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GEOTECHNICAL EXPLORATION REPORT
FRISCO AVENUE BRIDGE
ADOT CONTRACT NO. 2018-006
ADOT PROJECT NO. T0285 01D
CLIFTON, ARIZONA

Prepared for:

Ms. Tricia Brown, PE
Senior Project Manager
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2600 North Central Avenue, Suite 550
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Registered Professional Engineer (Civil)
CERTIFICATE NO.
25151
KEITH H.
DAHLEN
Date Signed 10/6/22
ARIZONA, U.S.A.

Prepared by:

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Ethos Project No. 2021039
October 6, 2022

October 6, 2022
Ethos Project No.: 2021039

Tricia Brown, PE
Senior Project Manager
Horrocks Engineers
2600 North Central Avenue, Suite 550
Phoenix, Arizona 85004

SUBJECT: Geotechnical Exploration Report
Frisco Avenue Bridge
ADOT Contract No. 2018-006
ADOT Project No. T0285 01D
Clifton, Arizona

Dear Tricia:

Ethos Engineering, LLC is pleased to present the findings of the geotechnical exploration for the proposed new bridge crossing of Chase Creek along Frisco Avenue located in Clifton, Arizona. Our services were conducted in general accordance with the scope of services presented in our proposal, dated April 23, 2021. This report provides the results of our investigation for foundation support for the proposed new single-span bridge. Also included are recommendations for subgrade preparation at the bridge and approach roadway embankment, slopes and excavation conditions for the project.

We appreciate the opportunity to be of service 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.
25151
KEITH H.
DAHLEN
Date Signed 10/6/22
ARIZONA, U.S.A.

Keith Dahlen, P.E.
Principal/Senior Geotechnical Engineer

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Jesse Huston, P.E.
Senior Geotechnical Engineer

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1.0 PROJECT DESCRIPTION

The project consists of replacing the existing two-lane single-span Frisco Avenue Bridge in the town of Clifton in Greenlee County, Arizona. This is the only access point for those living north of Chase Creek along Frisco Avenue in Clifton and access needs to be maintained during construction. Accordingly, the new bridge will be offset to the west of the existing bridge which will remain during construction of the new bridge. The project is sponsored by the Town of Clifton and federal funding from the Off-System bridge program with both federal and local shares included.

Horrocks Engineers (Horrocks) have been retained by ADOT to prepare the construction documents for the replacement of the existing bridge with a two-lane bridge. Drawings provided in their Bridge Selection Report indicate a single-span bridge with an overall length of approximately 46 feet, and width of approximately 24 feet. This bridge will be constructed just west of the existing Frisco Avenue Bridge over Chase Creek on a shifted alignment. Current (Stage III) plans by Horrocks include supporting each abutment with two drilled shafts. Wingwalls at the abutments will be integral with the abutment caps. Horrocks estimates strength and service loads for each shaft at 337 kips and 231 kips, respectively.

The structure is to be designed by load and resistance factor design (LRFD) methodology in accordance with the AASHTO LRFD Bridge Design Specifications.

2.0 FIELD EXPLORATION

The field investigation consisted of one boring drilled by Geomechanics Southwest, Inc., and two seismic refraction lines performed by Atlas Technical Consultants, LLC (Atlas). A second test boring planned for the north abutment was not drilled as the drill rig exceeded the allowable 10-ton load on the existing Frisco Avenue Bridge. Other available routes of access to the north abutment were not possible due to private property limitations. The investigation was modified to include two refraction seismic lines, one at each of the planned abutments.

The drilling was performed on June 21, 2022 and the seismic survey was completed on August 11, 2022. The field exploration was supervised by Ethos' field engineer, Magdelano Meza, EIT. Details of the field exploration are provided in the following sections. The test locations are shown on Figure 2.

2.1 TEST DRILLING

The boring was drilled with a truck-mounted CME 75 drill-rig advancing 8-inch outside-diameter hollow stem auger and HQ wireline core barrel. During the field exploration, the soils were visually classified, logged, and sampled by the Ethos' field engineer.

Disturbed soil samples were obtained using a standard penetration test (SPT) split spoon sampler with a 1.375-inch ID and 2-inch OD. Relatively undisturbed soil samples were obtained using a ring sampler with a 2.42-inch inside diameter (ID) and 3-inch OD. Bulk samples of drill cuttings were also collected at selected depths from the boring. The SPT and ring samplers were driven 18 and 12 inches or to refusal (i.e. 50 blows for less than a 6-inch interval), respectively, using an automatic hydraulic actuated 140-pound hammer, free falling 30 inches. Unless noted otherwise on the boring log, the sample driving resistance was recorded as the number of blows per six inches of penetration. The penetration results are presented on the borings logs adjacent to each sample.

The recovered soil samples were removed from the sampler, sealed to reduce moisture loss, and submitted to the ACS Services, LLC (ACS) laboratories. HQ core samples were collected and boxed for further examination. The boring was backfilled in accordance with permit requirements. The exploratory boring log is included in Appendix A.

2.2 REFRACTION SEISMIC

Surface data collection consisted of seismic refraction surveys to complement the test boring and provide a more complete picture of the underlying soil/bedrock profile. Two seismic lines (designated as SL-1 and SL-2) were completed by Atlas Technical Consultants, LLC., as a subconsultant to Ethos. The surface seismic lines consisted of 125-foot-long seismic refraction compression wave (p-wave) surveys performed along lines running nearly parallel with the two abutments. Seismic refraction microtremor (Remi) one-dimensional profiles were also performed as Lines RL-1 and RL-2. The seismic lines were completed using a Geometrics StrataView seismograph and 24 geophone array system.

Two-dimensional p-wave profiles and velocities were interpreted based on seismic data collected using a sledgehammer energy source. Results of the refraction seismic and Remi surveys are presented in a report provided in Appendix D, which show the seismic line locations and includes descriptions of the seismic refraction equipment and procedures used.

3.0 LABORATORY TESTING

Selected laboratory tests were assigned by Ethos and performed by ACS and Motzz. Lab testing was performed on representative samples recovered from the borings to support the field classification and to provide information regarding engineering characteristics and properties of the subsurface soils. The laboratory testing program is listed in Table 3.1. A summary of the laboratory test results along with individual test worksheets are included in Appendix B.

Table 3.1 – Laboratory Testing Program

Laboratory Test	Sample Type	Number of Tests	Purpose of Test
Sieve Analysis (ASTM C136)	Bulk/SPT	3	Soil Classification
Atterberg Limits (ASTM D4318)	Bulk/SPT	3	Soil Classification
Sulfates & Chloride (AZ 733/736)	SPT/Bulk	2	Concrete/Soil Degradation Potential
pH and Resistivity (AZ 236)	SPT/Bulk	2	Corrosion Potential

4.0 GENERAL SITE CONDITIONS

4.1 SURFACE CONDITIONS

The new bridge will cross Chase Creek at a point several feet west of where it discharges into the San Francisco River. Chase Creek is channelized with rock grouted walls which stand near vertical (some batter) to an approximate height of 15 feet. The existing concrete-slab with steel girder single span bridge (which is to be replaced) has a posted 10-ton load rating. The ground surface is relatively flat both north and south of the bridge, likely sloping slightly towards the east to the river. Vegetation consists of scattered trees which parallel both banks of the creek and scattered wild grass. The planned location of the new bridge and shifted roadway alignment is on property owned by the Freeport-McMoRan Mine. This area is generally light industrial with much of the property owned by the mine. Photo 1 shows the bottom of the existing bridge and one of the walls which line Chase Creek.

Photo 1 – Looking East at North End of Existing Frisco Avenue Bridge

4.2 REGIONAL AND LOCAL GEOLOGY

The project site is located near the southern boundary of the Transition Zone physiographic province, a rugged mountainous region that separates the Colorado Plateau from the Basin and Range province (Briggs 2016) in Eastern Arizona. Bedrock in the area is present at or very near the surface with alluvial/residual soil covering the rock at variable depths.

Published geologic mapping indicates that surficial geologic units at the project site consist of middle Miocene to Oligocene volcanic rocks including lava, tuff, fine-grained intrusive rock, and diverse pyroclastic rocks. These compositionally variable volcanic rocks include basalt, andesite, dacite, and rhyolite (Richard et al 2000). Test drilling indicates a mantle of alluvial derived soil with an approximate thickness of 30 or more feet which overlies the volcanic rock. These soils are likely overbank materials associated with the adjacent San Francisco River.

4.3 SITE SUBSURFACE CONDITIONS

The site soils encountered in Boring B-1 consisted of moderately firm, low plastic clayey gravel with sand (GC-GM) extending to a depth of 8 feet overlying soft to firm non-plastic to low plasticity sandy silt to silty sand (SM-ML), which was encountered to a depth of 19 feet. A layer of non-plastic silty gravel with sand (GP-GM) was encountered below the silty sand extending to the full depth of investigation (30 feet). It appears this layer may get coarser with depth and contain cobbles and boulders. Limited core recovery within this material made identification of the materials difficult.

The refraction seismic surveys included test lines on each side of Chase Creek near each planned abutment location. The results from the refraction seismic test lines were similar and agreed with the conditions encountered in Boring B-1, confirming similar denser materials present below a depth of about 20 feet. The boring log for B-1 is included in Appendix A of this report. A photo of the HQ core is also included in Appendix A. The photo confirms that coarse-grained alluvial soils were present to a depth of 30 feet. The results of the refraction seismic testing are included in Appendix D.

As the drill method used water, it is not known whether water was encountered within the boring. Groundwater at this location would be expected to be at a similar elevation to what is present within the adjacent San Francisco River. As the ground surface is roughly 13 to 15 feet above the surface of the adjacent riverbed, groundwater should be anticipated to impact construction of drilled shafts for this project.

4.4 SITE SEISMICITY

The project site is located in south-east Arizona which is an area of generally low seismic activity. From Table 3.10.3.1-1 (AASHTO 2012) and the results of refraction seismic surveys, the Site Classification is C (very dense soil to soft rock). In accordance with AASHTO (2017) the project site has the Horizontal Spectral Response Acceleration Coefficients with a 7 percent probability of exceedance in 75 years. The probabilistic horizontal spectral acceleration values for the designated return period and corresponding horizontal peak ground acceleration (PGA) were obtained from the United States Geological Survey (USGS) seismic hazards program website (USGS 2002). The values obtained from the website are based on 2009 AASHTO Guide Specifications for Load Resistance Factor Design (LRFD) Seismic Bridge Design and use 2002 USGS seismic hazard data. For structural design, the seismic parameters in Table 4.1 should be used.

Table 4.1: Summary of Seismic Parameters

Parameter	Value	AASHTO Reference
Latitude 33.05727° N, Longitude 109.30013° W		
Site Class Definition	C	Table 3.10.3.1-1
Site Coefficient, F_{PGA}	1.2	Table 3.10.3.2-1
Site Coefficient, F_a	1.2	Table 3.10.3.2-2
Site Coefficient, F_v	1.7	Table 3.10.3.2-3
PGA	0.081g	
Spectral Acceleration, S_{DS}	0.225g	Equation 3.10.4.2-3
Spectral Acceleration, S_{D1}	0.091g	Equation 3.10.4.2-6

5.0 ENGINEERING ANALYSES AND RECOMMENDATIONS

5.1 GENERAL

The following sections of this report present our recommendations regarding foundation design for the single-span bridge abutments, embankments (as needed), site preparations and grading, moisture protection, excavations and other construction considerations. According to the design team, short wingwalls will be cantilevered off from the abutments. These recommendations are based on our understanding of the project, our review of the current bridge plans, the results of our field exploration and laboratory testing for the site, and engineering analyses.

5.2 FOUNDATION RECOMMENDATIONS

The foundation recommendations provided in this section are based on the AASHTO LRFD Bridge Design Specifications (AASHTO 2012). The information presented in this section is based on the one exploratory bridge boring (B-1) and refraction seismic lines SL-1, SL-2, RL-1 and RL-2 (completed at each planned abutment location).

Our understanding, based on discussions with the design team, is that the abutments are planned to be supported on drilled shafts, which are feasible given the anticipated moderately light loads and dense materials present at depth.

In general, drilled shafts, which derive their support from both side shear and tip resistance within the denser soils present at depth, will provide adequate support of the abutments with limited post-construction settlement. Drilled shafts which terminate within the denser soils present below an approximate depth of 19 feet below the existing site grades should experience minimal post-construction settlement. Included herein are drilled shaft recommendations for the bridge abutments.

5.2.1 Drilled Shaft Foundations

5.2.1.1 Axial Resistance

The axial compression resistance of drilled shaft foundations for the project was determined using the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012) using both tip and side resistance. The drilled shaft foundations for the project were designed using the Beta method as outlined for cohesionless soils and IGM materials based on the subsurface profile encountered in Boring B-1 at the bridge location. For the beta method analysis, refusal blow counts were limited to 50 in cohesionless soils and 100 for IGM materials (AASHTO 2012). The axial resistance design charts presented in Appendix C are applicable for redundant conditions. For non-redundant conditions, the resistance should be reduced by 20 percent. The provided design charts in Appendix C can be used for non-redundant conditions by increasing the applied loads by a factor that is the inverse of the reduction factor, and then entering the charts with the increased loads. A resistance factor of 0.8 (i.e., 80 percent) for non-redundant conditions corresponds to a load factor of 1.25 (i.e. 1/0.8=1.25) or an increase in the load by 25 percent.

The following sections provide design recommendations for strength and service limit states for drilled shaft foundations at the Frisco Avenue bridge over Chase Creek. A minimum drilled-shaft diameter of 4 feet is recommended to facilitate construction of the shafts in coarser grained soils. We understand the top of the drilled shafts will be located approximately 5 feet below the existing site grades. A minimum shaft penetration of 15 feet below the top of shaft (i.e., 20 feet below existing site grades) is also recommended such that the shafts will bear on the dense soils present below a depth of about 19 feet.

5.2.1.1.1. Strength Limit State

Resistance factors used in the determination of the vertical resistance for drilled shafts are a function of the design methodology. The corresponding resistance factors for geotechnical resistance of drilled shafts are 0.55 and 0.5 for beta method side resistance and end bearing, respectively, as presented in Table 10.5.5.2.4-1 of AASHTO (2012). These resistance factors assume redundant foundations as defined in Section 10.5.5.2.4 of AASHTO (2012) and Section 10.5.5.2.4 of the ADOT *Bridge Practice Guidelines* (2011).

5.2.1.1.2. Service Limit State

The vertical resistance provided by the soil is a function of the relative movement between the drilled shaft and the surrounding soil. Article 10.8.2.2.2 of AASHTO (2012) provides relationships for the development of skin friction and end bearing as a function of settlement normalized to the drilled shaft diameter for various soil types. The vertical resistances for the drilled shafts at various levels of deflection were calculated using these relationships.

5.2.1.1.3. Group Effects - Axial

Design criteria for reductions in axial resistance resulting from group effects are presented in Sections 10.7.3.9 and 10.8.3.6 of the AASHTO (2012) manual. For cohesionless materials, the individual nominal resistance of each shaft in a group should be reduced by a factor, η, presented in Table 10.8.3.6.3-1 of AASHTO (2012) and reproduced in Table 5.1.

The design charts presented in Appendix C apply to single shafts and therefore do not include a group reduction factor. For axial capacity reductions due to group effects, the factored loads should be increased by the inverse of the appropriate reduction factor when using the design charts.

For a single row of drilled shafts, the minimum center-to-center spacing should be two diameters, and the appropriate reduction factors determined by linear interpolation for center-to-center spacing between two and three diameters. The reduction factors should be applied equally to all shafts within the group regardless of location within the group.

Table 5.1: Group Reduction Factors for Bearing Resistance in Cohesionless Materials

Shaft Group Configuration	Shaft Center-to-Center Spacing	Special Conditions	Reduction Factor for Group Effects, η
Single Row	*2D*	---	0.90
	3*D* or more	---	1.0
Multiple Row	2.5*D*	---	0.67
	3*D*	---	0.80
	4*D* or more	---	1.0
Single and Multiple Rows	2*D* or more	Shaft group cap in intimate contact with ground consisting of medium dense or denser soil, and no scour below the shaft cap is anticipated	1.0
Single and Multiple Rows	2*D* or more	Pressure grouting is used along the shaft sides to restore lateral stress losses caused by shaft installation, and the shaft tip is pressure grouted	1.0

5.2.1.2 Downdrag

Our understanding is that approach embankments to the new bridge abutments will be negligible. No fill is anticipated to be placed adjacent to the drilled shafts and as such, the ground is not expected to experience appreciable settlement. Therefore, no downdrag loads need to be considered for drilled shafts at the abutments.

5.2.1.3 Lateral Resistance

Lateral soil-structure interaction analyses of single shafts are typically performed using the computer program LPILE. This procedure estimates the lateral load-displacement behavior using a finite difference technique based on elastic beam column theory and soil reaction-displacement curves. Based on Reese and others (1984), the behavior of the soil surrounding the laterally loaded shaft is described by lateral load-transfer functions referred to as p-y curves. The soil reaction (p) is related to the shaft deflection (y) for various depths below the ground surface. In general, these curves are nonlinear and depend upon several parameters including depth, shaft diameter, and soil strength. Deflection, bending moment and shear profiles at specified intervals along the length of the shaft are computed.

5.2.1.3.1. LPILE Input Parameters

Recommended soil input parameters for use in LPILE analyses are provided in Table 5.2. The soil input parameters were developed using the LPILE technical manual (Ensoft, 2015) and results of the geotechnical investigation.

Table 5.2: Soil Input Parameters for LPILE Analyses

Soil Layer	Elevation Range (feet)	Soil Type in LPILE	Effective Unit Weight (pcf)	Friction Angle (degrees)	Horizontal Subgrade Modulus, k (pci)
1	3,468-3,460	Sand	115	34	125
2	3,460 – 3,449	Sand	110	28	60
3	Below 3,449	Sand	135	38	225

NOTES:
When the ground in front of the drilled shaft is sloping (or vertical), the lateral shaft resistance shall be ignored to a depth when the lateral distance in front of the drilled shaft extends a minimum of three (3) diameters in front of the shaft.
pcf = pounds per cubic foot, pci = pounds per cubic inch, psf = pounds per square foot

Our understanding based on review of the Stage III Plans and discussions with Horrocks is the drilled shafts will be located approximately 8 feet behind the masonry rock walls which line Chase Creek. Given the proximity of new shafts to the existing walls, we recommend a 50% reduction in the effective unit weight and horizontal subgrade modulus for evaluation of horizontal loads and deflection in the direction of the wall when utilizing LPILE and the above parameters. A ground deflection of less than 0.5 inches in the direction of the wall is considered adequate to minimize impacts to the existing masonry rock wall.

5.2.1.3.2. Group Effects - Lateral

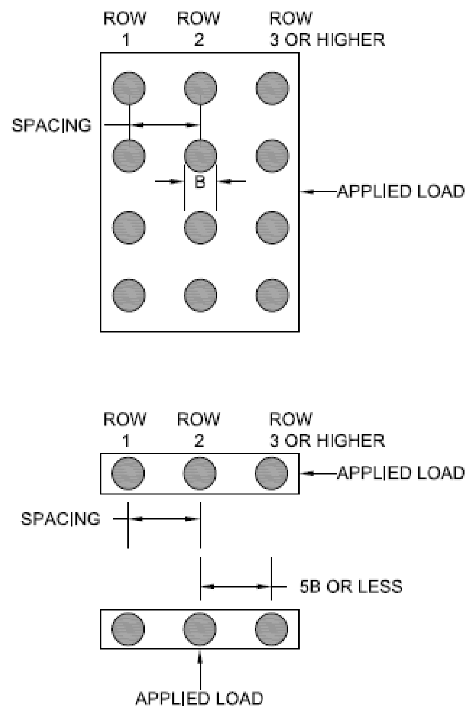
The design of laterally loaded drilled shafts must account for the influence from adjacent shafts in a group. Article 10.7.2.4 (AASHTO, 2012) defines a drilled-shaft group with respect to lateral loading as drilled shafts spaced less than five diameters center-to-center in the direction parallel and normal to the applied load. When the drilled shafts are in a group, that lateral resistance of the soil is reduced to account for the influence of adjacent drilled shafts by multiplying the values of p of the p-y curves by P-multiplier values (P_m). The values of P_m vary as a function of the center-to-center (CTC) spacing and position of the drilled shafts within the group. The loading direction and spacing are shown in Figure 5.1 which is based on Figure 10.7.2.4.1 from AASHTO (2012). Recommendations for P_m are shown in Table 5.3, based on AASHTO Table 10.7.2.4 1 (AASHTO, 2012) for CTC spacing of 3B and 5B. For CTC spacing determinations between different diameter shafts (i.e., at the center pier), the larger shaft diameter should be used when determining p-multiplier values for lateral loading.

Table 5.3: P-Multipliers for Multiple Row Shading

Center-to-Center (CTC) Spacing in the Direction of Loading	P-Multipliers, P_m		
	Row 1	Row 2	Row 3
3B	0.8	0.4	0.3
5B	1.0	0.85	0.7

NOTE:
B=drilled shaft diameter

Figure 5.1: Definition of Loading Direction and Spacing for Group Effects



5.2.1.4 Drilled Shaft Construction

Straight, drilled shaft excavations will likely be advanced with single-flight-auger or bucket-auger bits to the recommended depth. The subsurface conditions typically consist of coarser clayey to silty gravel with sand overlying sandy silt/silty sand overlying coarser grained alluvium consisting of sand and gravel with cobbles and likely small boulders. Drilled shaft excavations in these soils will likely encounter caving and/or sloughing of the more sandy and gravelly soil layers. Casing and/or slurry may be needed to advance the drilled shafts.

Cleaning of the drilled-shaft excavations should be performed just prior to placing concrete. It should be verified by inspection and measurement that the excavation is open to the design depth. The excavations should be cleaned so no more than 2 inches of slough or loose material are present in the bottom of the excavation. The drilled-shaft excavation should be cleaned of loose materials prior to concrete placement.

Groundwater is expected to impact the construction of drilled shafts. Integrity testing of each drilled shaft foundation should be performed by means of a cross-hole sonic logging (CSL) survey and a gamma-gamma logging (GGL) survey.

5.2.1.5 Foundation Subgrade Preparation

Preliminary Stage III plans prepared by Horrocks indicate that foundation elements for the bridge will be constructed on native site soils at an approximate depth of 5 feet below existing site grades. Trash, debris, vegetation (including roots) and other organics, any existing spread fill, any unstable (soft, loose, disturbed, water softened, sedimentation, collapsible, expansive, etc.) soils, and other deleterious materials should be removed from proposed structure foundation areas (including drilled shaft caps at abutments) prior to construction. All areas of excavation should be observed and approved by the geotechnical engineer after clearing and before any placement of foundations or backfilling operations begin at the site. Unless unstable soils are encountered at the bottom of pier cap elevation, scarification of the exposed surface at the base of the abutment caps should not be needed as the cap will be supported on drilled shafts.

5.2.1.6 Structure Backfill

All backfill placed for this project to build the site to final design grades (following foundation excavations) should consist of structure backfill meeting the requirements of Table 5.4. All structure backfill should be moisture conditioned to within 2 percent of the optimum moisture content and compacted to a minimum of 95 for general embankment and 100 percent (within 50 feet of abutment approach slabs) of maximum ASTM D698 Standard Proctor density. Consideration should be made at the time of construction in terms of compaction equipment to be used and the level of effort, lift thickness etc., for compaction immediately adjacent to walls.

Table 5.4: Gradation of Structure Backfill

Sieve Size (square openings)	Percent Passing by Weight
3 inch	100
3/4 inch	60 - 100
No. 8	35 - 80
No. 200	0 - 12

Additional requirements for Structure Backfill include the following:

- The plasticity index, determined in accordance with AASHTO T 90, should not exceed 5.
- The material should have a minimum angle of internal friction of 33 when tested in accordance with ASTM D3080.

5.3 SLOPES

5.3.1 Permanent Slopes

Fill slopes, if utilized, are anticipated to be minimal. Non-stabilized embankment fill slopes should be on the order of 3:1 (H:V) or flatter. Flatter slopes will promote re-vegetation and can accept landscaping. Slopes protected with slope paving or rock armored slopes should be no steeper than 2:1 (H:V). Permanent cut slopes, where required, should be no steeper than 2.5H:1V.

5.3.2 Temporary Slopes

Temporary excavations for construction of footings, drilled shaft caps, etc. can be made with conventional earthmoving equipment. Temporary slopes should be excavated in accordance with OSHA (1995). In accordance with Subpart P, Appendix A, the embankment and native soils to a depth of approximately 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 clean, 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.

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.

5.4 SURFACE DRAINAGE

Long-term performance of 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 to minimize moisture infiltration into the subgrade. Structural fill and backfill should be well compacted to reduce possible moisture infiltration through loose soil intervals.

5.5 EARTHWORK FACTORS AND WATER FOR COMPACTION

Testing was not performed to determine earthwork factors or ground compaction for this project. However, based on our visual assessment and in accordance with current recommendations provided in Section 5.2 of the ADOT Geotechnical Design Manual (2021), an excavation of factor of 10 percent shrink is recommended along with a ground compaction factor of 0.1 feet. Water for compaction is estimated at 80 gallons per cubic yard for both import and structure backfill.

5.6 PRELIMINARY CORROSION OR DEGRADATION POTENTIAL

5.6.1 Metal in Contact with Soil

The corrosion potential of near surface soils was characterized using laboratory pH and electrical resistivity testing, performed in accordance with Arizona Test Method 236. The laboratory pH value from one sample (B-1 at a depth of 6 to 10 feet) was 8.3. The resistivity value of the tested sample was 1,286 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) or similar guideline.

5.6.2 Concrete in Contact with Soil

One sample from the current investigation was used for soluble sulfates and chlorides (Arizona Test Method 733 and Arizona Test Method 736) to support the design of concrete structures. The results of these laboratory tests are included in Appendix B.

The total soluble sulfate value was 217 parts per million (ppm). The sulfate test measures the water-leachable or "available" sulfate content. This result was 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 318-14, 2014). The sample falls within Exposure Class S0 for water-soluble sulfate (SO42-) in soil by percent mass (SO4<0.1% or 1,000 ppm) and are categorized with a severity level of "not applicable" in terms of sulfate exposure. Based on Table 19.3.2.1, "Requirements for Concrete by Exposure Class," in Section 19.3.2 of ACI 318-14, there is no restriction on Portland cement type for concrete structures in contact with these materials.

The chloride test value was 16 ppm. Regarding chloride attack, Section 19.3.2 of ACI 318-14/318R-14 (2014) 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 of ACI 318-14/318R-14. The anticipated concrete exposure for this segment falls within Exposure Class C1. Table 19.3.2.1 of ACI 318-14/318R-14 should be referred to for requirements for concrete by exposure class. For Exposure Class C1, the minimum compressive strength of concrete specified for 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.

5.6.3 Recommendations

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. These results are general in nature and may not be representative of site conditions. A qualified corrosion engineer should be consulted if corrosion of underground utilities is a concern or if a detailed evaluation is necessary.

5.7 EARTHWORK

5.7.1 Site Preparation

All vegetation and debris should be removed from areas designated for pavement or embankment fill. Within the foundation footprint of each structure, remove all existing vegetation, existing structures, any uncontrolled fill, and any loose or otherwise unstable materials.

5.7.2 Fill Materials, Placement and Compaction

Construction of embankment fills should be in accordance with the project's special provisions. Fill material should be placed in loose lifts no thicker than 10 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 structure backfill should be moisture conditioned to within 2 percent of the optimum moisture content and compacted to a minimum of 95 for general embankment and 100 percent (within 50 feet of abutment approach slabs) of maximum ASTM D698 Standard Proctor density.

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 are provided. Our conclusions, opinions and recommendations are based on the completed test boring, refraction seismic surveys, 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.

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FIGURES

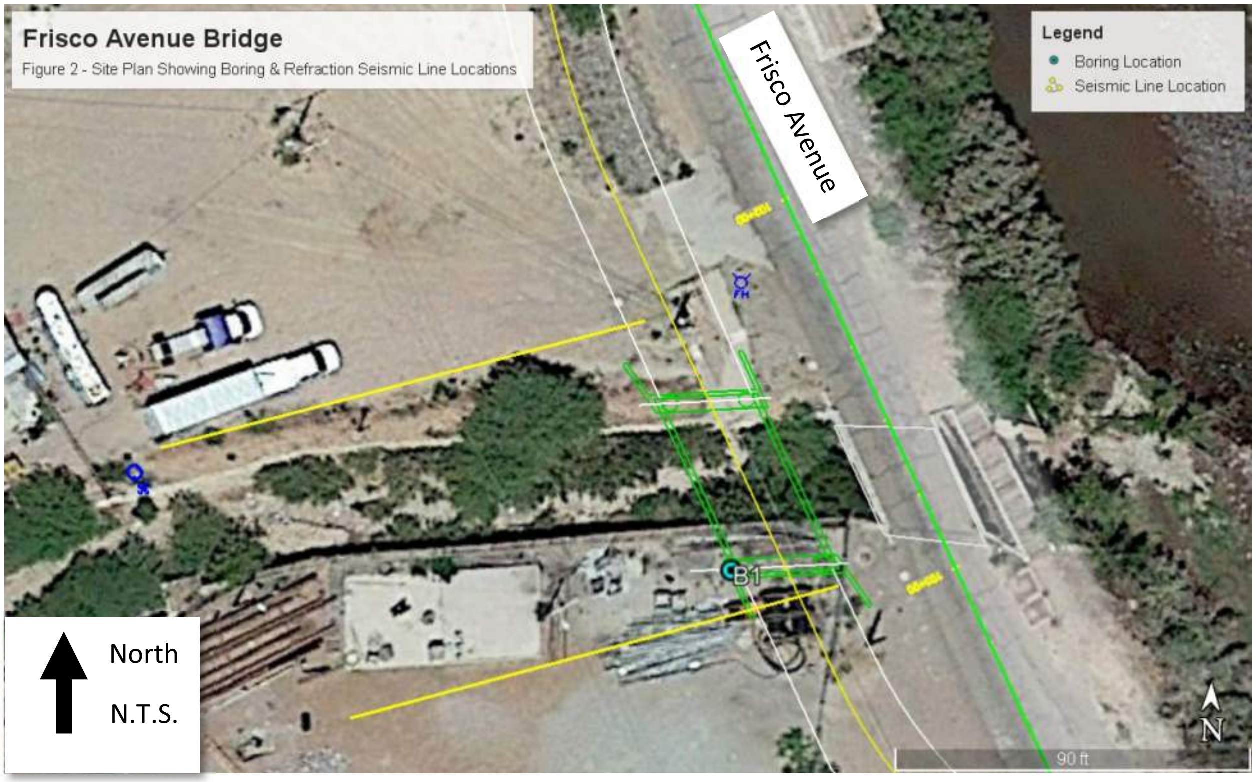
Source: 33.06064°, 109.31575°. Google Earth 5/4/2019 Imagery. Accessed on 3/23/2022.

Figure 1

Vicinity Map Showing Project Location

Frisco Avenue Bridge Replacement Project

Clifton, Arizona



Source: 33.06064°, 109.31575°. Google Earth 5/4/2019 Imagery. Accessed on 3/23/2022.

Figure 2

Site Map Showing Test Locations

Frisco Avenue Bridge Replacement Project

Clifton, Arizona

APPENDIX A

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:

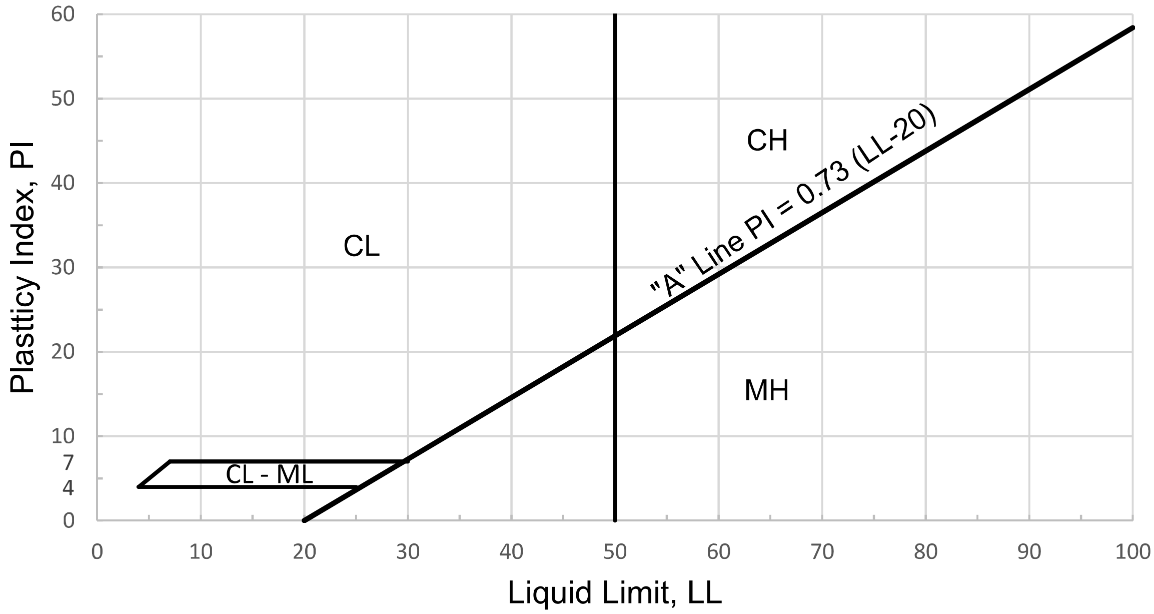
N	Relative Firmness
0-4	Very Soft
5-8	Soft
9-15	Moderately Firm
16-30	Firm
31-49	Very Firm
50+	Hard

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>

Plasticity Chart



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)

ROCK SAMPLING & BORING LOG INFORMATION

Borings are advanced in rock and rock-like materials using one of, or combinations of, various drilling methods, including wireline coring, pneumatic percussion, mud-rotary, or rock-bit. Typically, borings are advanced through unconsolidated overburden materials using hollow-stem auger, pneumatic percussion with casing, or mud-rotary with casing methods until competent rock is reached. The underlying rock is then usually cored using wireline methods with the upper casing sections left in-place to maintain the borehole until completion. Core may be recovered in N- or H-sizes, usually in a triple-tube core barrel with a split inner liner. N-series core has an approximate O.D. of 1.8 inches and H-series core has an approximate O.D. of 2.4 inches. The recovered core is logged and then placed into waxed cardboard boxes for future inspection, shipping, and storage.

The classifications and material descriptions of rocks presented on the boring logs are based on visual observation and classification in general accordance with widely used and accepted systems and formats. The classifications and names of rock types presented on the boring logs are given in general accordance with the field classification system adopted by the United States Bureau of Reclamation (USBR, as modified from R. B. Travis, 1955). The classifications are intended to provide general lithologic classification to the rock materials encountered and are not based on any detailed petrographic or mineralogical analyses. The descriptive rock information presented on the boring logs is in general accordance with the standards given in the Guidelines for Geotechnical Investigation and Geotechnical Report Presentation (ADOT, 1991), and supplemented as warranted by descriptive criteria presented in the Engineering Geology Field Manual (Second Edition, Volume I, USBR, 1998). Abbreviations of full descriptive terms may be shown on the boring logs and are parenthesized within the full descriptions given below. The percent core recovery, Rock Quality Designation (RQD), and percent fluid recovery are continuously logged during drilling. Selected samples from the recovered core may be submitted for laboratory testing, including unconfined compressive strength, triaxial compression, and direct shear, among others. The field logs are modified, where appropriate, based on further inspection of recovered core and/or laboratory testing of selected samples.

Degree of Rock Weathering

The degree of weathering of the rock mass is described based on the following criteria:

Designation	Field Identification
Fresh (F)	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered (SW)	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 inch into rock.
Moderately Weathered (MW)	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Highly Weathered (HW)	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration or rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed (Dec)	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to a soil with hand pressure.

Note: The designations and diagnostic characteristics listed above are most applicable to crystalline rocks with feldspars and ferromagnesian minerals. Weathering in some other rock types, particularly young and/or poorly indurated sedimentary units, may not entirely conform to the above designations and criteria.

Scale of Relative Rock Hardness

Relative rock hardness and strength of intact rock is described based on the following criteria:

Designation	Field Identification
Extremely Soft (ES)	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.
Very Soft (VS)	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocketknife. Scratched with fingernail.
Soft (S)	Can be peeled by a pocketknife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.
Medium Hard (MH)	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.
Hard (H)	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.
Very Hard (VH)	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.

Discontinuity Spacing Terms

The spacing of natural discontinuities in rock masses, including fractures, joints, bedding and bedding-parallel separations, foliation, and shears, is described based on the following criteria:

Spacing	Joint/Fracture Spacing Terms	Bedding/Foliation Spacing Terms
Less than 2 inches	Very Close (VC)	Very Thin (VTn) – Laminated if bedding less than ½ inch
2 inches – 1 foot	Close (C)	Thin (Tn)
1 foot – 3 feet	Moderately Close (MC)	Medium (M)
3 feet – 10 feet	Wide (W)	Thick (Tk)
More than 10 feet	Very Wide (VW)	Very Thick (VTk) [massive]

Note: The spacings of discontinuities indicated within the columnar field on the Boring Logs are typically provided as apparent spacing measurements (along core axis) between any and all adjacent observed natural discontinuities, regardless of type or orientation. The apparent spacing measurements may, however, over- or underestimate the true spacings of individual sets of joints or fractures depending upon their orientation and/or intersection. Where possible, the true spacing of individual sets of discontinuities are measured and noted in the Material Description. Additionally, the documentation of close or very close spacings of healed, filled, incipient, or otherwise intact and unbroken discontinuities may appear to conflict with RQD values recorded on the Boring Log, and the presence of such features are noted to the extent practical in the Material Description.

Project Name:	**Frisco Avenue Bridge**	Boring Number:	**B-1**
Project Location:	Clifton, Arizona	Boring Location:	South Abutment
Project Number:	2021039	Coordinates:	33.05721° N, -109.30015° W
Logger - Firm:	M. Meza - Ethos	Surface Elevation (feet):	3,468
Drilling Method:	HSA - HQ Core	Groundwater Depth (feet):	n/a
Driller - Firm:	Eric - GSI	Date(s):	6/21/2022

Elev (feet)	Depth (feet)	Drill Rate (min/ft)	Sample Interval	Sample Type (& Blowcounts)	% Recovery	Rock Quality Designation (RQD)	% Fluid Recovery	Spacing of Discontinuities	Orientation of Discontinuities	Degree of Weathering	Relative Rock Hardness	Material Description
3,467	1											**Clayey to SIlty Gravel with Sand (GC-GM)** Sub-Angular Gravel Well Graded Sand Low Plasticity Brown to Dark Brown Moist
3,466	2											
3,465	3											
3,464	4			S 4-4-7 (11)								
3,463	**5**											
3,462	6			R 14-11 (25)								
3,461	7											
3,460	8			A (5-10)								
3,459	9											**Sandy Silt (ML)** Trace Fine Sub-Angular Gravel Trace to Some Fine Sand Non Plastic Light Brown No Lime to Weak Lime Cementation Moist
3,458	**10**											
3,457	11			S 3-1-2 (3)								
3,456	12											
3,455	13											
3,454	14											**Silty Sand (SM)** Trace to Some Fine Sub-Angular Gravel Poorly Graded Sand Non Plastic Light Brown Moist
3,453	**15**											
3,452	16			R 17-28 (45)								
3,451	17											**Silty Gravel with Sand (GP-GM)** Subangular Gravel Considerable Medium Sand Non - Plastic Grey to Brown Moist
3,450	18											
3,449	19											
3,448	**20**											Note: Began Coring @ 20' Note: Considerable cobbles and some boulders below 20'
3,447	21			S - PR 50/5" (100)								
3,446	22							n/a	n/a	n/a	n/a	
3,445	23	8		HQ	30	n/a	0					
3,444	24											
3,443	**25**											

Sample Type
- S - SPT Spoon Sampler
- R - Ring Sampler
- A - Auger Cuttings
- HQ - Wireline Core

Drilling Operation
- NQ - Wireline Core
- HSA - Hollow Stem Auger
- GB - Gearbit
- HWT - Casing Adv. w/ Wireline GB
- HQ - Wireline Core

Discontinuities
- VW >10.0'
- W 3.0'-10.0'
- MC 1.0'-3.0'
- C 0.2'-1.0'
- VC 0-0.2'

Rock Hardness
- ES - Extremely Soft
- VS - Very Soft
- S - Soft
- MH - Medium Hard
- H - Hard
- VH - Very Hard

Notes
- NR - No Recovery
- PR - Poor Recovery
- BKN - Broken

ethos ENGINEERING, LLC.

Project Name:	Frisco Avenue Bridge	Boring Number:	B-1
Project Location:	Clifton, Arizona	Boring Location:	South Abutment
Project Number:	2021039	Coordinates:	33.05721° N, -109.30015° W
Logger - Firm:	M. Meza - Ethos	Surface Elevation (feet):	
Drilling Method:	HSA - HQ Core	Groundwater Depth (feet):	
Driller - Firm:	Eric - GSI	Date(s):	6/21/2022

Elev (feet)	Depth (feet)	Drill Rate (min/ft)	Sample Interval	Sample Type (& Blowcounts)	% Recovery	Rock Quality Designation (RQD)	% Fluid Recovery	Spacing of Discontinuities	Orientation of Discontinuities	Degree of Weathering	Relative Rock Hardness	Material Description
	26–30	6	HQ - Wireline Core	HQ	15	n/a	40	n/a	n/a	n/a	n/a	Silty Gravel with Sand (GP-GM) Cont'd.
	31											Stopped HQ Coring at 30' Backfilled Hole w/ Cutting
	32											
	33											
	34											
	35											
	36											
	37											
	38											
	39											
	40											
	41											
	42											
	43											
	44											
	45											
	46											
	47											
	48											
	49											
	50											

Sample Type
- S - SPT Spoon Sampler
- R - Ring Sampler
- A - Auger Cuttings
- HQ - Wireline Core

Drilling Operation
- NQ - Wireline Core
- HSA - Hollow Stem Auger
- GB - Gearbit
- HWT - Casing Adv. w/ Wireline GB
- HQ - Wireline Core

Discontinuities
- VW >10.0'
- W 3.0'-10.0'
- MC 1.0'-3.0'
- C 0.2'-1.0'
- VC 0-0.2'

Rock Hardness
- ES - Extremely Soft
- VS - Very Soft
- S - Soft
- MH - Medium Hard
- H - Hard
- VH - Very Hard

Notes
- NR - No Recovery
- PR - Poor Recovery
- BKN - Broken

ethos ENGINEERING, LLC.

Start @20'
22'
22
23
THIS SIDE UP
25'
26'
26'
28'
END @30'

APPENDIX B

Laboratory Test Results

Table B-1: Summary of Laboratory Test Results

Boring Number	Depth (ft)		USCS/Group Symbol (ASTM D2487)	Percent Fines (minus #200) (ASTM C136)	Liquid Limit (ASTM D4318)	Plasticity Index (ASTM D4318)	Moisture Content (%) (ASTM D2216/ D2937)	pH (AZ 236)	Resistivity ohm-cm[1] (AZ 236)	Sulfates (ppm)[2] (AZ 733)	Chlorides (ppm)[2] (AZ 736)
	Begin	End									
B-1	6	10	GC-GM	27.4	20	6	12.8	8.3	1,200	217	16
B-1	10	11.5	ML	77.3	NV	NP	18.5				
B-1	15	16	SM	19.8	NV	NP	8.8				
Average				**41.5**	**---**	**---**	**13.4**	**8.3**	**1,200**	**217**	**16**
Standard Deviation				**31.2**	**---**	**---**	**4.9**	**---**	**---**	**---**	**---**
Maximum				**77**	**20**	**6**	**18.5**	**8.3**	**1,200**	**217**	**16**
Minimum				**19.8**	**NV**	**NP**	**8.8**	**8.3**	**1,200**	**217**	**16**
Count				**3**	**3**	**3**	**3**	**1**	**1**	**1**	**1**

Notes:

1) ohm-cm = ohm-centimeters
2) ppm = parts per million

ACS SERVICES LLC

Laboratory Soil Test Results

ACS PROJECT #	2201755	Material Type:	Soil
ACS Lab #	22-3662-1	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	6/21/2022
Project Name:	Frisco Avenue Bridge	Sampled By:	Client
Project Address:	-	Test Date:	8/16/2022
Project City	Graham County	Tested By:	Brian Karl
Sample Location:	B-1 @ 6 - 10	Reviewed By:	Dylan Ward

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"	4	96	
3/4"	4	92	
1/2"	9	83	
3/8"	6	77	
1/4"	11	66	
#4	7	59	
#8	7	52	
#10	2	50	
#16	6	44	
#30	5	38	
#40	2	36	
#50	1	35	
#100	2	33	
#200	5	27.4	

Liquid Limit (AASHTO T-89)	26
Plastic Limit (AASHTO T-90)	20
Plasticity Index (AASHTO T-90)	6
Moisture Content (AASHTO T-265)	12.8
USCS Soil Classification	GC-GM

Group Name (ASTM D2487)
Silty, clayey GRAVEL with sand

Dylan Ward
Laboratory Manager

Dylan Ward
Signature

ACS Services LLC	Laboratory Soil Test Results

ACS PROJECT #	2201755	Material Type:	Soil
ACS Lab #	22-3662-2	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	6/21/2022
Project Name:	Frisco Avenue Bridge	Sampled By:	Client
Project Address:	-	Test Date:	8/16/2022
Project City	Graham County	Tested By:	Brian Karl
Sample Location:	B-1 @ 10.0 - 11.5	Reviewed By:	Dylan Ward

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"	0	100	
3/8"	0	100	
1/4"	0	100	
#4	0	99	
#8	0	99	
#10	0	99	
#16	0	99	
#30	0	99	
#40	0	99	
#50	0	99	
#100	2	96	
#200	19	77.3	

Liquid Limit (AASHTO T-89)	
Plastic Limit (AASHTO T-90)	
Plasticity Index (AASHTO T-90)	NP
Moisture Content (AASHTO T-265)	18.5
USCS Soil Classification	ML

Group Name (ASTM D2487)
SILT with sand

Dylan Ward
Laboratory Manager

Dylan Ward
Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2201755	Material Type:	Soil
ACS Lab #	22-3662-3	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	6/21/2022
Project Name:	Frisco Avenue Bridge	Sampled By:	Client
Project Address:	-	Test Date:	8/16/2022
Project City	Graham County	Tested By:	Brian Karl
Sample Location:	B-1 @ 15 - 16	Reviewed By:	Dylan Ward

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"	7	93	
3/4"	12	82	
1/2"	6	76	
3/8"	6	70	
1/4"	5	65	
#4	2	63	
#8	5	58	
#10	1	57	
#16	5	52	
#30	9	43	
#40	6	37	
#50	6	31	
#100	7	25	
#200	5	19.8	

Liquid Limit (AASHTO T-89)	
Plastic Limit (AASHTO T-90)	
Plasticity Index (AASHTO T-90)	NP
Moisture Content (AASHTO T-265)	8.8

USCS Soil Classification	SM

Group Name (ASTM D2487)
Silty SAND with gravel

Dylan Ward
Laboratory Manager

Dylan Ward
Signature

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

Project #	2201755	Material Type:	Soil
Lab #	22-3662-1	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	6/21/2022
Project Name:	Frisco Avenue Bridge	Sampled By:	Client
Project Address:	-	Test Date:	Wednesday, August 17, 2022
Project City:	Graham County	Tested By:	Brian Karl
Sample Source:	B-1 @ 6 - 10	Resistivity Box:	-
		Reviewed By:	Dylan Ward

pH Reading = 8.3

P = (SBF) x R x M

Where:

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

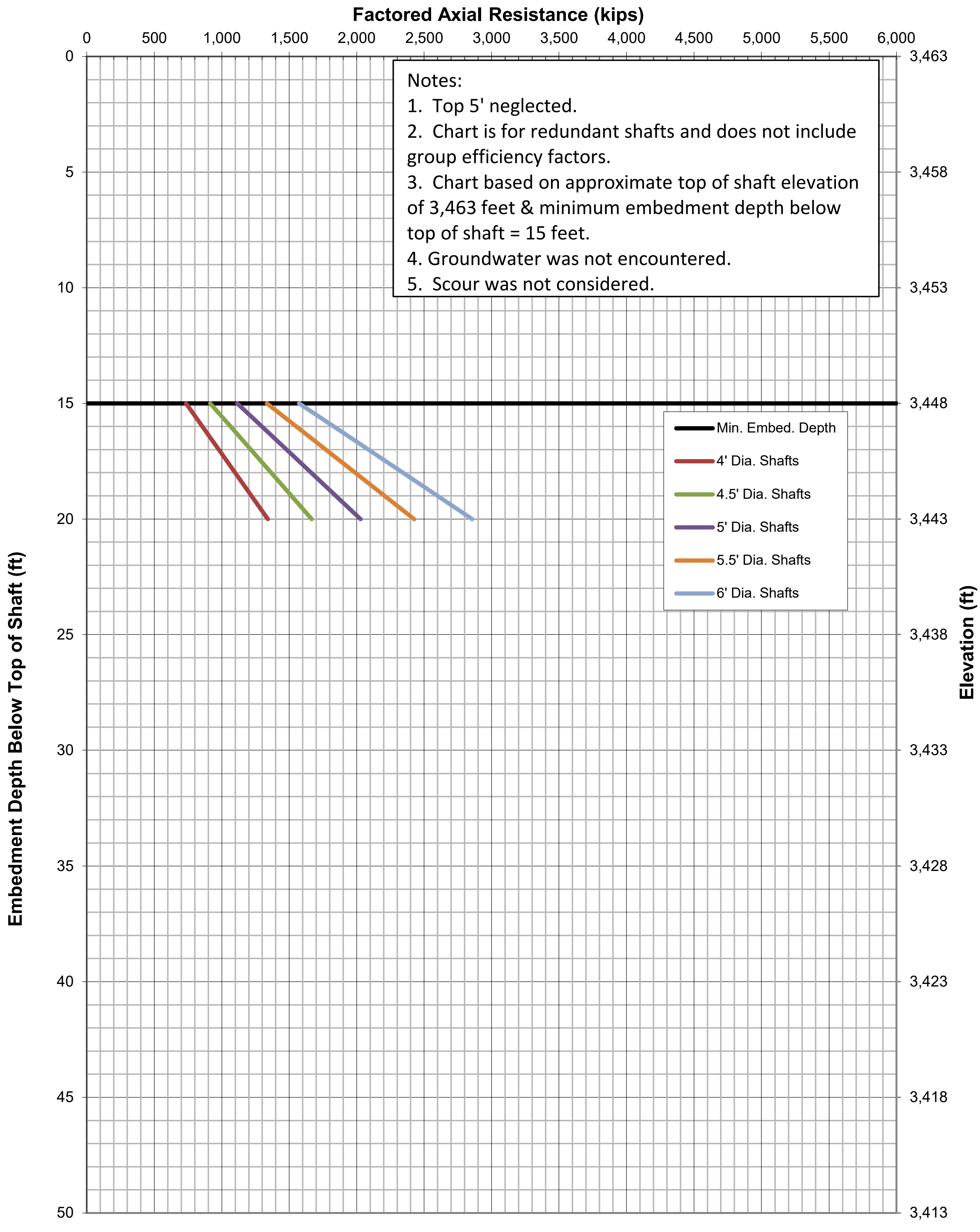
Water Added	SBF (cm)	Dial Reading (OHMS)	Multiplier	**P** (OHM-cm)
200	1	2.5	1000	2500
50	1	1.6	1000	1600
50	1	1.4	1000	1400
50	1	1.3	1000	1300
50	1	1.25	1000	1250
50	1	1.2	1000	1200
50	1	1.2	1000	1200
50	1	1.3	1000	1300

Brian Karl
Lab Supervisor

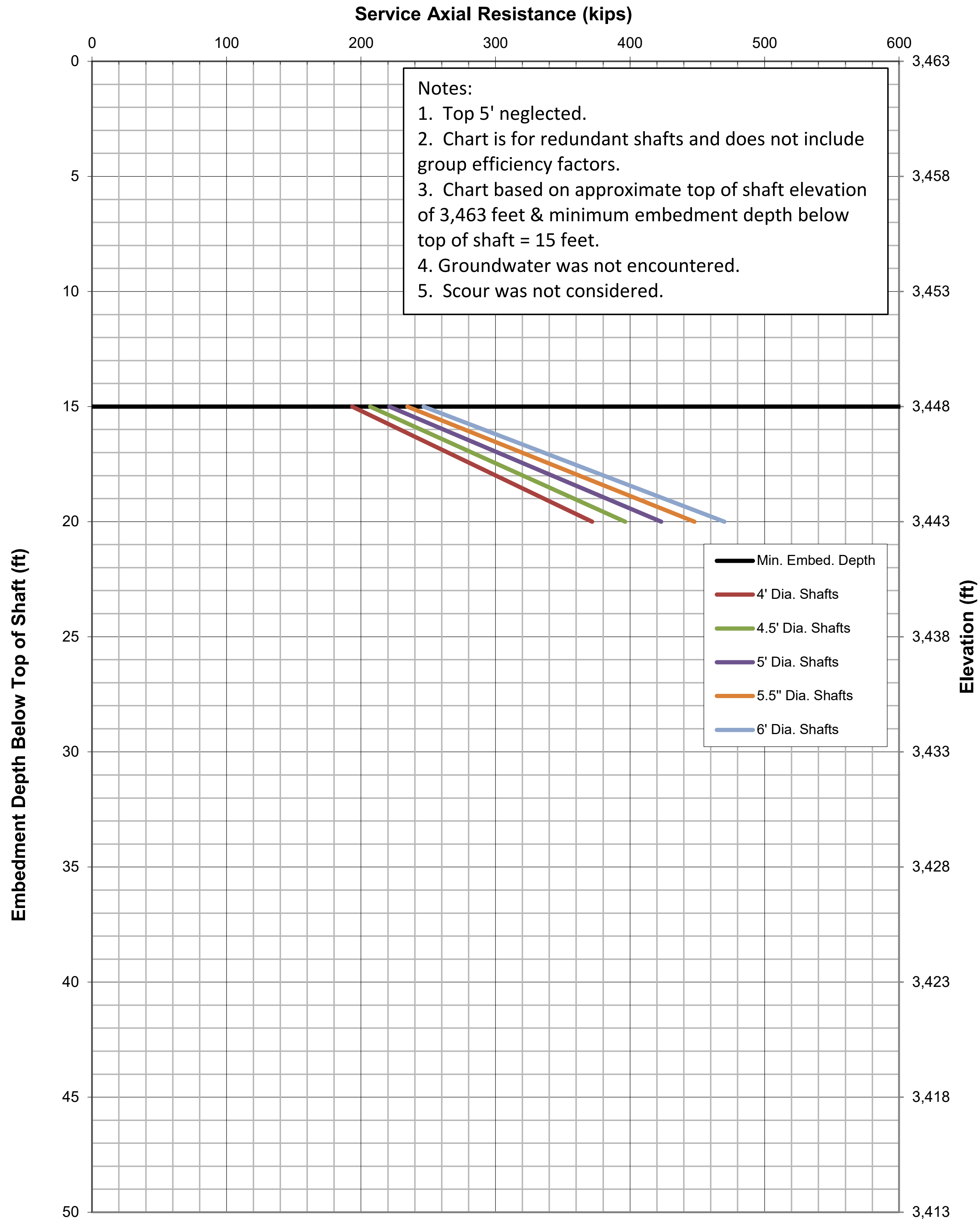
Dylan Ward
Laboratory Manager

APPENDIX C

Drilled Shaft Axial Resistance Charts



ethos ENGINEERING, LLC.		**DRILLED SHAFT FOUNDATION DESIGN CHART** **Strength Limit Axial Resistance in Kips** **2021039 - Frisco Avenue Bridge** **Bridge Abutments**	Figure C1
Designer:	Date:		
K. Mackay	8/18/2022		



ethos ENGINEERING, LLC.

Designer:	Date:
K. Mackay	8/18/2022

DRILLED SHAFT FOUNDATION DESIGN CHART

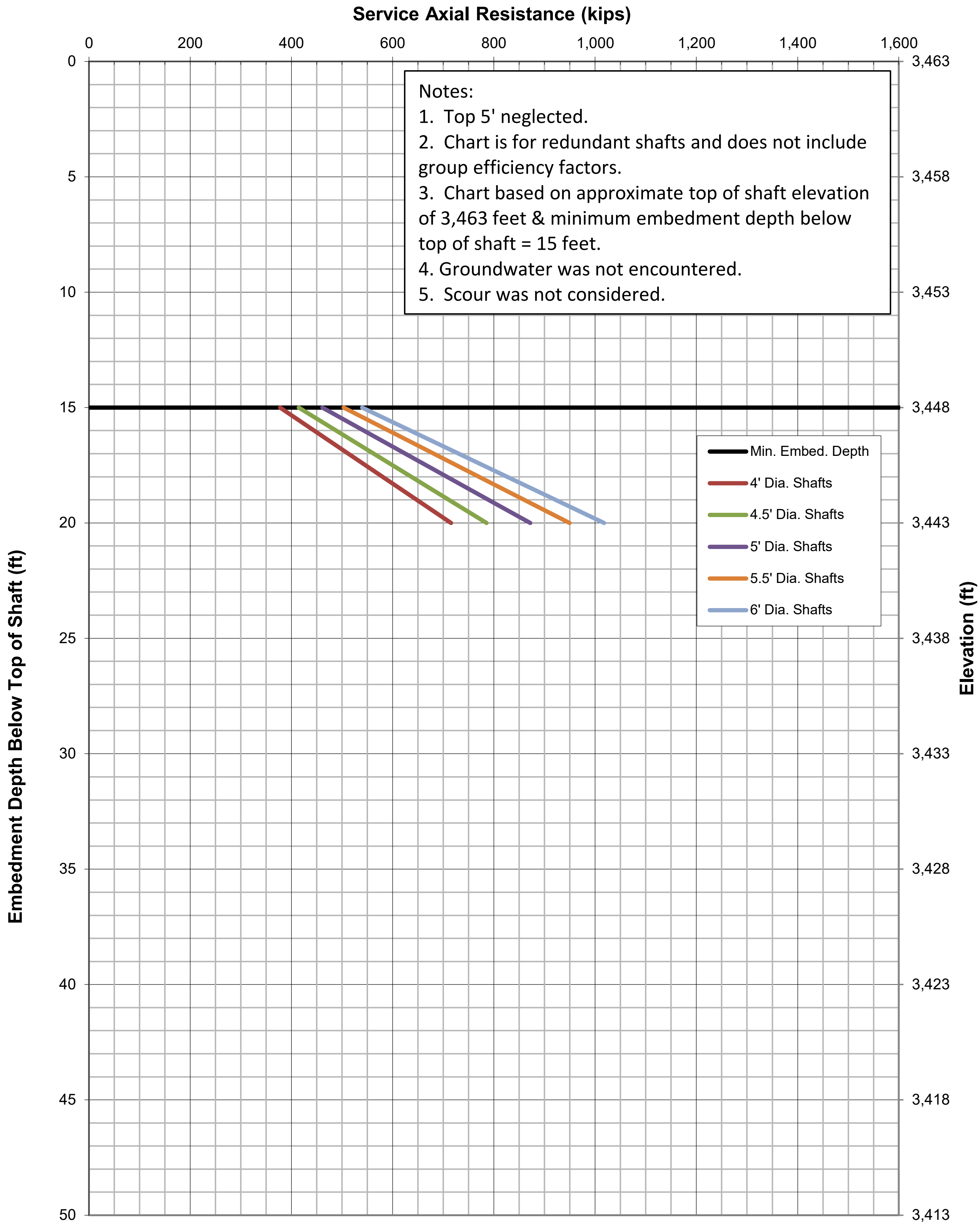
Service Limit (0.10 Inch Settlement) - Axial Resistance in Kips

2021039 - Frisco Avenue Bridge

Bridge Abutments

Figure

C2



ethos ENGINEERING, LLC.

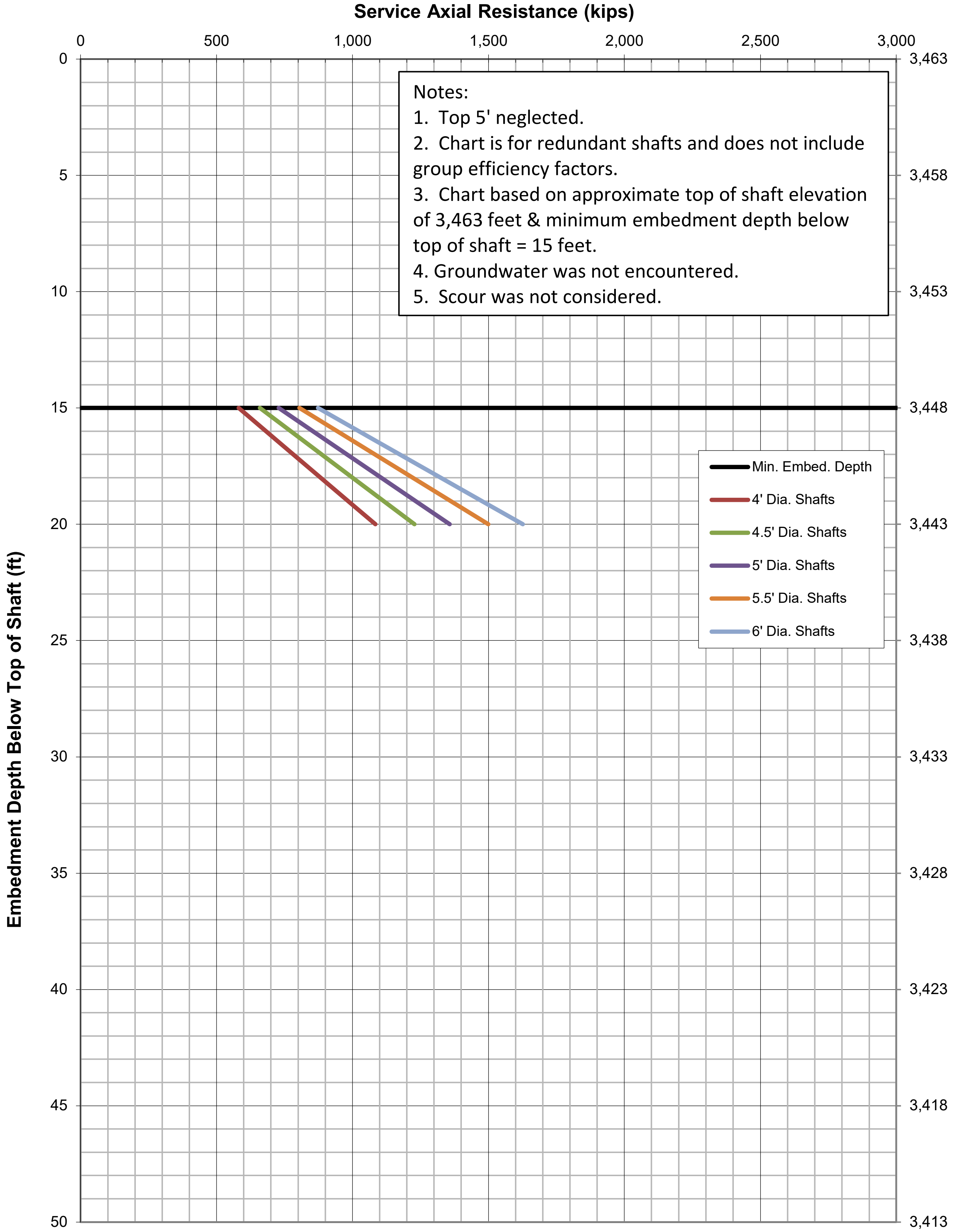
Designer:	Date:
K. Mackay	8/18/2022

DRILLED SHAFT FOUNDATION DESIGN CHART
Service Limit (0.25 Inch Settlement) - Axial Resistance in Kips

2021039 - Frisco Avenue Bridge
Bridge Abutments

Figure

C3



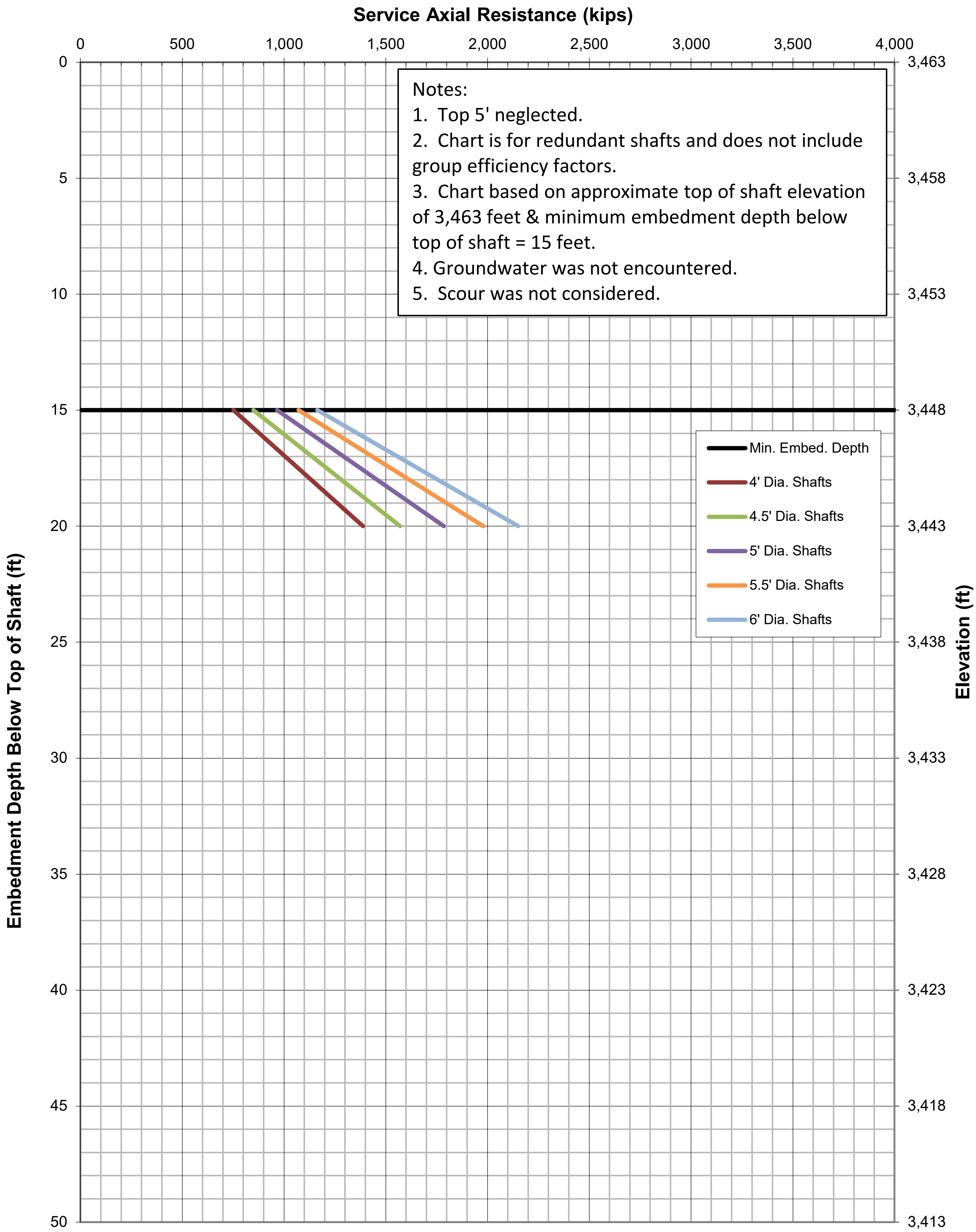
ethos ENGINEERING, LLC.

Designer:	Date:
K. Mackay	8/18/2022

DRILLED SHAFT FOUNDATION DESIGN CHART
Service Limit (0.5 Inch Settlement) - Axial Resistance in Kips

2021039 - Frisco Avenue Bridge
Bridge Abutments

Figure
C4



ethos ENGINEERING, LLC.

Designer:	Date:
K. Mackay	8/18/2022

DRILLED SHAFT FOUNDATION DESIGN CHART

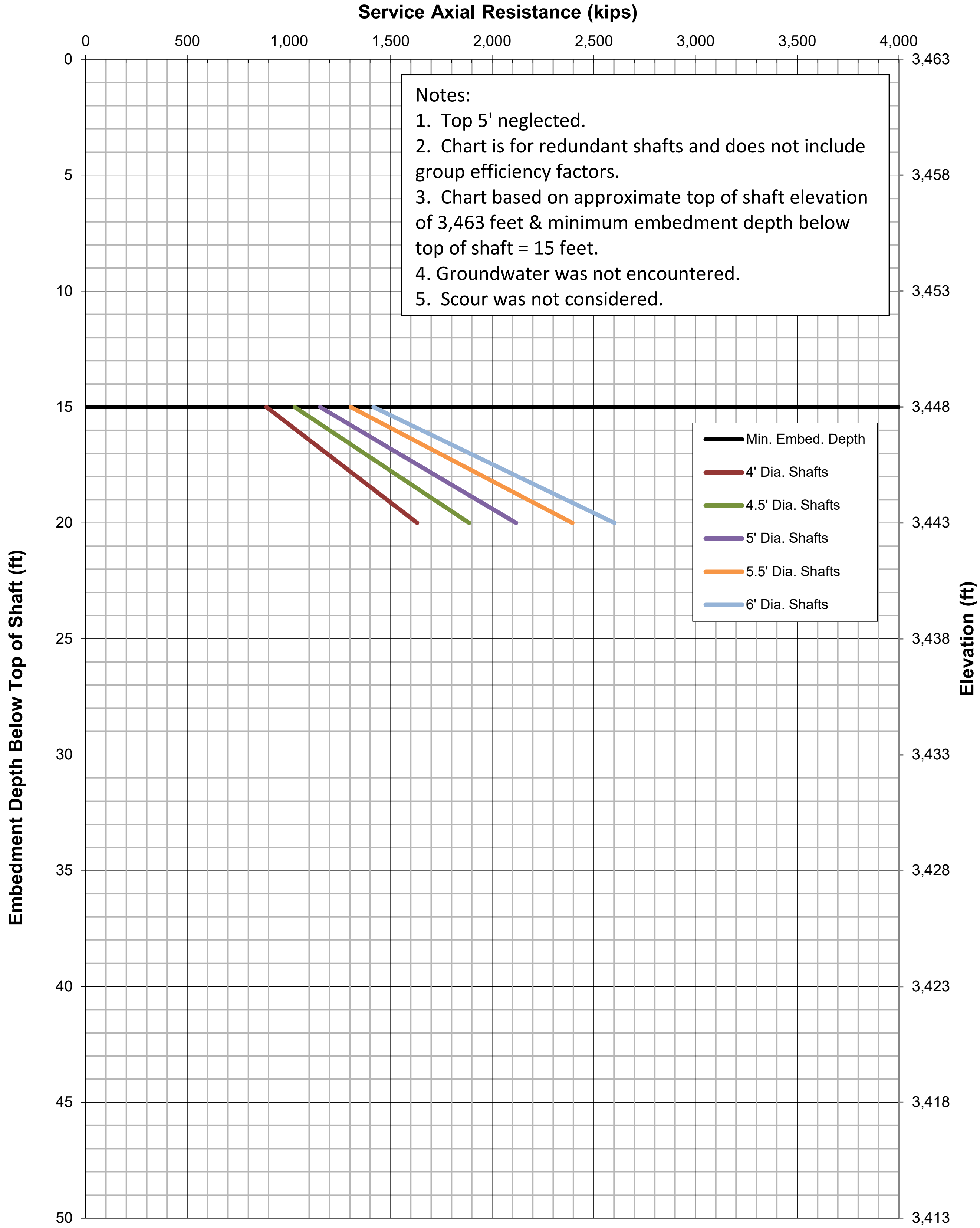
Service Limit (0.75 Inch Settlement) - Axial Resistance in Kips

2021039 - Frisco Avenue Bridge

Bridge Abutments

Figure

C5



ethos ENGINEERING, LLC.		**DRILLED SHAFT FOUNDATION DESIGN CHART** **Service Limit (1 Inch Settlement) - Axial Resistance in Kips** **2021039 - Frisco Avenue Bridge** **Bridge Abutments**	Figure C6
Designer:	Date:		
K. Mackay	8/18/2022		

APPENDIX D

Seismic Refraction Evaluation Report

GEOTECHNICAL EVALUATION

FRISCO AVENUE BRIDGE SEISMIC STUDY

Clifton, Arizona

PREPARED FOR:

Ethos Engineering LLC
9180 South Kyrene Road #104
Tempe, AZ 85284

PREPARED BY:

Atlas Technical Consultants LLC
9185 South Farmer Avenue, Suite 111
Tempe, AZ 85284

August 25, 2022

9185 South Farmer Avenue, Suite 111
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(480) 855-3201 | oneatlas.com

August 25, 2022

Atlas No. 222035SWG
Report No. 1

MR. KEITH DAHLEN P.E.
ETHOS ENGINEERING
9180 SOUTH KYRENE ROAD #104
TEMPE, AZ 85284

Subject: **Geophysical Evaluation**
Frisco Avenue Bridge Seismic Study
Clifton, Arizona

Dear Mr. Dahlen:

In accordance with your authorization, Atlas Technical Consultants (Atlas) has performed a geophysical evaluation pertaining to the characterization of subsurface conditions at the project site located in Clifton, Arizona. The primary purpose of our seismic investigation is to develop subsurface velocity profiles along predetermined seismic traverses to characterize subsurface conditions. This report presents the survey methodology, equipment used, analysis, and findings from our study. Our field services were conducted on August 11, 2022.

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

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED GEOLOGIST
CERTIFICATE NO.
52449
MARK D.
EDWARDS
Date Signed 8/25/2022
ARIZONA, U.S.A.
Expires 06/30/23

Andrew S. Baird
Project Geologist/Geophysicist

Mark D. Edwards, R.G.
Principal Geologist/Geophysicist

SW:LRC:ASB:mde:ds

Distribution: Kdahen@ethosengineering.com

CONTENTS

TABLE

FIGURES

1. INTRODUCTION

Atlas has performed a geophysical evaluation pertaining to the characterization of subsurface conditions at the project site located in Clifton, Arizona (Figure 1). The primary purpose of our seismic investigation is to develop subsurface velocity profiles along predetermined seismic traverses to characterize subsurface conditions. This report presents the survey methodology, equipment used, analysis, and findings from our study. Our field services were conducted on August 11, 2022.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of two seismic refraction microtremor (ReMi) profiles at the project site: RL-1 and RL-2.
- Performance of two seismic refraction lines at the project site: SL-1 and SL-2.
- Compilation and analysis of the data collected, producing this report.
- Preparation of this data report presenting our results and conclusions.

3. SITE AND PROJECT DESCRIPTION

Our geophysical evaluation was performed at the project site located adjacent to Frisco Avenue in Clifton, Arizona. ReMi profiles, RL-1 and RL-2, and seismic refraction profiles, SL-1 and SL-2, were conducted at predetermined evaluation locations west of Frisco Avenue (Figure 2). The SL-1 seismic line was performed within an active work yard, SL-2 within the driveway/parking lot of an adjacent business. Figures 3a and 3b depict the general site conditions in the study area.

Based on our discussions with the on-site Ethos field representative, our designed exploration depth with the seismic refraction method was up to approximately 30 feet below ground surface (bgs) and for the ReMi method was up to approximately 100 feet bgs at the evaluated traverse locations, with the actual depth resolved dependent on subsurface site conditions and other variables.

4. EVALUATION METHODOLOGY AND ANALYSIS

4.1 Seismic Refraction Microtremor (ReMi)

Our scope of services for the project included the performance of two one-dimensional (1-D) ReMi profiles (RL-1 and RL-2) at the designated locations at the project site (Figure 2). The ReMi passive seismic technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The ReMi method does not require an increase of material velocity with depth; therefore, low velocity zones (velocity inversions) are detectable with the ReMi method.

Our ReMi evaluation included the use of a 24-channel Geometrics Strataview seismograph and 24, 4.5-Hz vertical component geophones. The geophones were spaced 5 feet apart for a total line length of 115 feet. A total of fourteen records, each 32 seconds in duration, was recorded at each line location and then downloaded to a field computer. The data were later processed using Surface Plus 9.1 – Advanced Surface Wave Processing Software (Geogiga Technology Corp., 2020), which uses the refraction microtremor method (Louie, 2001) and other surface wave analysis methods. The program generates phase-velocity dispersion curves for each record and provides an interactive dispersion modeling tool where the users determine the best fitting model. The result is a 1-D shear-wave velocity model of the site with roughly 85 to 95 percent accuracy. Figure 3 depicts the general site conditions in the study area.

4.2 Seismic Refraction

Our scope of services for the project included the performance of two seismic refraction profiles (SL-1 and SL-2) at predetermined locations at the project site (Figure 2). The seismic P-wave (compression wave) refraction method was conducted at the project site to characterize subsurface conditions and to develop subsurface velocity profiles at the study area. 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 manually operated hammer and strike plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 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 Time-Distance plots which can be used to evaluate presence of delayed P-wave arrivals, absence of P-wave arrivals, and estimate thickness and velocity information of the subsurface materials.

Seismic lines SL-1 and SL-2 were approximately 125 feet long between off-end shot locations using a 115-foot-long geophone spread. The shot points (signal generation locations) were conducted at the ends, midpoint, and intermediate points along the lines. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse.

The refraction method generally requires that subsurface velocities increase with depth. 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 and vertical variations in velocity, such as those caused by buried channels or washes, core stones or intrusions, nested cobbles and/or boulders, or by variations in the amount of rock weathering and depths to buried erosional surfaces of bedrock, can also result in the misinterpretation of the subsurface conditions. The presence of delayed P-wave arrivals or absence of arrivals at geophones can indicate the presence of an asperity or air gap within the subsurface soils possibly related to a developing earth fissure. However, it should be noted that other features of significant lateral and vertical extent, such as buried channels or washes, laterally restrictive sand lenses within clay, and other geologic situations can also yield similar seismic responses to those caused by earth fissures.

As previously indicated, two seismic refraction traverses were conducted as part of our study. The collected seismic refraction tomography data was processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using Rayfract® Version 4.02 (Intelligent Resources Inc., 2021). Rayfract® uses first arrival picks and elevation data to produce subsurface velocity models by the Wavepath Eikonal Traveltime (WET) method (Schuster, 1993).

5. RESULTS

The results from our seismic ReMi and seismic refraction analysis are shown on Table 1 and Figures 4a through 5b. Table 1 and Figure 4 presents the results from our ReMi evaluation. Based on our analysis of the collected data, the average characteristic site shear-wave velocity down to a depth of 100 feet bgs is 1,284 feet per second at location RL-1, and 1,337 feet per second at location RL-2. These values correspond to IBC seismic Site Class 'C'. It should be noted the ReMi result represent the average condition across the length of the line.

Table 1 – ReMi Results

Line No.	Depth (feet)	Shear Wave Velocity (feet/second)
RL-1 (W-E)	0-6	617 ft/s
	6-10	659 ft/s
	10-15	786 ft/s
	15-20	1,020 ft/s
	20-27	1,168 ft/s
	27-36	1,170 ft/s
	36-46	1,549 ft/s
	46-58	1,556 ft/s
	58-71	1,605 ft/s
	71-88	1,824 ft/s
	88-100	2,167 ft/s
RL-2 (W-E)	0-6	814 ft/s
	6-9	726 ft/s
	9-14	731 ft/s
	14-19	1,242 ft/s
	19-25	1,261 ft/s
	25-33	1,284 ft/s
	33-43	1,499 ft/s
	43-55	1,505 ft/s
	55-69	1,512 ft/s
	69-87	1,615 ft/s
	87-100	2,027 ft/s

As depicted, the tomography models reveal distinct relatively low velocity materials in the near-surface and generally relatively higher velocity materials at depth. The relatively lower velocity materials are possibly topsoil, residuum, man-placed fill materials, colluvium, and/or alluvium. The relatively higher velocity materials might represent nested cobbles and boulders, and/or the possible presence of cemented soils (Gila Conglomerate is possible) and weathered to non-weathered bedrock. Also evident in the tomography models are substantial and distinct lateral variations in velocity which may be related to nested cobbles, cemented soils, boulders, intrusions, bedrock fracturing, bedrock pinnacles, differential weathering, varying depths to bedrock, and/or a combination of these features regarding the subsurface materials.

6. 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, recommendations, 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. Atlas 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 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.

7. SELECTED REFERENCE

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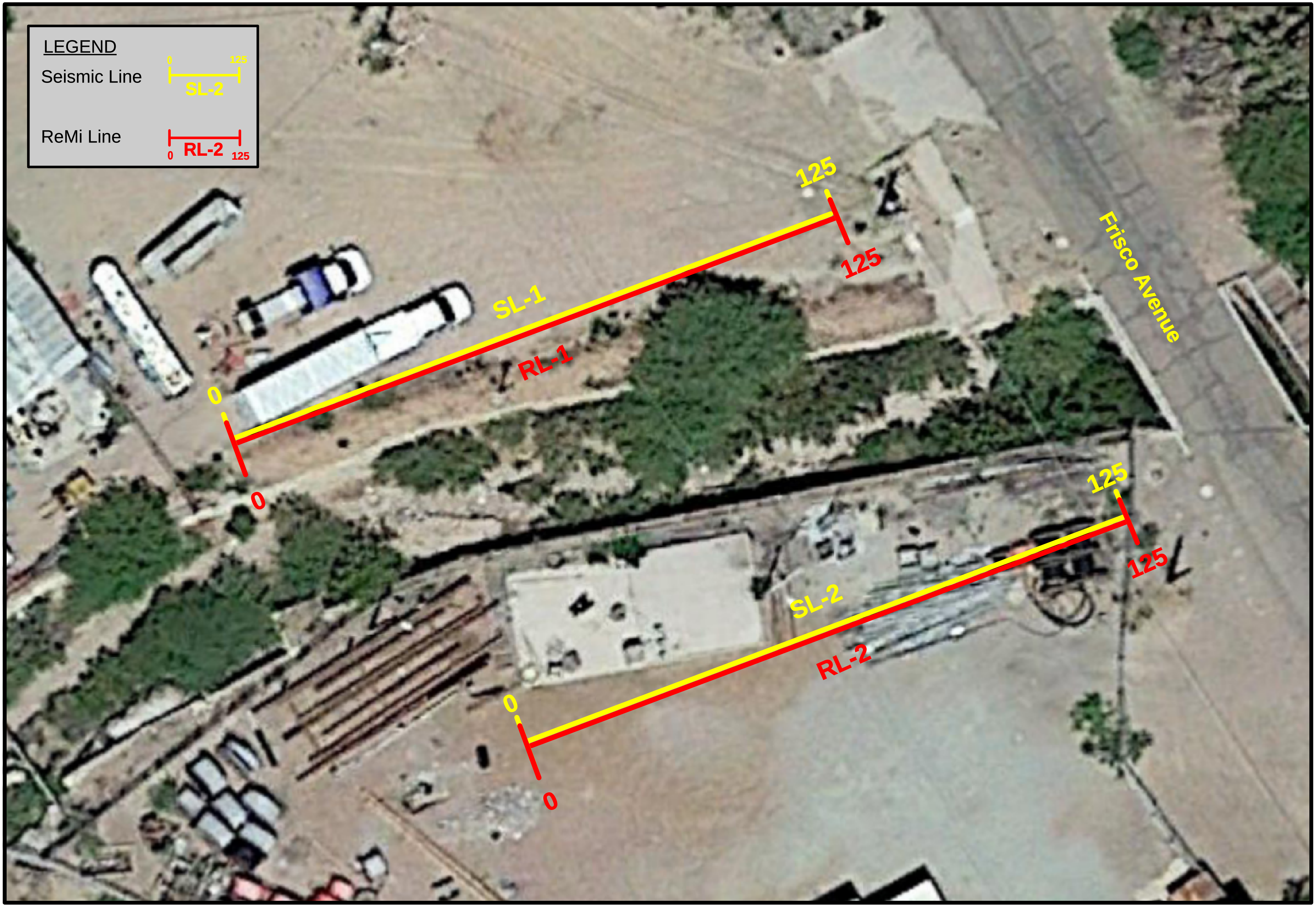
SITE LOCATION MAP	N	Frisco Avenue Bridge Seismic Study Morenci, Arizona Project No.: 222035SWG · Date: 08/22	ATLAS Figure 1



SEISMIC LINE LOCATION MAP

N

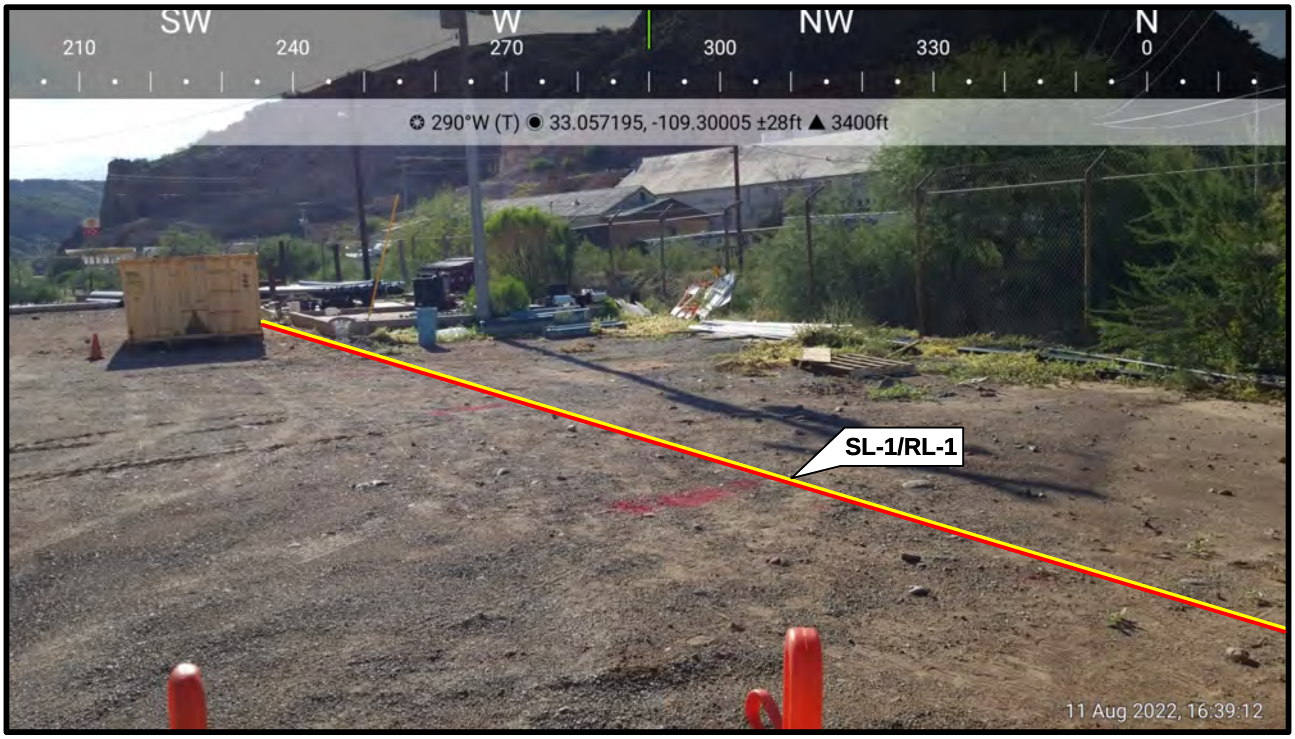
Frisco Avenue Bridge Seismic Study
Morenci, Arizona

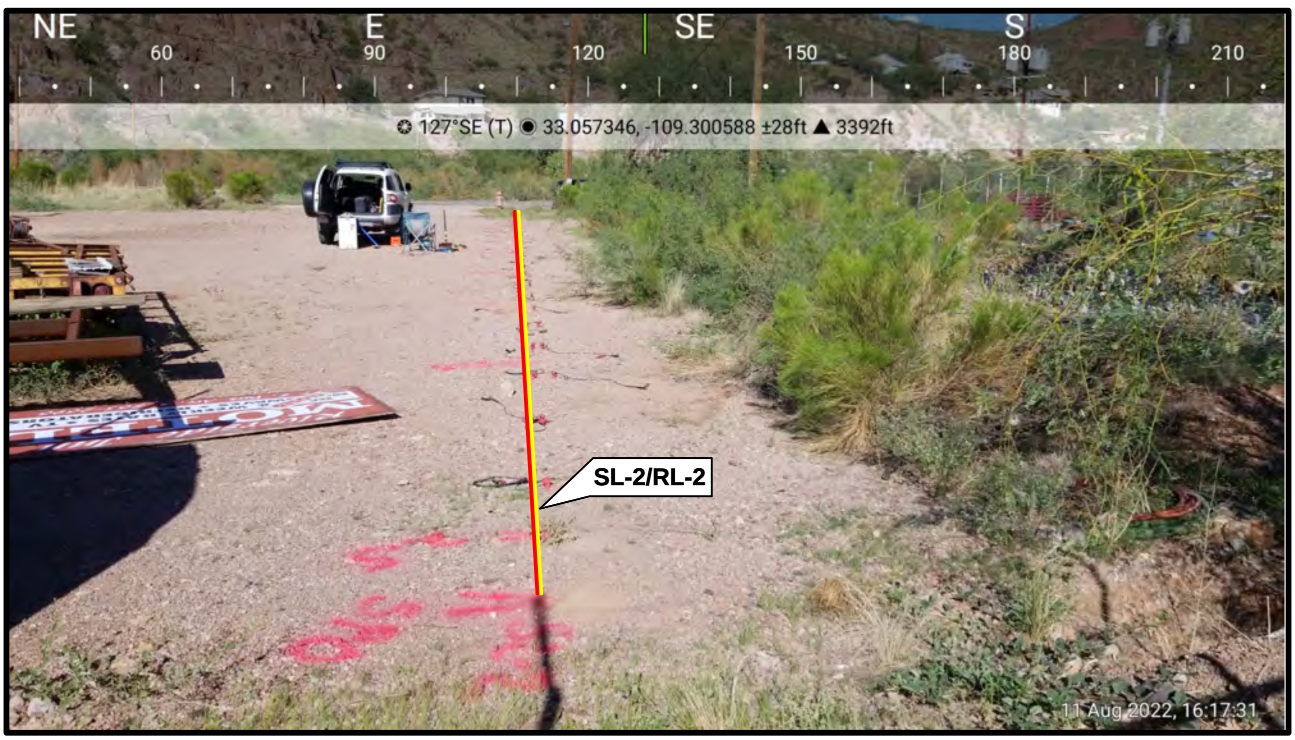
Project No.: 222035SWG | Date: 08/22

ATLAS

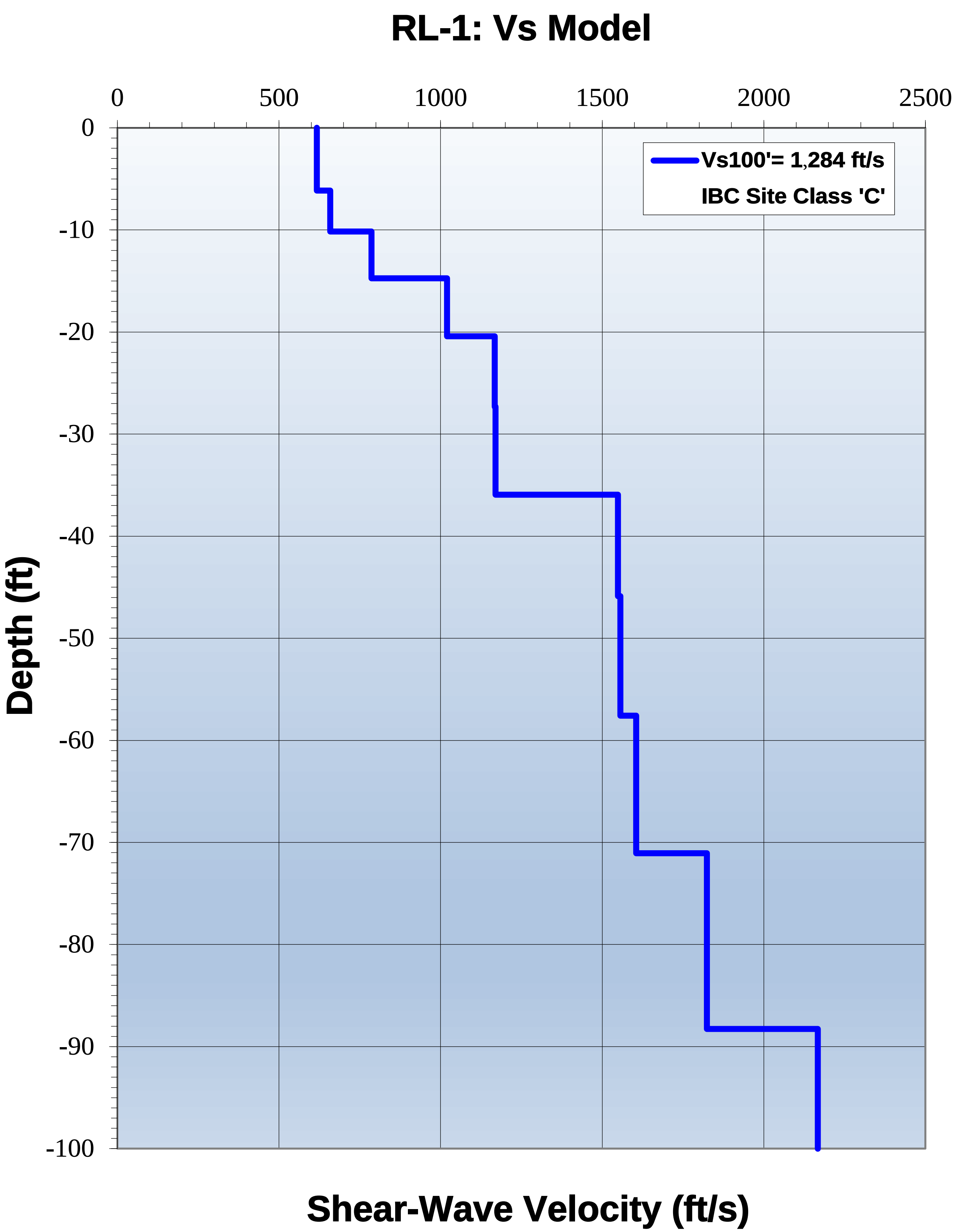
Figure 2

0 25 50
Approximate scale in feet

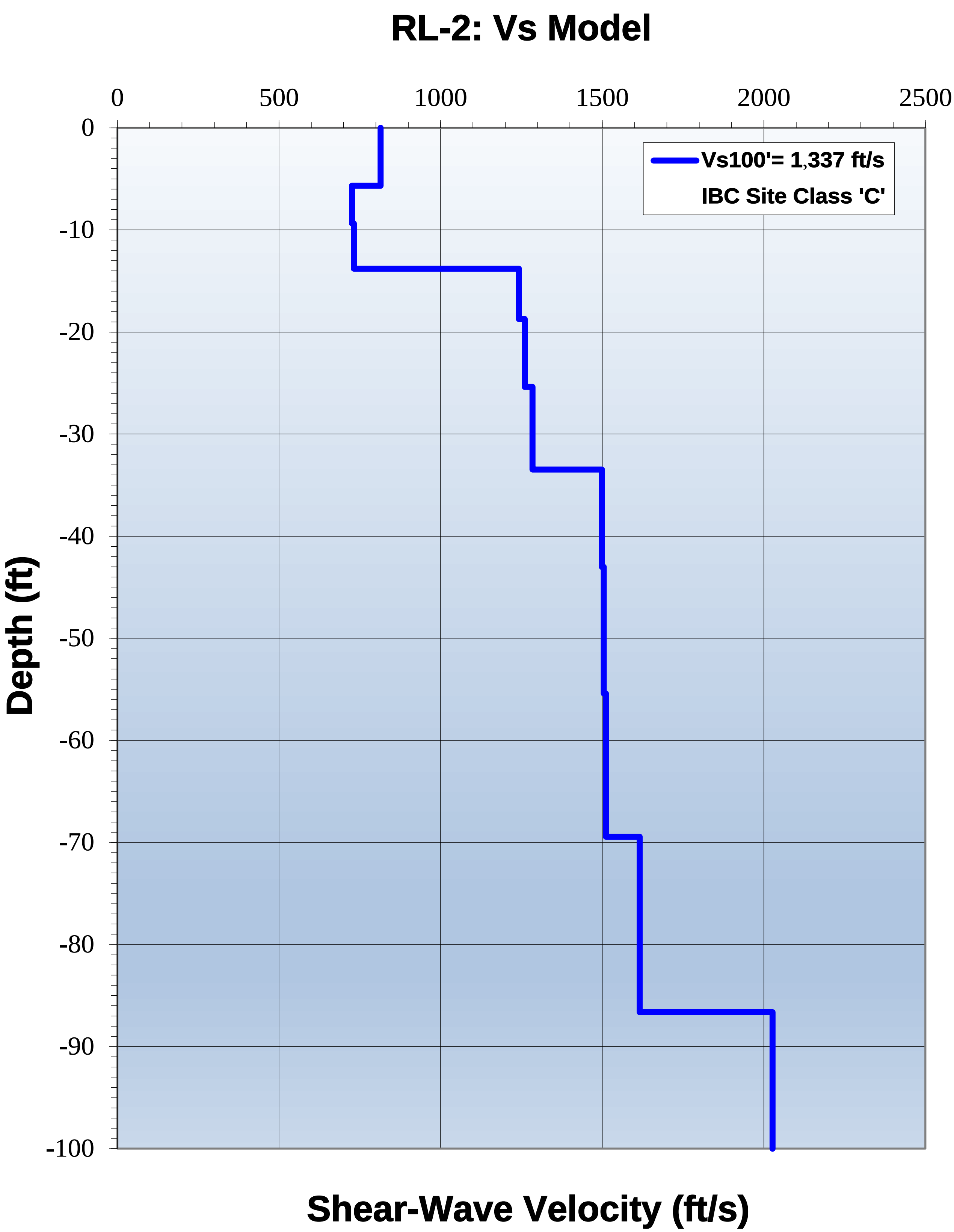




SITE PHOTOGRAPHS	Frisco Avenue Bridge Seismic Study Morenci, Arizona	ATLAS Figure 3
	Project No.: 222035SWG | Date: 08/22	

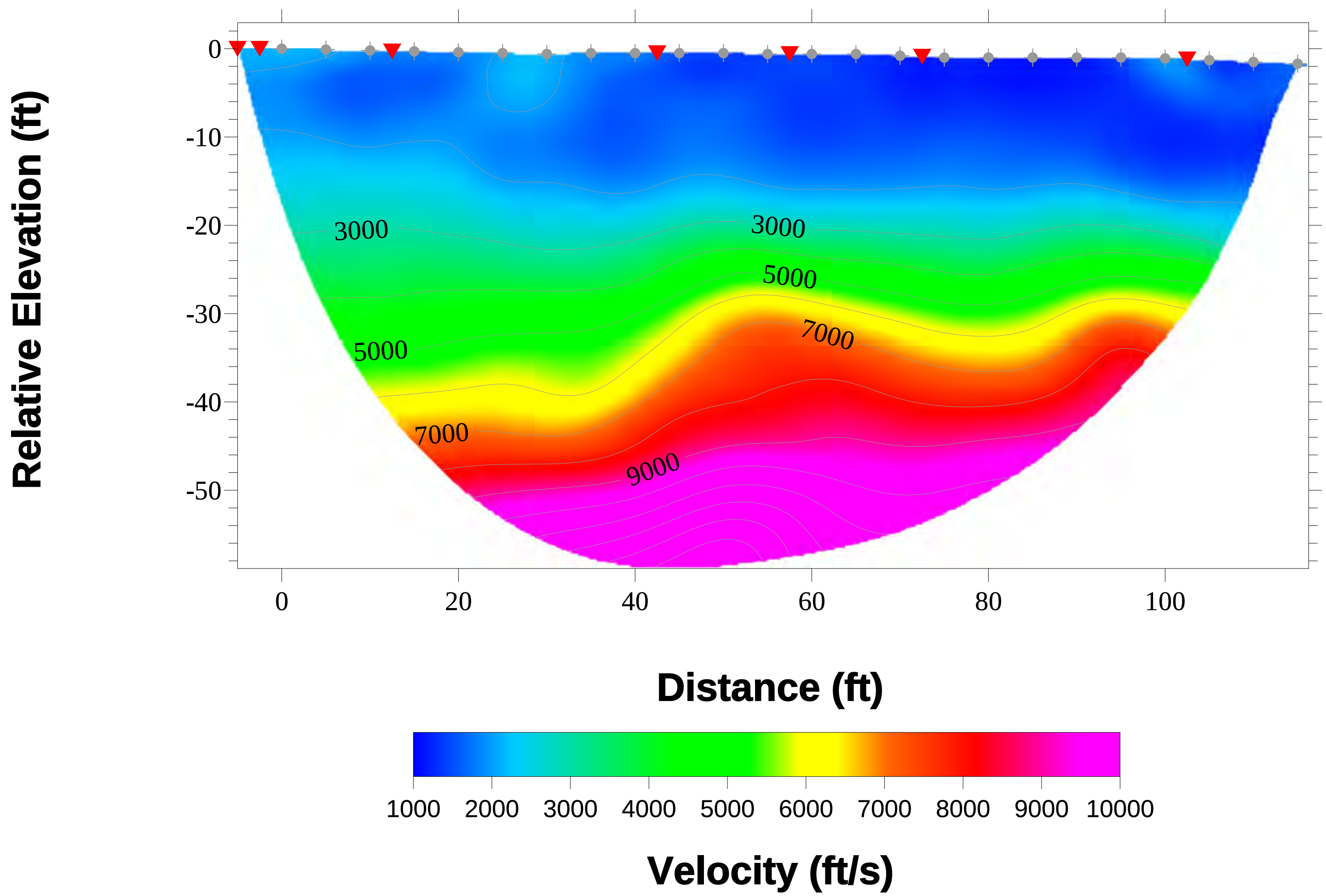


ReMi RESULTS RL-1	Frisco Avenue Bridge Seismic Study Morenci, Arizona Project No.: 222035SWG — Date: 08/22	ATLAS Figure 4a



ReMi RESULTS RL-2	Frisco Avenue Bridge Seismic Study Morenci, Arizona Project No.: 222035SWG / Date: 08/22	ATLAS Figure 4b

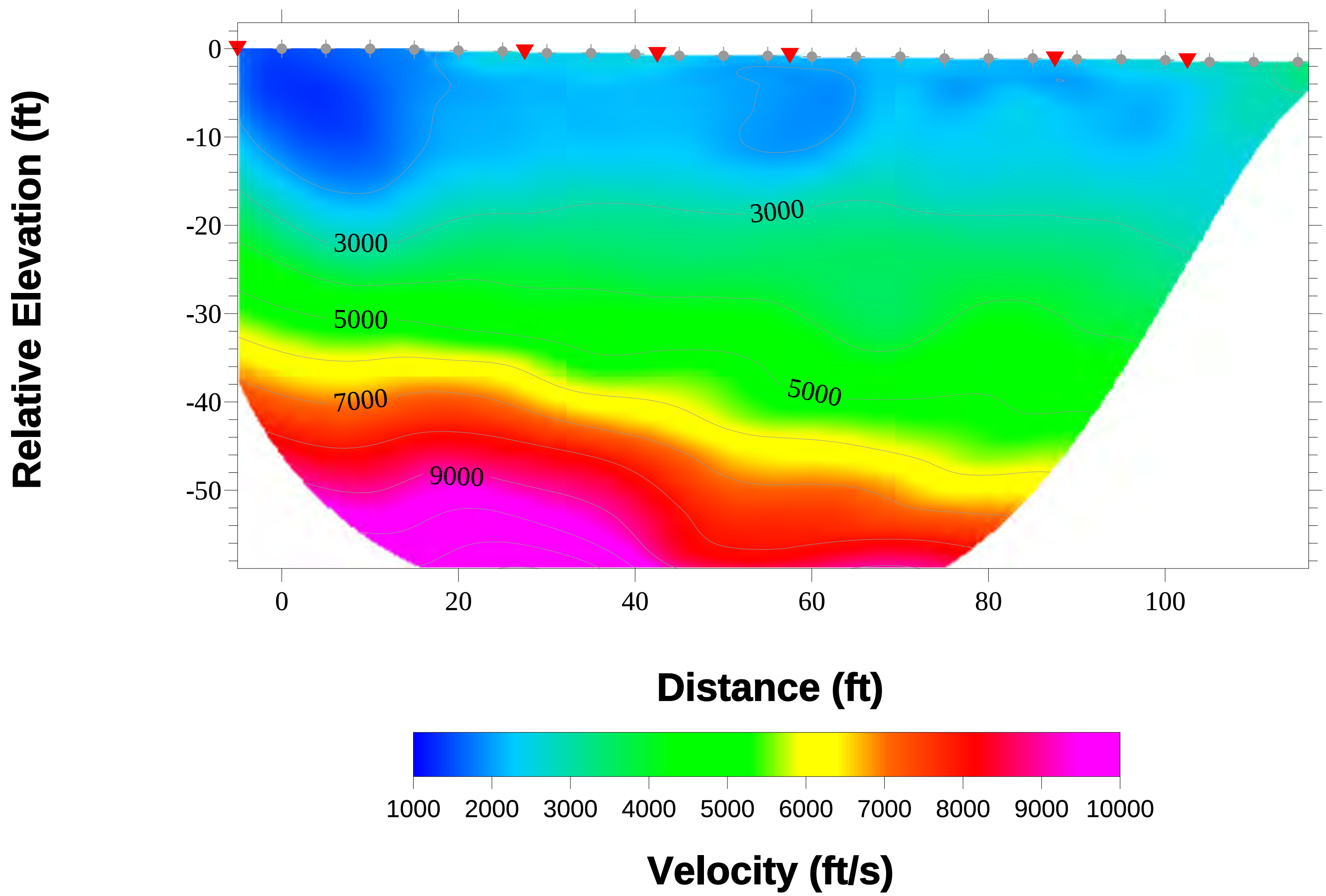
SL-1



SEISMIC PROFILE SL-1	Frisco Avenue Bridge Seismic Study Morenci, Arizona Project No.: 222035SWG Date: 08/22	ATLAS Figure 5a

Note: Contour Interval = 1,000 feet per second

SL-2



SEISMIC PROFILE SL-2	Frisco Avenue Bridge Seismic Study Morenci, Arizona	ATLAS Figure 5b
	Project No.: 222035SWG Date: 08/22	

Note: Contour Interval = 1,000 feet per second