	Figure 2 –	Seismic Line Location Map
	Figure 3 –	Site Photograph
	Figure 4 –	Seismic Profile, SL-1

Distribution: jhuston@ethosengineers.com

SELECTED REFERENCES

Arizona Geological Survey (AZGS), 2000, Geologic Map of Arizona, Arizona. Arizona Geological Survey Digital Geologic Maps, Map-35, 1 map sheet, map scale 1:1,000,000, https://library.azgs.arizona.edu/item/AGMS-1552428422626-489, accessed June 5, 2024.

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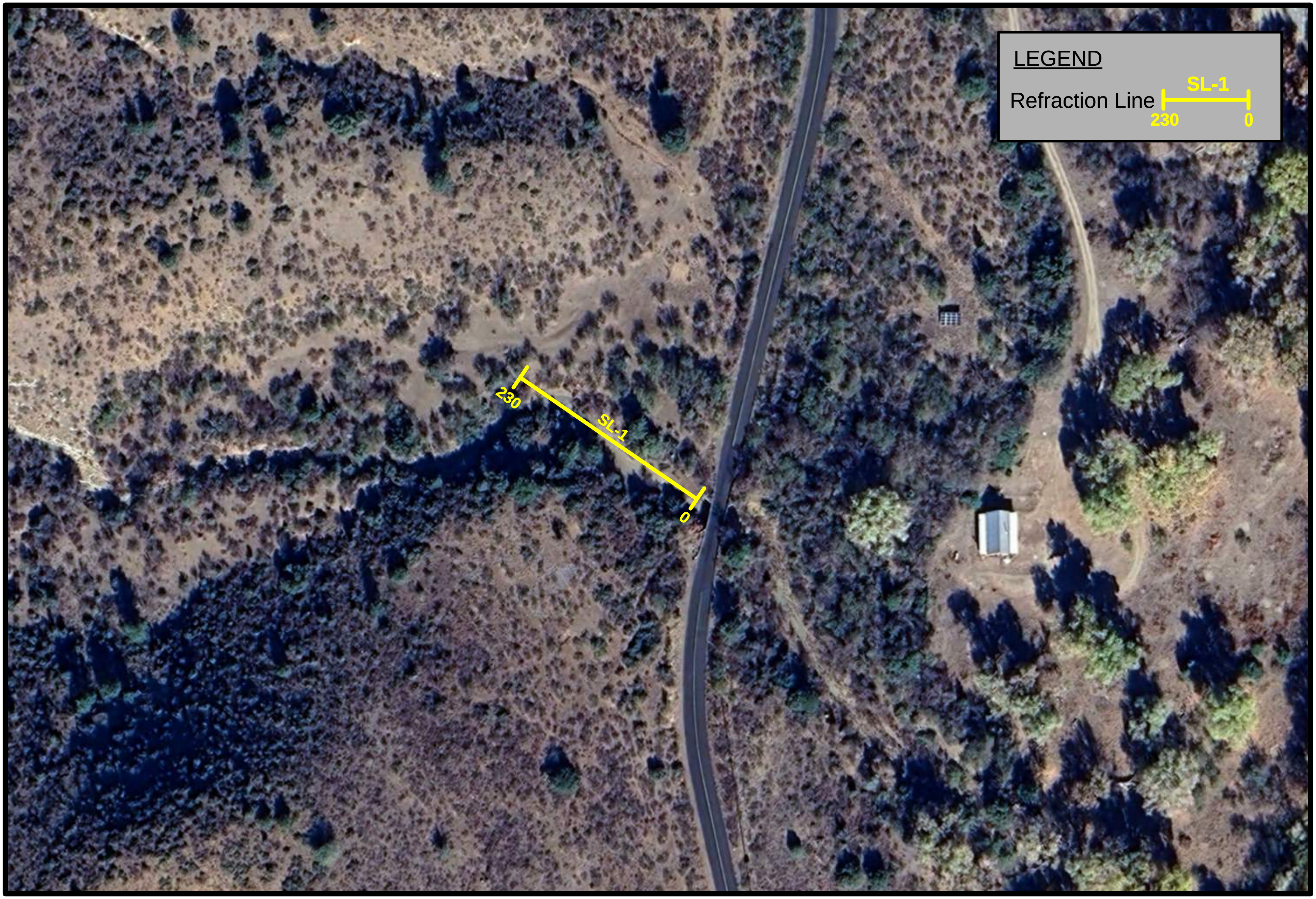
SITE LOCATION MAP

N

Yavapai County Wash Bridge
Kirkland Junction, Arizona

Advantage GEOPHYSICS

Project No.: 2024021	Date: 06/2024	Figure 1



SEISMIC LINE LOCATION MAP

N

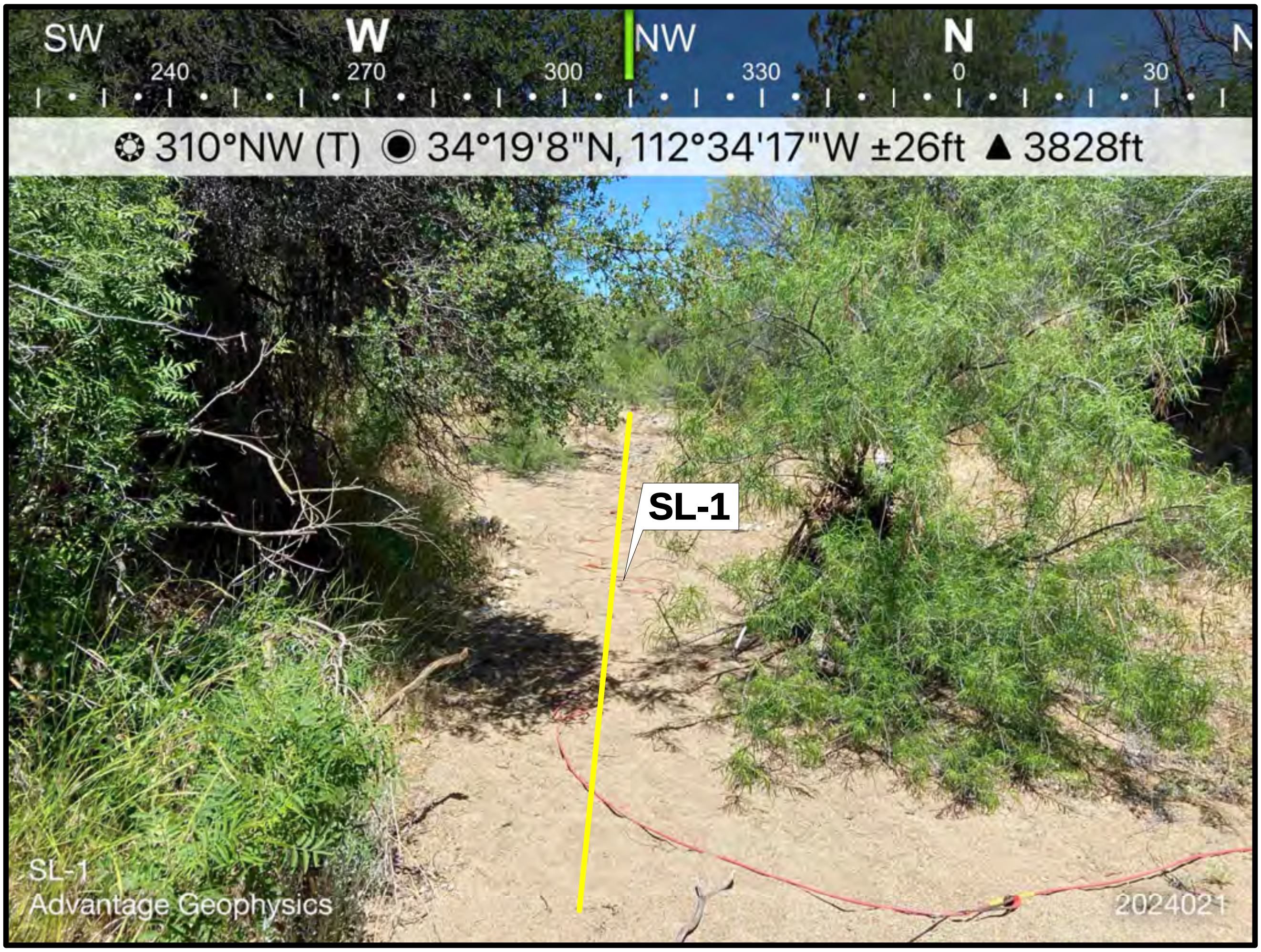
Yavapai County Wash Bridge
Kirkland Junction, Arizona

Project No.: 2024021 | Date: 06/2024

Advantage GEOPHYSICS

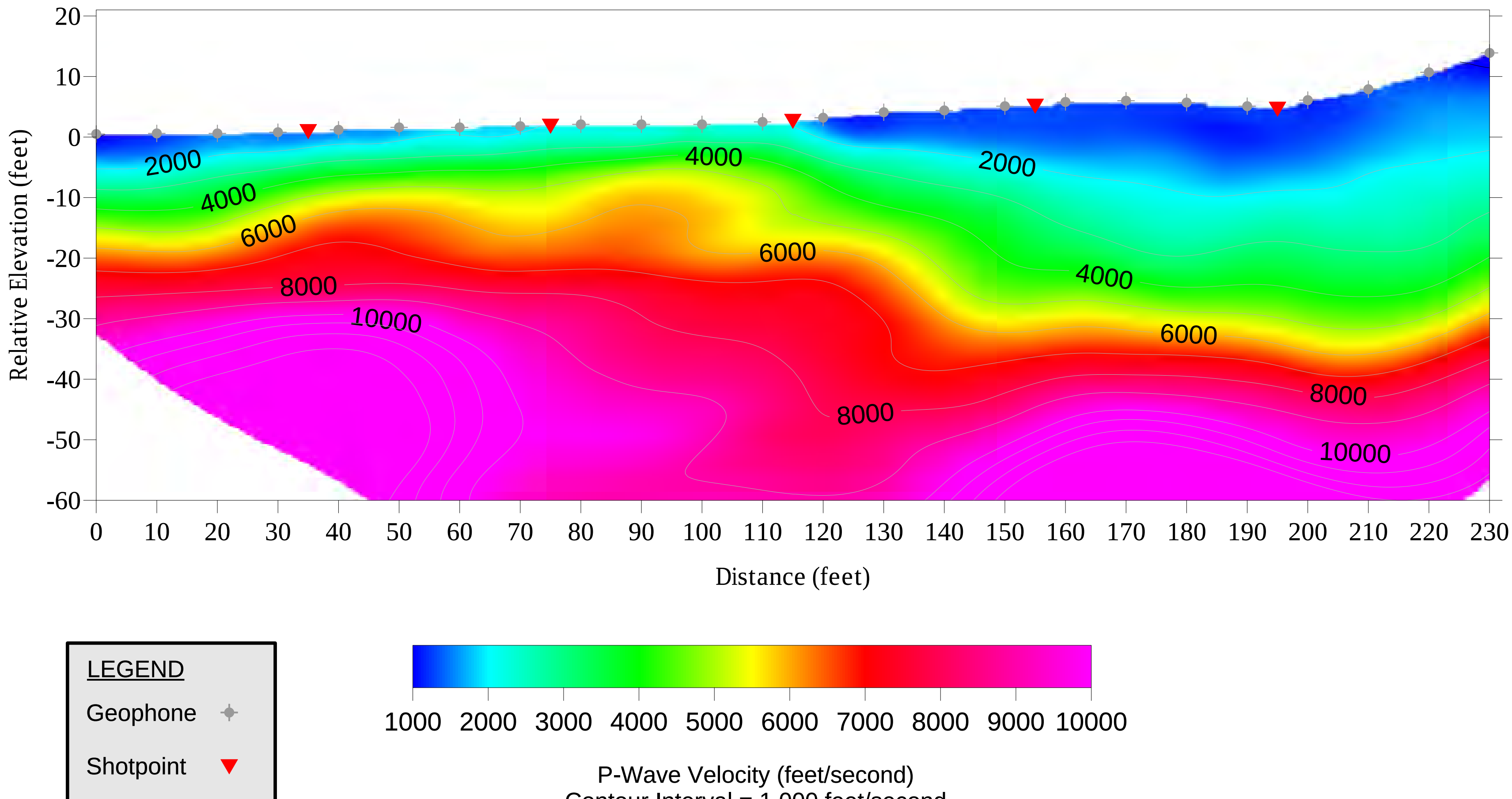
Figure 2

0 140 280
approximate scale in feet



SITE PHOTOGRAPH RL-1	Yavapai County Wash Bridge Kirkland Junction, Arizona	Advantage GEOPHYSICS
	Project No.: 2024021 / Date: 06/2024	Figure 3

SL-1



P-Wave Velocity (feet/second)
Contour Interval = 1,000 feet/second

SEISMIC PROFILE SL-1	N	Yavapai County Wash Bridge Kirkland Junction, Arizona		Advantage GEOPHYSICS
		Project No.: 2024020	Date: 06/2024	Figure 4

APPENDIX B

Laboratory Test Results

TABLE B-1: SUMMARY OF LABORATORY TEST RESULTS

Boring Number	Depth (feet)		USCS/Group Symbol (ASTM D2487)	Percent Fines (minus #200) (ASTM C136)	Liquid Limit (ASTM D4318)	Plasticity Index (ASTM D4318)	Moisture Content (%) (ASTM D2216/T265)	In-Place Dry Density (pcf) (ASTM D2937)	Optimum Moisture Content (%)[1] (ASTM D698A)	Maximum Dry Density (pcf)[1] (ASTM D698A)	Direct Shear (ASTM D3080)	pH (AZ 236)	Resistivity (ohm-cm) (AZ 236)	Sulfates (ppm) (AZ 733)	Chlorides (ppm) (AZ 736)
	Begin	End													
B-1	1.0	5.0	SM	21	NV	NP	2.6		7.3	135.0		8.5	4,190	6	19
B-1	5.0	6.0					6.7	82.4			X				
B-1	2.0	3.0					4.1	111.0							
B-1	14.0	15.5	SM	22	NV	NP	4.4								
Average				**---**	**---**	**---**	**4.5**	**96.7**	**7.3**	**135.0**	**---**	**8.5**	**4,190**	**6**	**19**
Standard Deviation				**---**	**---**	**---**	**1.5**	**14.3**	**---**	**---**	**---**	**---**	**---**	**---**	**---**
Maximum				**22**	**---**	**---**	**6.7**	**111.0**	**7.3**	**135.0**	**---**	**8.5**	**4,190**	**6**	**19**
Minimum				**21**	**NV**	**NP**	**2.6**	**82.4**	**7.3**	**135.0**	**---**	**8.5**	**4,190**	**6**	**19**
Count				**2**	**2**	**2**	**4**	**2**	**1**	**1**	**1**	**1**	**1**	**1**	**1**

Notes: pcf = pounds per cubic foot; ohm-cm = ohm-centimeters; ppm = parts per million
1) Values include rock correction. See test worksheet for details.

ACS Services LLC

Job #	2401464	Material Type:	Soils
Lab #	24-3517	Extraction Date:	6/7/2024
Client:	Ethos Enigeering	Extracted By:	Client
Project Name:	Wash Bridge # 8229	Laboratory Test Date	6/13/2024
Project Address:	South Wagoner	Laboratory Tested By:	Keagen Mayfield
Project City:	Prescott	Reviewed By:	Fernando Montero
Material Source:	-		

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937)

ID #	Sample Location	Wet Wt. (g)	Moisture Dry Wt. (g)	**Moist Content**	# Of Rings	Wet Wt'+Rings (g)	Wt. of Rings (g)	**Dry Density (pcf)**
24-3517-2	B-1 @ 2.0 - 3.0	557.9	536	4.1%	4	728.1	170.2	110.952

PROJECT: ACS Project #2401464
LOCATION: Mesa, AZ
MATERIAL: Native Soil

JOB NO: 17-2023-4239
WORK ORDER NO: 76
DATE ASSIGNED: 6/10/24

DENSITY OF SOIL IN PLACE BY THE DRIVE-CYLINDER METHOD (ASTM D2937)

		MOISTURE						
LAB #	BORING	WET WT. (g)	DRY WT. (g)	MOISTURE CONTENT	NUMBER OF RINGS	WET WEIGHT & RINGS (g)	WEIGHT OF RINGS (g)	DRY DENSITY (pcf)
24-3088	ACS Lab No. 24-3088 B-1 @ 5-6'	369.2	346.0	6.7%	6	898.9	261.9	82.4

WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

REVIEWED BY ____________________

ACS SERVICES LLC

Laboratory Soil Test Results

ACS PROJECT #	2401464	Material Type:	Soils
ACS Lab #	24-3517-1	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	6/7/2024
Project Name:	Wash Bridge #8229	Sampled By:	Client
Project Address:	South Wagoner Rd	Test Date:	6/14/2024
Project City	Prescott	Tested By:	Mahalia Davis
Sample Location:	B-1 @ 1.0 - 5.0	Reviewed By:	Fernando Montero

Sieve Analysis (ASTM C-136 / AASHTO T 27 / ARIZ 201)

Sieve Size	% Retained	% Passed	Specs
6"	0	100	
3"	0	100	
2 1/2"	1	99	
2"	0	99	
1 1/2"	1	98	
1"	3	94	
3/4"	3	91	
1/2"	6	85	
3/8"	4	81	
1/4"	6	75	
#4	4	71	
#8	8	63	
#10	3	60	
#16	8	52	
#30	11	41	
#40	5	36	
#50	4	32	
#100	7	25	
#200	5	21	

Liquid Limit (AASHTO T-89)	23
Plastic Limit (AASHTO T-90)	16
Plasticity Index (AASHTO T-90)	7
Moisture Content (AASHTO T-265)	2.6

USCS Soil Classification	SC-SM

Group Name (ASTM D2487)
Silty, clayey SAND with gravel

Fernando Montero
Laboratory Manager

Fernando Montero
Signature

ACS SERVICES LLC

Laboratory Soil Test Results

ACS PROJECT #	2401464	Material Type:	Soils
ACS Lab #	24-3517-4	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	6/7/2024
Project Name:	Wash Bridge #8229	Sampled By:	Client
Project Address:	South Wagoner Rd	Test Date:	6/11/2024
Project City	Prescott	Tested By:	Mahalia Davis
Sample Location:	B-1 @ 14.0 - 15.5	Reviewed By:	Fernando Montero

Sieve Analysis (ASTM C-136 / AASHTO T 27 / ARIZ 201)

Sieve Size	% Retained	% Passed	Specs
6"	0	100	
3"	0	100	
2 1/2"	0	100	
2"	0	100	
1 1/2"	0	100	
1"	0	100	
3/4"	0	100	
1/2"	1	99	
3/8"	3	96	
1/4"	6	90	
#4	4	86	
#8	13	73	
#10	3	70	
#16	9	60	
#30	11	49	
#40	4	45	
#50	6	39	
#100	9	29	
#200	7	22	

Liquid Limit (AASHTO T-89)	
Plastic Limit (AASHTO T-90)	
Plasticity Index (AASHTO T-90)	NP
Moisture Content (AASHTO T-265)	4.4

USCS Soil Classification	SM

Group Name (ASTM D2487)
Silty SAND

Testing sizes reduced from standard minimums due to lack of material

Fernando Montero

Laboratory Manager

Fernando Montero

Signature

ACS Services LLC	Maximum Dry Density & Optimum Moisture
	☐ AASHTO T 99 | ☐ AASHTO T 180 | ☑ ASTM D698 | ☐ ASTM D1557

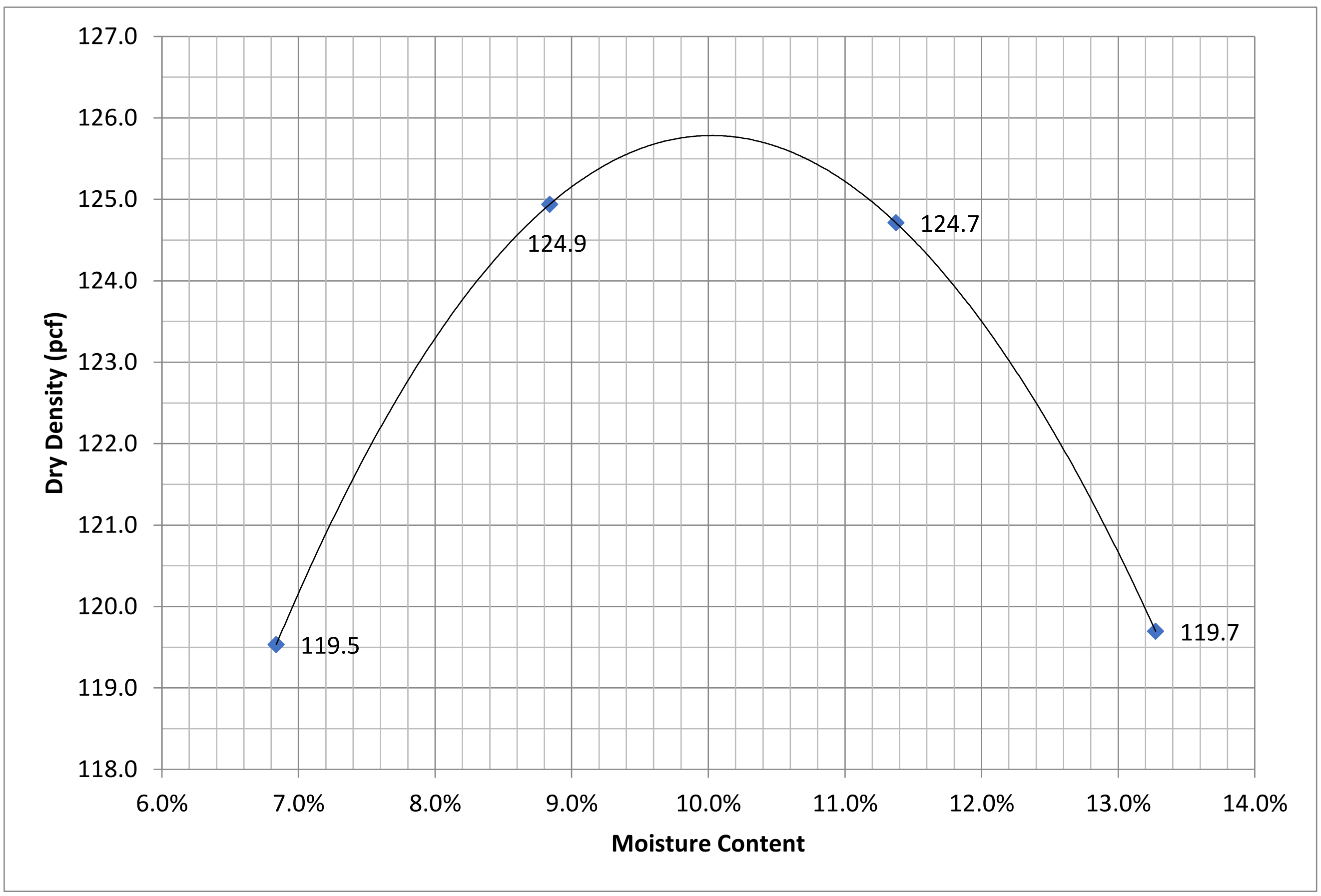
ACS Project #	2401464	Material Type:	Soils
ACS Lab #	24-3517-1	Material Supplier:	-
Client Name:	Ethos Engineering, LLC	Sample Date:	6/7/2024
Project Name:	Wash Bridge #8229	Sampled By:	Client
Project Address:	South Wagoner Rd	Date Tested:	6/13/2024
Project City:	Prescott	Tested By:	Keagen Mayfield
		Reviewed By:	Fernando Montero

Sample Location: B-1 @ 1.0 - 5.0

Method: ☑ A ☐ B ☐ C ☐ D

Dry Density	119.5	124.9	124.7	119.7
Moisture Content	6.8%	8.8%	11.4%	13.3%

Uncorrected Dry Density	125.8	Uncorrected Moisture Content	10.0
% Rock	30	% Passing	70
Rock Corrected Dry Density	**135.0**	Rock Corrected Moisture Content	**7.3**
Specific Gravity of Oversize Aggregate	**2.600**		



Fernando Montero

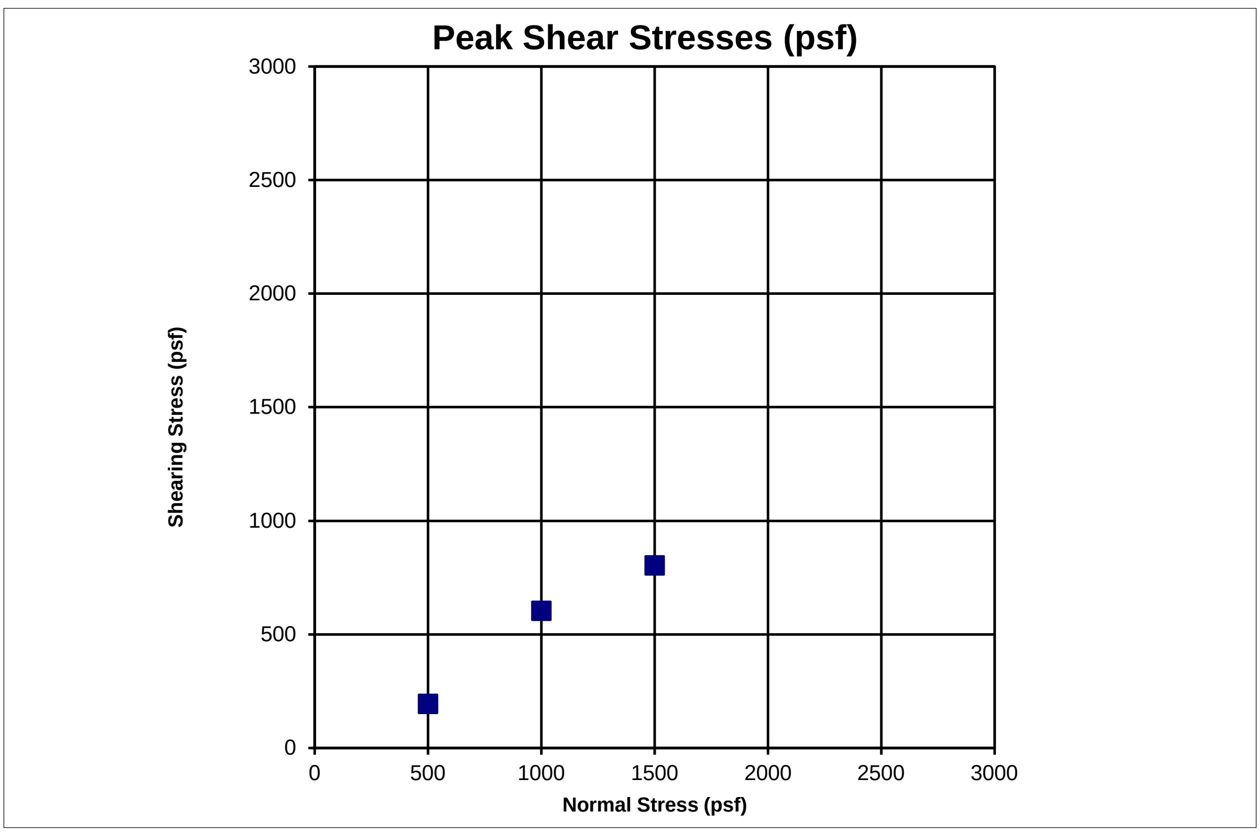
Project Manager

PROJECT:	ACS Project #2401464
LOCATION:	Mesa, AZ
MATERIAL:	Native Soil
SAMPLE SOURCE:	ACS Lab No. 24-3517-3 B-1 @ 5-6'
SAMPLE PREPARATION:	Saturated - .5, 1, and 1.5 ksf

JOB NO:	17-2023-4239
WORK ORDER NO:	76
LAB NO:	24-3088
DATE ASSIGNED:	6/10/2024

DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D3080)

Initial thickness of specimen (in.):	1.00	1.00	1.00
Initial diameter of specimen (in.):	2.42	2.42	2.42
Final thickness before shear (in.):	0.978	0.955	0.966
Shearing device used:	Humboldt Automated Shear Test System by Trautwein Soil Testing Equipment		
Rate of deformation (in/min):	0.02	0.02	0.02
Direct shear point:	1	2	3
Dry mass of specimen (g):	85.5	90.1	89.7
Initial Moisture Content:	13.6%	12.5%	13.9%
Initial Wet Density (pcf):	80.4	84.0	84.6
Initial Dry Density (pcf):	70.8	74.6	74.3
Final Moisture Content:	44.4%	36.4%	42.1%
Final Wet Density (pcf):	104.6	106.6	109.3
Final Dry Density (pcf):	72.4	78.2	76.9
Normal Stress (psf):	500	1000	1500
Maximum Shearing Stress (psf):	194	604	804
Vertical Deformation @ Max Shear (in):	0.318	0.215	0.268
Horizontal Deformation @ Max Shear (in):	0.377	0.486	0.414



WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

REVIEWED BY ______________________

PROJECT: ACS Project #2401464
LOCATION: Mesa, AZ
MATERIAL: Native Soil
SAMPLE SOURCE: ACS Lab No. 24-3517-3
SAMPLE PREPARATION: Saturated - .5, 1, and 1.5 ksf

JOB NO: 17-2023-4239
WORK ORDER NO: 76
LAB NO: 24-3088
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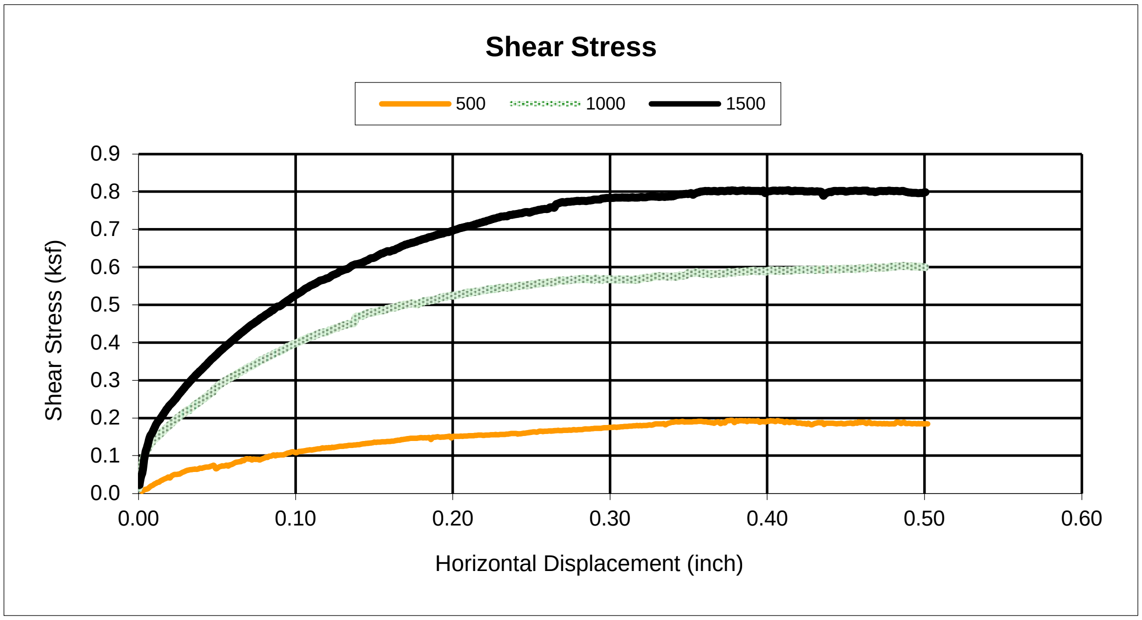
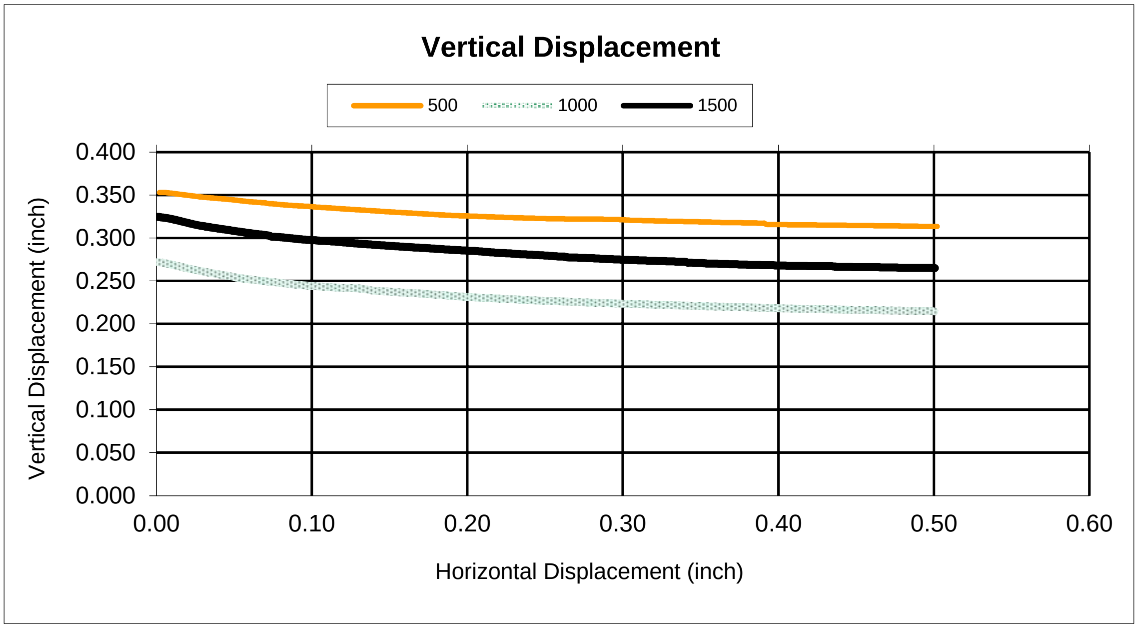
NORMAL LOADS (psf): 500 1000 1500

DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D3080)





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ACS Services LLC	Soil pH and Resistivity Determination AASHTO T-289 AASHTO T-288 / ARIZ 236

Project #	2401464	Material Type:	Soils
Lab #	24-3517-1	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	6/7/2024
Project Name:	Wash Bridge #8229	Sampled By:	Client
Project Address:	South Wagoner Rd	Test Date:	Thursday, June 13, 2024
Project City:	Prescott	Tested By:	Colin Eggebrecht
Sample Source:	B-1 @ 1.0 - 5.0	Resistivity Box:	1
		Reviewed By:	Fernando Montero

pH Reading = 8.5

P = (SBF) x R x M

Where:

SBF = Soil Box Factor, cm
R = Dial Reading, OHMS
M = Multiplier

Water Added	SBF (cm)	Dial Reading (OHMS)	Multiplier	**P** (OHM-cm)
200	7.22	5.8	100	4190
50	7.22	6.8	100	4910
50	7.22	6.7	100	4840

Keagen Mayfield
Lab Supervisor

Fernando Montero
Laboratory Manager

MOTZZ LABORATORY, INC.

Report: 951227
Reported: 6/15/2024
Received: 6/10/2024
PO: 2401464

Laboratory Analysis Report

ACS Services LLC
Fernando Montero
2235 W Broadway Road
Mesa, AZ 85202

Project: 2401464

Lab Number	Sample ID
951227-1	24-3517-1

Test Parameter

Test	*Method*	*Result*	*Units*
Sulfate	ARIZ 733b	6	ppm
Chloride	ARIZ 736b	19	ppm

APPENDIX C

Spread Footing Factored Bearing Resistance Chart

Figure C-1: Spread Footing Bearing Resistance
RCBC at Wash Bridge
Footing Length = 32 ft, Depth of Embedment = 1 ft,
Bottom of Footing Elevation = 3,811 feet (+/- 2 feet)