ARIZONA DEPARTMENT OF TRANSPORTATION

FINAL GEOTECHNICAL EXPLORATION REPORT
STATE ROUTE 80, OLD BISBEE TO ERIE STREET
SHARED USE PATH
ADOT PROJECT NO. 080 CH BIS T0621 01C
FEDERAL AID NO. BIS-0(207)T
ADOT CONTRACT NO. 2022-006.08
BISBEE, ARIZONA

Prepared for:

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Ethos Project No. 2025025
April 3, 2026

Magdaleno Meza

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ARIZONA, U.S.A.

April 3, 2026
Ethos Project No.: 2025025

Kimley-Horn & Associates, Inc.
7740 North 16th Street, Suite 300
Phoenix, Arizona 85020

Attn: Mr. Nathan Merrill, P.E.

SUBJECT: Final Geotechnical Exploration Report
State Route 80, Old Bisbee to Erie Street
Shared Use Path
ADOT Project No. 080 CH BIS T0621 01C
Federal Aid No. BIS-0(207)T
ADOT Contract No. 2022-006.08
Bisbee, Arizona

Dear Nathan:

Ethos Engineering, LLC is pleased to present the findings of the geotechnical exploration for the proposed State Route 80 (SR 80), Old Bisbee to Erie Street project located in Bisbee, Arizona. Our services were conducted in general accordance with the scope of services presented in our revised proposal dated October 28, 2024. The results of our field investigation and laboratory testing, and geotechnical engineering recommendations for support of the roadway and foundation designs, are included herein. A Final Pavement Design Summary and Final Materials Design Report (Ethos 2026), based on the results included herein, are presented under separate cover.

We appreciate the opportunity to be of service to Kimley-Horn & Associates, Inc. (KHA) on this project. If you have any questions regarding this report, please do not hesitate to contact us.

Sincerely,
Ethos Engineering, LLC

Magdaleno Meza

Registered Professional Engineer (Civil)
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1.0 PROJECT DESCRIPTION

The City of Bisbee (City) project will construct the west segment of a new shared use path (SUP) from downtown Bisbee to Erie Street for a total length of approximately 900 feet. Included will be the following construction elements:

- Modify the existing Main Street roadway at State Route 80 (SR 80) to allow for a shared-use pathway (SUP) underneath the bridge. The SUP will be 8-foot wide under the bridge and transition up to 12-foot wide north and south of SR 80.
- Construct a combination soldier pile and concrete cantilever retaining wall adjacent to Main Street from the Queen mine entrance to the SR 80 Main Street underpass (UP). It is anticipated this retaining wall will be up to approximately 10-foot tall by 360-foot long and based on cross-sections will have an approximately 1.5 Horizontal:1 Vertical (H:V) slope behind.

The site is shown on the Vicinity Map presented on Figure 1.

The exploration included site reconnaissance, subsurface exploration, soil sampling, field and laboratory testing, engineering analyses, and preparation of this report. The purpose of this report is to provide information regarding the surface and subsurface soil conditions and general site geology, and to provide geotechnical recommendations for design and construction for the proposed retaining walls and roadway elements at the site.

2.0 FIELD EXPLORATION

Prior to our field exploration, Ethos obtained ADOT right-of-way use permit #1228522, dated December 15, 2025, and Bisbee permit #ROW-2025-0391, dated August 13, 2025. Upon receipt of the permits, we staked the boring locations, some of which required field adjustment due to the presence of underground or overhead utilities, surface features or other constraints. Upon locating the borings, Ethos coordinated clearing each area with Arizona 811. Various traffic control plans were prepared for this work by Quail Construction (Quail), as a subcontractor to Ethos.

The subsurface exploration for the project was performed on January 15, 2026 and included 3 exploratory borings for the proposed retaining wall and roadway improvements with drill depths ranging from 6 to 16.5 feet. All three borings were drilled through asphaltic concrete (AC) and aggregate base (AB). Table 2.1 presents a summary of the drilling program, which includes the primary design element of the boring, boring coordinates and boring depths.

Table 2.1: Subsurface Exploration Details

Boring ID	Project Element	Latitude [°N]	Longitude [°E]	Boring Depth[feet]
B-1	Retaining Wall/ Pavement	31.44080	-109.91318	16
B-2	Retaining Wall/ Pavement	31.44053	-109.91283	16.5
B-3	Retaining Wall/ Pavement	31.44031	-109.91243	6[1]

NOTES
1. Boring encountered auger refusal before achieving proposed depth of 10 feet.

No existing geotechnical data along SR 80 was provided. The boring spacings and depths are generally consistent with ADOT guidelines, with modifications based on our existing knowledge of the project corridor geotechnical support conditions. The exploration locations are shown on the Site Maps Showing Test Locations included as Figure 2.

Drilling was performed by ACS Services, LLC. (ACS) using a truck-mounted CME-75 drill rig utilizing hollow-stem auger. Sampling was performed using a standard penetration test (SPT) split spoon sampler with a 1.375-inch inner diameter (ID) and 2-inch outside diameter (OD) or an open-end drive sampler (2.42-inch-diameter brass rings) at maximum 5-foot intervals in each boring using an automatic hydraulic actuated 140-pound hammer, free falling 30 inches. 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. Unless noted otherwise on the boring logs, the sample driving resistance was recorded as number of blows per six inches of penetration. The penetration results are presented on the borings logs adjacent to each sample. Representative bulk samples of near-surface drill cuttings were also collected from selected borings.

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. Logs of the completed borings are presented in Appendix A.

The recovered soil samples were removed from the sampler, sealed to reduce moisture loss, and submitted to the ACS laboratory. Completed borings were backfilled with cuttings in accordance with the applicable permit requirements.

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 soils. A summary of the laboratory testing program is presented in Table 3.1.

Table 3.1: Laboratory Testing Program

Item/Description	Number
Grain Size Analysis (Total – Coarse and Fine) – ASTM C136 & C117	3
Atterberg Limits (Plasticity Index) - ASTM D4318	3
Soil Density and Moisture Content – ASTM D2937	3
Consolidation – ASTM D2435	1
Direct Shear – ASTM D3080	1
pH and Resistivity - AZ Method 236e	1
Sulfates and Chlorides - AZ Method 733b/736b	1

The laboratory testing program included the following tests, each followed by a brief discussion of the test relevance:

- Moisture content, sieve analysis and Atterberg limits (plasticity index) – Soil classification and moisture condition, and for computation of correlated R-values for use in pavement design.
- Density of ring samples – Determination of in-place density and moisture, used in compressibility and earthwork computations.
- One-dimensional consolidation – Determination of the magnitude and rate of consolidation of soil, used in estimating the magnitude and rate of both differential and total settlement.
- Direct shear – Estimation of shear strength parameters for foundation design.
- pH and resistivity – Determination of corrosion potential on steel pipe.
- Total soluble sulfates and chlorides – Assessment of the potential impact of the existing soils on concrete. Used to determine which type of concrete should be used.

A summary of the laboratory test results is presented in Table B-1 in Appendix B. The individual results of the laboratory testing program are also included in Appendix B. The results of moisture content and in-place dry density testing are also presented on the boring logs in Appendix A at the corresponding sample depth.

4.0 SITE CONDITIONS AND GEOTECHNICAL PROFILE

4.1 Surface Site Conditions

The study area consists of a two-lane roadway that serves as an off-ramp for eastbound (EB) SR 80 traffic onto westbound (WB) Main Street and an onramp for EB Main Street onto EB SR 80. The roadway generally slopes northwest towards downtown Bisbee. An existing retaining wall parallels the road behind the sidewalk beginning at the SR 80 underpass and terminating 160 feet before the intersection with Dart Road which leads to the Queen Mine to the south. The existing retaining wall appears to be a cast in place that is approximately 3.5 to 4 feet in height with a 1.5:1 (H:V) slope behind. The slope appears to be composed of heavily weathered gravelly material which may be residual sedimentary rock with moderate amounts of vegetation present throughout the slope. The slope appears to be in good standing with no significant signs of erosion present at its surface.

Photo 1: View of Boring B-2 Looking South

Asphaltic concrete (AC) in generally poor to fair condition was encountered at each of the boring locations. The approximate AC and aggregate base (AB) thicknesses are summarized below in Table 4.1.

Table 4.1 – Pavement Core and Subgrade Thickness Summary

Boring ID	AC (inches)	AB (inches)	Total Thickness (inches)
B-1	4.0	6.0	10.0
B-2	6.0	4.0	10.0
B-3	5.0	5.0	10.0

Notes: AC – Asphaltic Concrete Pavement; AB – Aggregate Base

4.2 Regional Geologic Setting

The project site is located within the southeastern portion of the Basin and Range Geologic Province of the southwestern United States. The Basin and Range Province is characterized by a modern landscape consisting of broad alluvial valleys interspersed with and bounded by uplifted and fault-block mountain ranges, often with well-developed pediments and alluvial fans. Generally, the mountain ranges and valleys trend in a north-south to northwest-southeast direction. The modern landscape was formed by late Tertiary (Miocene-Pliocene) extensional tectonism and high-angle normal faulting, followed by subsequent erosion of the uplifted mountains and alluvial deposition of the sediments in the newly-formed basins.

The project site is located along a fault line within the Mule Mountain Range of southeastern Arizona that is generally oriented north-south and surrounded by flat quaternary basin features.

Published geologic mapping indicated surficial geologic units at the project site to include Permian to Pennsylvanian sedimentary rocks; and early Proterozoic Metamorphic rocks (Richard et al 2000). While not encountered at the surface in our borings, these rocks are known to exist at depth (i.e. B-3) and to the south at the Queen Mine. Each surficial geological unit is listed below followed by a brief description:

- Permian to Pennsylvanian Sedimentary rocks – Interbedded sandstone, shale, and limestone usually characterized by ledgy outcrops. Orange to reddish sandstone forms cliffs near Sedona. This unit includes theNaco Group in southern Arizona. It was deposited in coastal-plain to shallow-marine settings during time of variable and changing sea level. Rocks of this map unit in southern Arizona may be in part equivalent to Permian rocks of map unit P in central and northern Arizona (AZGS 2022).

- Early Proterozoic Metamorphic rocks - Undivided metasedimentary, metavolcanic, and gneissic rocks (AZGS 2022).

4.3 Subsurface Profile

Based on the conditions encountered in the borings, subsurface conditions can be generalized as presented in Table 4.2. Refer to the boring logs in Appendix A and laboratory test results in Appendix B for additional details.

Table 4.2: Generalized Subsurface Profile

Stratum	Approximate Stratum Bottom Depth (feet)	USCS Material Types	Relative Firmness
1	16.5 (maximum depth explored)	Silty, Clayey Sand with Gravel (SC-SM) and Silty, Clayey Gravel with Sand (GC-GM)	Soft to Firm

The silty, clayey soils generally exhibited low plasticity. Greater subordinate amounts of gravel presented in shallow site soils and generally increased with depth. The relative firmness and percent fines of the subsurface soils generally decreased at depth. Boring B-3 encountered auger refusal at a depth of 5.5 feet on bedrock material.

4.4 Soil Shear Strength

Direct shear testing was performed to characterize the shear strength of the subsurface soils encountered during the field investigation. The testing was performed on a relatively undisturbed sample collected during our field investigation. A single multi-point direct shear test was performed on an in-situ sample that was inundated during testing. Based on the results of the direct shear testing, and for purposes of shear strength characterization, the subsurface soils were characterized as described in Table 4.3.

Table 4.3: Soil Groups for Shear Strength Characterization

Soil Group	General Description
SC-SM	Silty, Clayey Sand with subordinate amounts of gravel with low plasticity

Table 4.4 presents the generalized shear strength parameters for the soil outlined in Table 4.3 based on interpretation of the direct shear test results. Direct shear test worksheets with detailed individual results are included in Appendix B.

Table 4.4: Shear Strength Parameters

Soil Types	Effective Unit Weight (γ) [pcf]	Friction Angle (ϕ) [degrees]	Cohesion (c) [psf]
SC-SM	120	32	100

NOTE:
1. pcf = pounds per cubic foot, psf = pounds per square foot

4.5 Moisture Sensitive Soils

4.5.1 Collapsible Soils

In arid regions, shallow natural soils subjected to loading can quickly settle a significant amount when wetted, generally referred to as “collapse.” These collapsible soils are generally able to support structural loads with tolerable settlements under consistent dry conditions. However, once these soils are wetted, they can collapse. In order to assess the collapse potential of the soils throughout the project corridor, one (1) laboratory consolidation test was performed. The result of the test is presented in Table B-1, in Appendix B, at the end of the report. The result indicates the collapse potential in the upper 10 feet ranges below 2 percent. The categories are generally divided into three categories of severity based on Samtani and Nowatzki (2006):

- Slight - < 2%
- Moderate – 2% < and < 6%
- Moderately Severe - > 6%

The collapse potential of the in-situ soils across the project corridor should be considered of low concern.

4.6 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], 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 borings (and limited depth of investigation), a Site Class of D was used for the analysis. 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 Tables 4.5 to 4.7.

Table 4.5: Seismic Design Parameters[1]

Location	Site Class	Seismic Zone	Seismic Design Parameter	Spectral Acceleration Value [g]
SR 80	D	1	Peak Ground Acceleration (PGA)	0.077
			Short Period Acceleration (S_S)	0.179
			Long Period Acceleration (S_1)	0.051

NOTES
1. Latitude 31.440647°, Longitude -109.913
2. g = gravity

Table 4.6: Seismic Site Factors

Seismic Site Factor	Value
PGA Site Factor, F_{pga}	1.6
Short Period Acceleration Site Factor (F_a)	1.6
Long Period Acceleration Site Factor (F_v)	2.4

Using the values presented in Tables 4.6 and 4.7, points for developing the design response for the site are presented in Table 4.8.

Table 4.7: Points for Design Response Spectrum

Points	Value
A_s (equal to F_{pga} x PGA)	0.123
S_{DS} (equal to F_a x S_s)	0.286
S_{D1} (equal to F_v x S_1)	0.122

Due to the low site seismicity, the impact of seismic forces on the stability of embankments and structures is negligible.

4.7 Moisture and Groundwater Conditions

The moisture condition in the borings was generally moist throughout the boring depths. Groundwater was not encountered during the investigation. A review of the index well data available on the Arizona Department of Water Resources (ADWR) website, showed that there are no index well sites within 5 miles of the project site. Based on the scope of the project, groundwater is not anticipated to impact design and construction of the retaining wall, pavement and associated improvements.

4.8 Land Subsidence and Earth Fissures

ADWR monitors land subsidence in areas throughout Arizona using interferometric synthetic aperture radar (InSAR). The project lies within the Bowie and San Simon study area. The InSAR measured subsidence within this area between May 2010 to April 2025 was measured at approximately 7.9 to 15.7 inches (ADWR 2024). From May 2023 to April 2024, the land subsidence rate was measured using InSAR data for the Bowie and San Simon Areas at a rate of approximately 0.8 to 1.2 inches per year (ADWR, 2025).

Earth fissures typically develop at the boundaries of land subsidence areas and over areas where relatively shallow bedrock is present. The Arizona Geological Survey (AZGS) documents earth fissures through the preparation of maps showing confirmed and unconfirmed fissures. Based on a review of the AZGS Natural Hazards in Arizona web tool (AZGS, 2025), the nearest known earth fissures are located approximately 4.0 miles southwest of the site northeast of the intersection of Parker Road and W Coyote Song Road (AZGS,2025). The risk of earth fissures is currently low, due to the project area residing in the Bowie and San Simon study area subsidence area, subsidence and any potential earth fissures formation should be monitored and documented in the future. As of the time of this geotechnical investigation earth fissures should not be of concern.

5.0 ENGINEERING ANALYSES AND RECOMMENDATIONS

5.1 General

The following sections of this report present our recommendations regarding foundation design for the new retaining wall(s), construction considerations, site preparations and grading, moisture protection, excavations and backfills.

5.2 Foundation Recommendations

The foundation recommendations provided in this section are based on the AASHTO LRFD, dated 2012. Our understanding, based on discussions with the design team, is that the proposed retaining wall will be a soldier pile wall given the limited right of way. The evaluations and recommendations for the soldier pile wall are included herein.

5.2.1 Nominal Lateral Loads Acting on Retaining Walls

Walls retaining soil should be designed for the lateral earth pressure imposed by the soil. 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 below assume the backfill will be comprised of native soils. The limits of structure backfill placement are assumed to be the entire limits of excavations for the abutments and abutment wingwalls, and in all cases the structure backfill should extend a minimum of 3 feet laterally from the back edge of all walls.

Walls 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 37 psf per foot of wall height. Walls restrained from lateral movement should be designed for the at rest condition using an equivalent fluid unit weight of 56 psf per foot of wall height. Retaining walls should be designed to drain water and avoid hydrostatic pressures. These recommendations assume a horizontal backfill surface, no surcharge loadings and adequate drainage. Surcharge loads from traffic, sloped backfills, or other sources, will impose additional pressures. 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, 3.00 for passive conditions, and .470 for at-rest conditions. The effective width for passive resistance is 3b where b is the shaft diameter. These values are based on an internal friction angle of 32 degrees for the native soils.

Horizontal loads acting on foundations cast in open excavations against undisturbed native soil or properly placed and compacted fill will be resisted by friction acting along the base of the footing and by passive earth pressures against the loaded side of the footing. If the design makes use of passive earth pressure against backfill, it is important that a representative of the geotechnical engineer be present to monitor and test backfill placement and compaction in order to develop passive resistance with low strains.

If heavy mechanical compaction equipment will be operating within a distance of one-half the retained height (defined as being from the backfill grade at the back of the wall to the wall base), additional earth pressure induced by compaction should be used in wall design. The additional earth pressure should be estimated using the procedure presented by Clough and Duncan (1991). If compaction equipment used adjacent to the walls is to consist of small rollers and tampers, the additional earth pressure should not be used.

5.2.1.1 LPILE Input Parameters

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 (p)-displacement (y) curves. The p-y curves define the behavior of the soil surrounding the laterally loaded shaft. These curves are nonlinear and are developed using soil strength, depth below ground and shaft diameter amongst others.

The two deeper borings did not encounter rock at depth but rock was encountered in one shallow boring. Thus, soil parameters were utilized to be conservatism. Recommended soil input parameters for use in LPILE analyses are provided in Table 5.1. The soil input parameters were developed using the LPILE technical manual (Ensoft, 2019) and results of the geotechnical investigation.

Table 5.1: Soil Input Parameters for LPILE Analyses

Soil Layer	Depth[1] [feet]	Soil Type in LPILE	Effective Unit Weight [pcf]	Friction Angle [degrees]	Cohesion [psf]	Horizontal Subgrade Modulus, k [pci]
1	0 – 20	Silt	120	32	100	330

NOTES:
1. Zero depth refers to the existing ground surface elevation at the toe of existing retaining wall.
2. pcf = pounds per cubic foot, pci = pounds per cubic inch, psf = pounds per square foot

5.2.1.2 Soldier Pile Construction

The soils consist primarily of interbedded soft to firm fine-grained native soils with subordinate amounts of gravel transitioning to rock at depth. Soldier pile excavations may encounter caving and/or sloughing of the native soils and harder rock at depth. Temporary casing, slurry, and/or rock bits may be needed to advance soldier piles/drilled shafts. All construction techniques should be in accordance with Section 609 of the ADOT Standard Specifications (ADOT 2021) and the project-specific special provisions. Quality control during the drilled-shaft construction should include those items specifically called out in the ADOT Standard Specifications (ADOT 2021), Section 609. A quality control report should be submitted for each shaft, stating that all details were monitored and meet the project requirements.

Straight, soldier pile excavations will likely be advanced with single-flight-auger or bucket auger bits or rock bits to the recommended depth. Cleaning of the soldier pile excavations should be performed in accordance with Section 609 of the ADOT Standard Specifications (ADOT 2021). It should be verified by inspection and measurement that the excavation is open to that depth. The shaft excavation should be cleaned so no more than 2 inches of slough or loose material is present in the bottom of the excavation.

Groundwater should not impact the construction of soldier piles. Groundwater was not encountered within geotechnical borings within the project corridor.

5.3 Pavements

5.3.1 Pavement Subgrade Support

Laboratory testing of subgrade samples for support of pavements was performed as we anticipate the new pavement will be supported on either embankment fill or native soils. Based on the soils encountered in the borings, the existing subgrade conditions appear to provide generally good pavement support conditions along this corridor. The pavement analysis is submitted under separate cover. The results of laboratory grain-size analysis, Atterberg limits, and R-value (correlated and actual) testing are presented in Table 5.2.

Table 5.2: Results of Subgrade Testing

Boring ID	Depth (feet)	USCS	%Passing #200 Sieve	Plasticity Index	Correlated R-Value	Tested R-value
B-1	1-5	SC-SM	23	6	58	--
B-2	1-5	SC-SM	23	6	58	--
B-3	1-5	GC-GM	18	7	59	--
				Average	**58.3**	**--**
				Standard Deviation	**1.0**	**--**

The results of 3 correlated R-values ranged from 58 to 59 with a mean correlated R-value of 58.3 and a standard deviation of 1.0. In accordance with ADOT Pavement Design Manual (ADOT, 2017), the Rmean was calculated to be 58.3 along with a construction control R-value (Rcc) of 56.2, however, for conservatism, we recommend a design R-value of 55 be used for design of pavements. Using a seasonal variation factor of 2.6 determined for Bisbee, Arizona from Figure 2-1, a resilient modulus (Mr) value of 20,175 psi was obtained for pavement design. To reduce subgrade improvement quantities, we recommend a Rcc of 55 be used to establish subgrade acceptance criteria.

5.4 Slopes

5.4.1 Permanent Slopes

Non-stabilized embankment cut and 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 not steeper than 2:1 (H:V).

5.4.2 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 (2020). In accordance with Subpart P, Appendix A, the 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 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 will be feasible in some of the more fine-grained areas of the project, pending further analysis.

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 Surface Drainage

Long-term performance of structures and slabs 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. Engineered fill and backfill should be well compacted to reduce possible moisture infiltration through loose soil intervals.

5.6 Site Grading for Pavement

The following site grading recommendations are intended to provide support for the proposed pavements and foundations. The grading activities at the site should be performed under observation and testing directed by the Ethos geotechnical engineer.

Trash, debris, vegetation (including roots) and other organics, any existing spread fill, any unstable (soft, loose, disturbed, water softened, etc.) soils, and other deleterious materials should be removed from proposed pavement areas prior to construction. Site grading should extend laterally a minimum of 2 feet beyond pavement areas. 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.

5.6.1 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 structure areas prior to construction. This site grading should extend laterally a minimum of 2 feet beyond 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.

5.7 Corrosion Potential

Representative samples of the near-surface soils were tested for pH, laboratory minimum resistivity, soluble sulfates, and soluble chlorides to help characterize the corrosion potential to buried metal and concrete. The testing was performed in accordance with Arizona Test Methods 236c, 733b and 736b, respectively, and the results are presented in Table 5.3 and included in Appendix B.

Table 5.3: Results of Corrosion Testing

Location	Sample Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Soluble Sulfate (ppm)	Soluble Chloride (ppm)
B-2	1.0 – 5.0	7.9	1,010	--	--
B-3	1.0 – 5.0	--	--	4,320	48

Notes: ohm-cm – ohm centimeters; ppm – parts per million

The laboratory pH value of the in situ soil was of 7.9 and the minimum resistivity value measured 1.010 ohm-centimeters (ohm-cm). Pipe locations where the pH is less than 6.0, 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. 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).

Total soluble sulfate values were 4,320 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). All the samples fall within Exposure Class S2 for water-soluble sulfate (SO_4^{2-}) in soil by percent mass (0.2%<SO_4<0.4% or 1,500 to 10,000 ppm) with a value of 4,320 ppm and are categorized with a moderate severity level in terms of sulfate exposure. Based on Table 19.3.2.1, “Requirements for Concrete by Exposure Class,” in Section 19.3.2 (ACI 2019), Portland cement type should be limited to type V for concrete structures in contact with these materials.

The chlorides test result value was 48 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). The anticipated concrete exposure for this segment falls 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 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.

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.

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 borings, 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: 31.43015°, -109.90027°; May 4, 2023 Image, Airbus Imagery. Accessed on 2/6/2026.

Figure 1

Vicinity Map Showing Project Location
SR 80 Old Bisbee to Erie Street - Shared-Use Path
Bisbee, Arizona

Source: 31.44067°, -109.91271°; April 11, 2023 Image, Airbus Imagery. Accessed on 2/6/2026.

Figure 2

Site Map Showing Test Locations
SR 80 Old Bisbee to Erie Street - Shared-Use Path
Bisbee, Arizona

APPENDIX A

Boring Logs

SOIL SAMPLING & BORING LOG INFORMATION

GENERAL INFORMATION

The material and in-situ moisture descriptions of soils presented on the boring logs are based on visual observation and classification in general 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.

SAMPLING DESCRIPTION

The blow counts are recorded on the boring logs in 6-inch increments unless sampler refusal (50 blows for less than 6 inches) occurs.

Type	Symbol	Description	Type	Symbol	Description
Bulk		Drill cuttings, approximately 30 to 40 pounds.	Modified California Sampler (Ring)		3-inch OD, lined with 2.42-inch ID rings, advanced 12 inches using a 140-lb hammer with a 30-inch drop (see ASTM D3550 for more details).
Auger		Drill cuttings, approximately 5 to 10 pounds.	Shelby Tube (ST)		3-inch OD thin-walled sample. Hydraulically driven/pushed (see ASTM D1587 for more details).
Hand		Cuttings taken from a test pit or hand auger.	Rock Core		N-series (1.8-inch OD) core or H-series (2.4-inch OD) core (see ASTM D2113 for more details).
Standard Penetration Test (SPT) Sampler		2-inch OD advanced 18 inches using 140-lb hammer with a 30-inch drop (see ASTM D1586 for more details).	Dynamic Cone Penetrometer (DCP)		Solid stem cone-shaped sampler advanced 12 inches using a 15-lb mass with a 20-inch drop. Correlations from Sowers and Hedges 1966 are used to approximate N-values. (see ASTM D6951 for more details).

RELATIVE CONSISTENCY, FIRMNESS, OR DENSITY

The relative density for cohesionless, coarse-grained soils and the consistency and firmness for fine-grained soils, near saturated and unsaturated, respectively, are described on the test boring logs based on standard penetration test (SPT) blows per foot (N) (ASTM D1586) or modified California sampler ring sampler (ASTM D3550) blow counts. See Sampling Description table for details.

Cohesive Soils - Saturated or Near Saturated			Cohesive Soils – Unsaturated or Cemented Soils			Cohesionless Coarse-Grained Soils		
SPT/ N-Value (blows/ft)	Ring Sampler (blows/ft)	Consistency	SPT/ N-Value (blows/ft)	Ring Sampler (blows/ft)	Firmness	SPT/ N-Value (blows/ft)	Ring Sampler (blows/ft)	Relative Density
<2	<3	Very Soft	<4	<6	Very Soft	<4	<6	Very Loose
3-4	4-6	Soft	5-8	7-12	Soft	5-10	7-15	Loose
5-8	7-12	Medium Stiff	9-15	13-23	Medium Firm	11-30	16-45	Medium Dense
9-15	13-23	Stiff	16-30	24-45	Firm	31-50	46-75	Dense
16-30	24-45	Very Stiff	31-50	46-75	Very Firm	>50	>75	Very Dense
>30	>45	Hard	>50	>75	Hard			

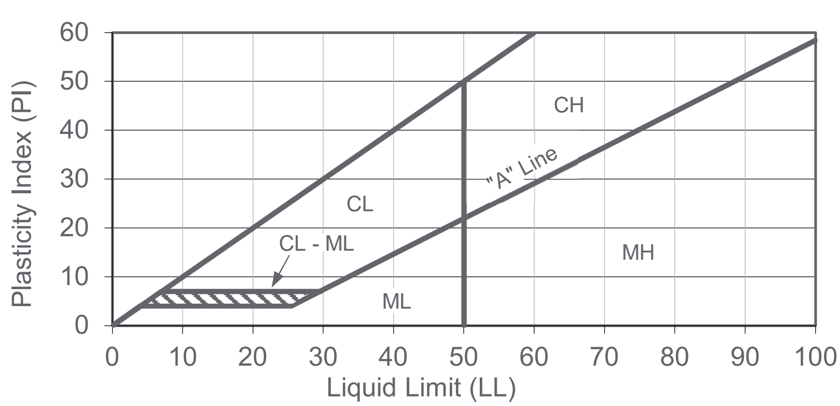
UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

The information below provides a general overview of the USCS classification system. See ASTM D2487 for more details.

Coarse-Grained Soils (more than 50% of material is larger than No. 200 sieve size)				
Criteria for Assigning Group Symbols & Names		**Graphic**	**Symbol**	**Group Description**
Gravels (more than 50% of coarse fraction larger than No. 4 sieve size)	Clean Gravels (less than 5% fines)		GW	Well-graded gravels, gravel-sand mixtures or sand-gravel-cobble (SGC) mixtures.
			GP	Poorly-graded gravels, gravel-sand mixtures or sand-gravel-cobble (SGC) mixtures.
	Gravels with more than 12% fines		GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
Sand (50% or more of coarse fraction smaller than No. 4 sieve size)	Clean Sands (less than 5% fines)		SW	Well-graded sands, gravelly sand mixtures.
			SP	Poorly-graded sands, gravelly sand mixtures.
	Sands with more than 12% fines		SM	Silty sands, sand-silt mixtures with a PI < 4 or plots below the “A” line.
			SC	Clayey sands, sand-clay mixtures.
Coarse-grained soils with between 5% and 12% passing the No. 200 sieve have a dual symbol.				

Fine-Grained Soils (50% or more of material is smaller than No. 200 sieve size)				
Silts and Clays (liquid limit < 50)	Plasticity Index > 7 and plots on or above "A" line		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
	Plasticity Index < 4 or plots below "A" line		ML	Inorganic silts, clayey silts, with low plasticity.
Silts and Clays (liquid limit $\geq$ 50)	Plasticity Index plots on or above "A" line		CH	Inorganic clays of high plasticity, silty and sandy clays of high plasticity.
	Plasticity Index plots below "A" line		MH	Inorganic silts of high plasticity, silty soils, elastic silts.
Fine-grained soils with plasticity that plots in the hatched zone on the plasticity chart have a dual symbol.				

PLASTICITY CHART



PLASTICITY

Description	Amount
Nonplastic	No Value (NV)
Low	LL < 30
Medium	30 $\leq$ LL < 50
High	50 $\leq$ LL

CEMENTATION

The table below provides a general overview of soil cementation. Cementation is evaluated using four criteria: reaction to hydrochloric acid (HCl); visual observation of calcium carbonate; finger pressure required to break down the soil; and blow counts from the sample when present. The soil must meet all four criteria to be classified as cemented.

Description	Criteria			
	HCl Reaction	**Observation of Calcium Carbonate**	**Finger Pressure**	**Blow Counts**
Weakly	Slight	Some filaments and possible nodules	Crumbles with moderate finger pressure	N >15
Moderately	Moderate to Strong	Filaments continuous throughout, nodules present, and sample is white/gray	Considerable finger pressure required to break into chunks	> 30 for last interval
Strongly	Strong	Filaments continuous and almost indistinguishable, nodules are larger, and sample is white	Will not crumble or break with firm finger pressure	Refusal

PARTICLE SIZES

Particle Name	Size (mm)	Sieve Size
Silt/Clay	<0.075	<#200
Sand (fine)	0.075 to 0.425	#200 to #40
Sand (medium)	0.425 to 2	#40 to #10
Sand (coarse)	2 to 4.75	#10 to #4
Gravel (fine)	4.75 to 19	#4 to 3/4-inch
Gravel (coarse)	19 to 75	3/4-inch to 3-inch
Cobbles	75 to 300	3-inch to 12-inch
Boulders	>300	> 12 inch

WATER LEVEL

Symbol	Description
	Water level while drilling
	Water level after drilling/static
	Perched water

Water levels on the boring logs are those measured at the time of the investigation. Groundwater levels often vary seasonally and will vary over time.

AMOUNT MODIFIERS

Description	Amount
Rare*	<5%
Trace	~10%
Some	~20%
Considerable	~30%

*Only used to describe coarse-grained fraction (i.e. sand, gravel, cobbles, or boulders) when present

ANGULARITY

Description	Visual Example
Angular	
Subangular	
Subrounded	
Rounded	

ethos ENGINEERING, LLC.

PROJECT NAME: SR 80, Old Bisbee to Erie Street - West Segment		BORING ID:	B-1
PROJECT LOCATION: Bisbee, Arizona		AGENCY PROJECT NUMBER: ADOT Project No. 080 CH BIS T0621 01C	
PROJECT NUMBER: 2025025	CLIENT NAME: Kimley-Horn & Associates	DATE STARTED: 01/15/2026	DATE COMPLETED: 01/15/2026
DRILLING METHOD(S): Hollow Stem Auger	TOOLING: 6-5/8" OD Hollow Stem Auger	HAMMER TYPE: Auto	HAMMER EFFICIENCY: 71%
SURFACE ELEVATION: 5301'	STATION: 14+10 / OFFSET AND OFFSET DIRECTION: 8 R	Main St Cst CL	
DRILLING FIRM: ACS Services, LLC	DRILLER: N. Zamora / FIELD ENGINEER: M. Meza	RIG TYPE: CME-75	RIG NUMBER: 237717
REPORTED DEPTH: 16.0'	GROUNDWATER DEPTH: N/A / REVIEWED BY: P. Garza	LATITUDE: 31.44080	LONGITUDE: -109.91318

Elevation (feet)	Depth (feet)	Soil Description and Remarks	Samples: Bulk	Samples: Driven	Blow Counts/6"	Moisture Content (%)	Dry Density (pcf)	Atterberg Limits (LL-PL-PI)	Fines (%)
5300	0.3	**ASPHALT CONCRETE**, 4 inches							
	0.8	**AGGREGATE BASE**, 6 inches							
	0.8–7	**SILTY, CLAYEY SAND WITH GRAVEL** (SC-SM), some predominantly fine angular to subangular gravel, fine to coarse subangular sand, low plasticity, light reddish brown to brown, moist, soft to moderately firm	X	X	5-7-8 (15)	10.0		23-17-6	23.0
			X	X	8-4-4 (8)				
5295	5			X	5-5	19.9	96.0		
	7–16	**SILTY, CLAYEY GRAVEL WITH SAND** (GC-GM), some fine to coarse subangular sand, predominantly fine angular to subangular gravel, low plasticity, light reddish brown to brown, moist, moderately firm to firm							
5290	10			X	5-8-10 (18)				
5285	15			X	10-10				
	16								

Auger depth of 15 feet. Sampler depth of 16 feet. Backfilled with cuttings and capped with grout dyed black.

ethos ENGINEERING, LLC.	PROJECT NAME: SR 80, Old Bisbee to Erie Street - West Segment	**BORING ID:**	**B-2**
	PROJECT LOCATION: Bisbee, Arizona	AGENCY PROJECT NUMBER: ADOT Project No. 080 CH BIS T0621 01C	
PROJECT NUMBER: 2025025	CLIENT NAME: Kimley-Horn & Associates	DATE STARTED: 01/15/2026	DATE COMPLETED: 01/15/2026
DRILLING METHOD(S): Hollow Stem Auger	TOOLING: 6-5/8" OD Hollow Stem Auger	HAMMER TYPE: Auto	HAMMER EFFICIENCY: 71%
SURFACE ELEVATION: 5303'	STATION: 15+64 / OFFSET AND OFFSET DIRECTION: 6 R	Main St Cst CL	
DRILLING FIRM: ACS Services, LLC	DRILLER: N. Zamora / FIELD ENGINEER: M. Meza	RIG TYPE: CME-75	RIG NUMBER: 237717
REPORTED DEPTH: 16.5'	GROUNDWATER DEPTH: N/A / REVIEWED BY: P. Garza	LATITUDE: 31.44053	LONGITUDE: -109.91283

Elevation (feet)	Depth (feet)	Graphic Log	Soil Description and Remarks	Samples: Bulk	Samples: Driven	Blow Counts/6"	Moisture Content (%)	Dry Density (pcf)	Atterberg Limits (LL-PL-PI)	Fines (%)
			ASPHALT CONCRETE, 6 inches — 0.5							
			AGGREGATE BASE, 4 inches — 0.8							
			SILTY, CLAYEY SAND WITH GRAVEL (SC-SM), considerable predominantly fine angular to subangular gravel, fine to coarse subangular sand, low plasticity, light reddish brown to brown, moist, moderately firm to firm	X	X	7-7-10 (17)	10.5		23-17-6	23.0
5300				X	X	15-18				
	5		Note: decreased gravel below 5 feet		X	2-2-8 (10)				
5295										
	10		Note: soft below 10 feet		X	4-7	18.2	104.6		
5290										
	15		— 16.5		X	4-4-4 (8)				

Auger depth of 15 feet. Sampler depth of 16 feet, 6 inches.
Backfilled with cuttings and capped with grout dyed black.

ethos ENGINEERING, LLC.	PROJECT NAME: SR 80, Old Bisbee to Erie Street - West Segment		**BORING ID:**	**B-3**
	PROJECT LOCATION: Bisbee, Arizona		AGENCY PROJECT NUMBER: ADOT Project No. 080 CH BIS T0621 01C	
PROJECT NUMBER: 2025025	CLIENT NAME: Kimley-Horn & Associates		DATE STARTED: 01/15/2026	DATE COMPLETED: 01/15/2026
DRILLING METHOD(S): Hollow Stem Auger	TOOLING: 6-5/8" OD Hollow Stem Auger		HAMMER TYPE: Auto	HAMMER EFFICIENCY: 71%
SURFACE ELEVATION: 5306'	STATION: 17+10	OFFSET AND OFFSET DIRECTION: 14 R	Main St Cst CL	
DRILLING FIRM: ACS Services, LLC	DRILLER: N. Zamora	FIELD ENGINEER: M. Meza	RIG TYPE: CME-75	RIG NUMBER: 237717
REPORTED DEPTH: 6.0'	GROUNDWATER DEPTH: N/A	REVIEWED BY: P. Garza	LATITUDE: 31.44031	LONGITUDE: -109.91243

Elevation (feet)	Depth (feet)	Graphic Log	Soil Description and Remarks	Samples: Bulk	Samples: Driven	Samples: Blow Counts/6"	Laboratory Results: Moisture Content (%)	Laboratory Results: Dry Density (pcf)	Laboratory Results: Atterberg Limits (LL-PL-PI)	Laboratory Results: Fines (%)
			ASPHALT CONCRETE, 5 inches 0.4							
5305			**AGGREGATE BASE**, 5 inches 0.8							
			SILTY, CLAYEY GRAVEL WITH SAND (GC-GM), considerable fine to coarse subangular sand, predominantly fine angular to subangular gravel, low plasticity, light reddish brown to brown, moist, firm			15-10-8 (18)	11.7		27-20-7	18.0
						18-16-8 (24)				
	5		Note: hard below 5 feet			50/6"	17.9	81.6		
5300			6							

Sampler refusal at 5 feet, 6 inches. Auger refusal on bedrock at 6 feet. Backfilled with cuttings and capped with 12 inches of grout dyed black.

APPENDIX B

Laboratory Test Results

TABLE B-1: SUMMARY OF LABORATORY TEST RESULTS

Boring Number	Sample 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)	Consolidation Performed (ASTM D2435)	Collapse [%] (ASTM D2435)	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	SC-SM	23	23	6	10.0								
B-1	5.0	6.0					19.9	96.0							
B-2	1.0	5.0	SC-SM	23	23	6	10.5					7.9	1,010		
B-2	3.0	4.0									X				
B-2	10.0	11.0					18.2	104.6	X	0.7					
B-3	1.0	5.0	GC-GM	18	27	7	11.7							4,320	48
B-3	5.0	5.5					17.9	81.6							
Average				**21.3**	**24.3**	**6.3**	**14.7**	**94.1**	**--**	**0.7**	**--**	**7.9**	**1,010**	**4,320**	**48**
Standard Deviation				**2.9**	**2.3**	**0.6**	**4.4**	**11.6**	**--**	**--**	**--**	**--**	**--**	**--**	**--**
Maximum				**23.0**	**27**	**7**	**19.9**	**104.6**	**--**	**0.7**	**--**	**7.9**	**1,010**	**4,320**	**48**
Minimum				**18.0**	**23**	**6**	**10.0**	**81.6**	**--**	**0.7**	**--**	**7.9**	**1,010**	**4,320**	**48**
Count				**3**	**3**	**3**	**6**	**3**	**1**	**1**	**1**	**1**	**1**	**1**	**1**

Notes: pcf = pounds per cubic foot; ohm-cm = ohm-centimeters; ppm = parts per million
[1] Values do not include rock correction.

SIEVE ANALYSIS AND ATTERBERG LIMITS (PLASTICITY INDEX)

ASTM D2487 & ASTM D4318

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2501555	Material Type:	Soils
ACS Lab #	26-1146-1	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	1/16/2026
Project Name:	SR 80 Shared Use Path - West Segment	Sampled By:	Client
Project Address:	478 N. Dart Rd	Test Date:	1/19/2026
Project City	Bisbee	Tested By:	Colin Eggebrecht
Sample Location:	B-1 @ 1 - 5	Reviewed By:	Keagen Mayfield

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"	2	98	
1/2"	6	93	
3/8"	4	88	
1/4"	7	81	
#4	4	77	
#8	13	63	
#10	3	60	
#16	8	52	
#30	8	44	
#40	4	40	
#50	4	36	
#100	7	28	
#200	5	23	

Liquid Limit (ASTM D4318)	23
Plastic Limit (ASTM D4318)	17
Plasticity Index (ASTM D4318)	6
Moisture Content (ASTM D2216)	10.0

USCS Soil Classification	SC-SM

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

Keagen Mayfield

Laboratory Manager

Keagen Mayfield

Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2501555	Material Type:	Soils
ACS Lab #	26-1146-3	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	1/16/2026
Project Name:	SR 80 Shared Use Path - West Segment	Sampled By:	Client
Project Address:	478 N. Dart Rd	Test Date:	1/19/2026
Project City	Bisbee	Tested By:	Colin Eggebrecht
Sample Location:	B-2 @ 1 - 5	Reviewed By:	Keagen Mayfield

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"	1	95	
1/2"	8	87	
3/8"	6	81	
1/4"	12	68	
#4	7	62	
#8	11	51	
#10	2	48	
#16	6	42	
#30	5	37	
#40	2	35	
#50	2	32	
#100	5	27	
#200	4	23	

Liquid Limit (ASTM D4318)	23
Plastic Limit (ASTM D4318)	17
Plasticity Index (ASTM D4318)	6
Moisture Content (ASTM D2216)	10.5
USCS Soil Classification	SC-SM

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

Keagen Mayfield
Laboratory Manager

Keagen Mayfield
Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2501555	Material Type:	Soils
ACS Lab #	26-1146-6	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	1/16/2026
Project Name:	SR 80 Shared Use Path - West Segment	Sampled By:	Client
Project Address:	478 N. Dart Rd	Test Date:	1/19/2026
Project City	Bisbee	Tested By:	Colin Eggebrecht
Sample Location:	B-3 @ 1 - 5	Reviewed By:	Keagen Mayfield

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"	6	94	
1/2"	16	78	
3/8"	9	70	
1/4"	13	56	
#4	6	50	
#8	10	40	
#10	2	38	
#16	5	33	
#30	5	28	
#40	2	26	
#50	2	25	
#100	4	21	
#200	3	18	

Liquid Limit (ASTM D4318)	27
Plastic Limit (ASTM D4318)	20
Plasticity Index (ASTM D4318)	7
Moisture Content (ASTM D2216)	11.7

USCS Soil Classification	GC-GM

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

Keagen Mayfield
Laboratory Manager

Keagen Mayfield
Signature

IN-PLACE DRY DENSITY (DENSITY OF RINGS)

ASTM 2937

ACS Services LLC

Job #	2501555	Material Type:	Soils
Lab #	26-1146	Extraction Date:	1/12/2026
Client:	Ethos Engineering, LLC	Extracted By:	Client
Project Name:	SR 80 Shared Use Path - West Segment	Laboratory Test Date	1/19/2026
Project Address:	478 N. Dart Rd	Laboratory Tested By:	Colin Eggebrecht
Project City:	Bisbee	Reviewed By:	Keagen Mayfield
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)**
26-1146-2	B-1 @ 5 - 6	556.1	463.7	19.9%	4	731.4	175.3	96.0
26-1146-5	B-2 @ 10 - 11	298.5	252.6	18.2%	2	384.3	85.8	104.6
26-1146-7	B-3 @ 5-5.5	348.4	295.6	17.9%	3	478.9	130.5	81.6

DIRECT SHEAR

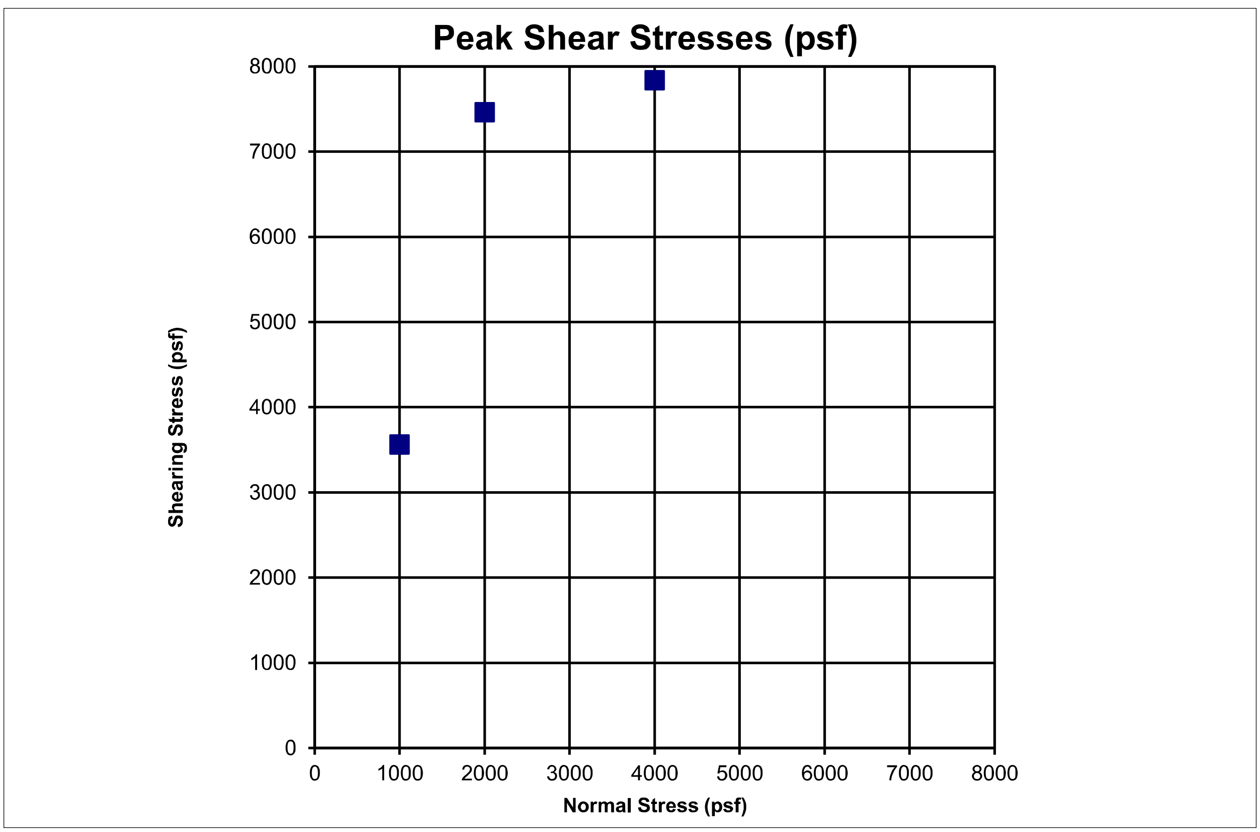
ASTM D3080

PROJECT:	ACS Services, LLC
LOCATION:	Project #2501555 Lab ID-26-1146-4
MATERIAL:	See boring logs
SAMPLE SOURCE:	B-2 @ 3-4'
SAMPLE PREPARATION:	Sat

JOB NO:	US-EI-1720234239
WORK ORDER NO:	146
LAB NO:	26-0077
DATE ASSIGNED:	1/21/2026

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.):	0.997	0.996	0.993
Shearing device used:	Humboldt Automated Shear Test System by Trautwein Soil Testing Equipment		
Rate of deformation (in/min):	0.002	0.002	0.002
Direct shear point:	1	2	3
Dry mass of specimen (g):	128.8	129.1	136.2
Initial Moisture Content:	17.0%	19.6%	20.5%
Initial Wet Density (pcf):	124.8	127.9	135.9
Initial Dry Density (pcf):	106.7	106.9	112.8
Final Moisture Content:	22.8%	24.3%	23.5%
Final Wet Density (pcf):	131.4	133.5	140.3
Final Dry Density (pcf):	107.0	107.4	113.6
Normal Stress (psf):	1000	2000	4000
Maximum Shearing Stress (psf):	3564	7464	7840
Vertical Deformation @ Max Shear (in):	0.271	0.320	0.316
Horizontal Deformation @ Max Shear (in):	0.498	0.500	0.500



WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

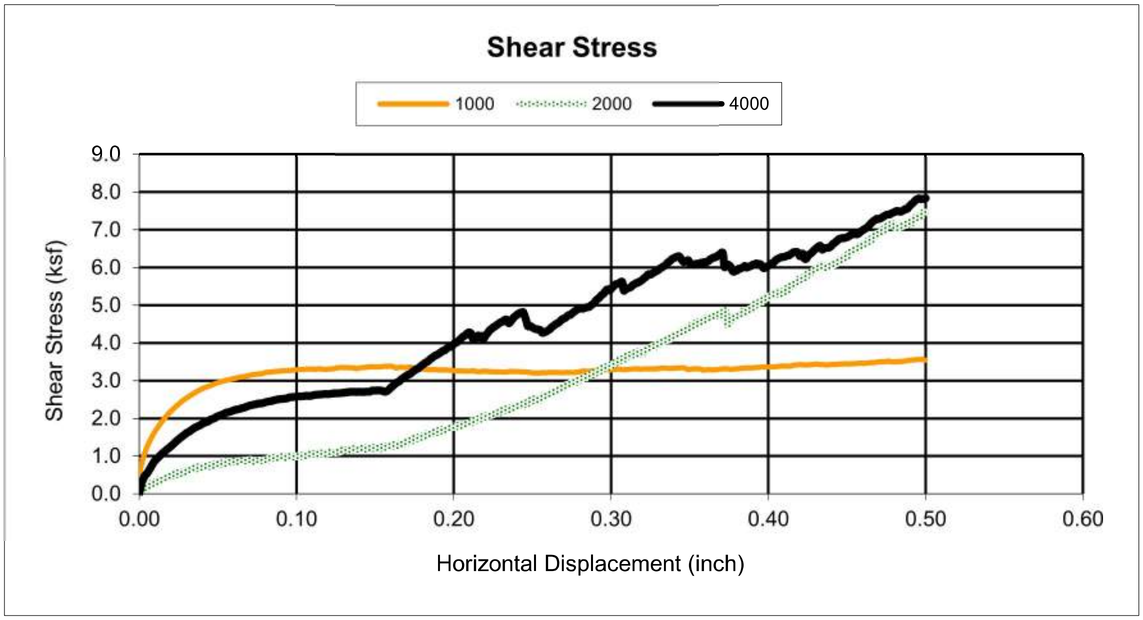
REVIEWED BY 01/27/26

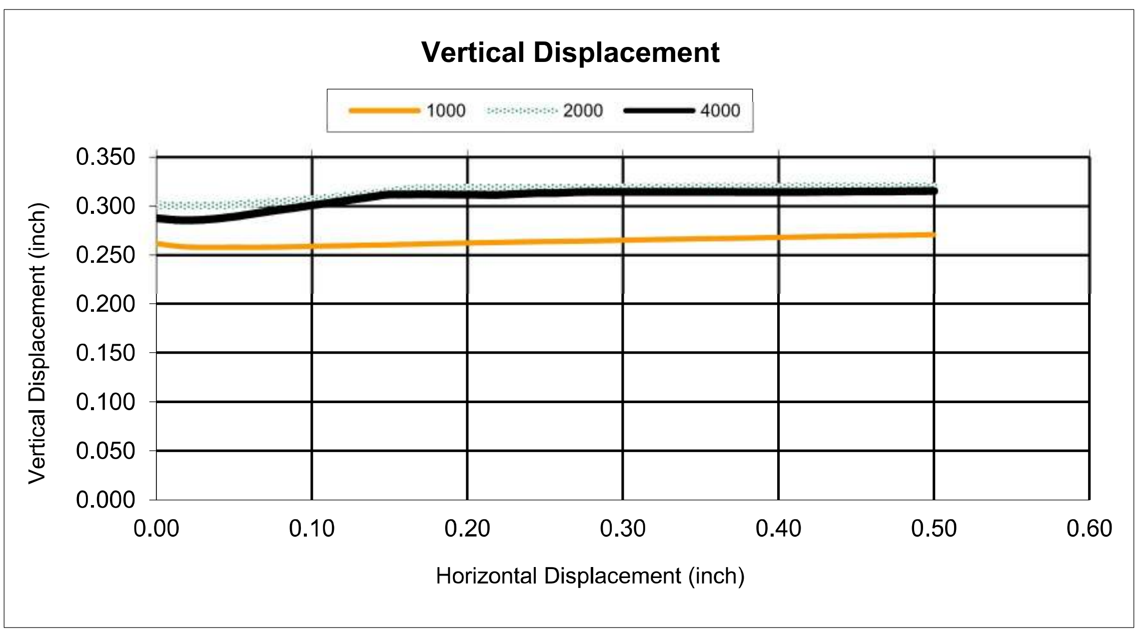
PROJECT: ACS Services, LLC
LOCATION: Project #2501555 Lab ID-26-1146-4
MATERIAL: See boring logs
SAMPLE SOURCE: B-2 @ 3-4'
SAMPLE PREPARATION: Sat

JOB NO: US-EI-1720234239
WORK ORDER NO: 146
LAB NO: 26-0077
DATE ASSIGNED: 1/21/2026

NORMAL LOADS (psf): 1000 2000 4000

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





WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

ONE-DIMENSIONAL CONSOLIDATION

ASTM D2435

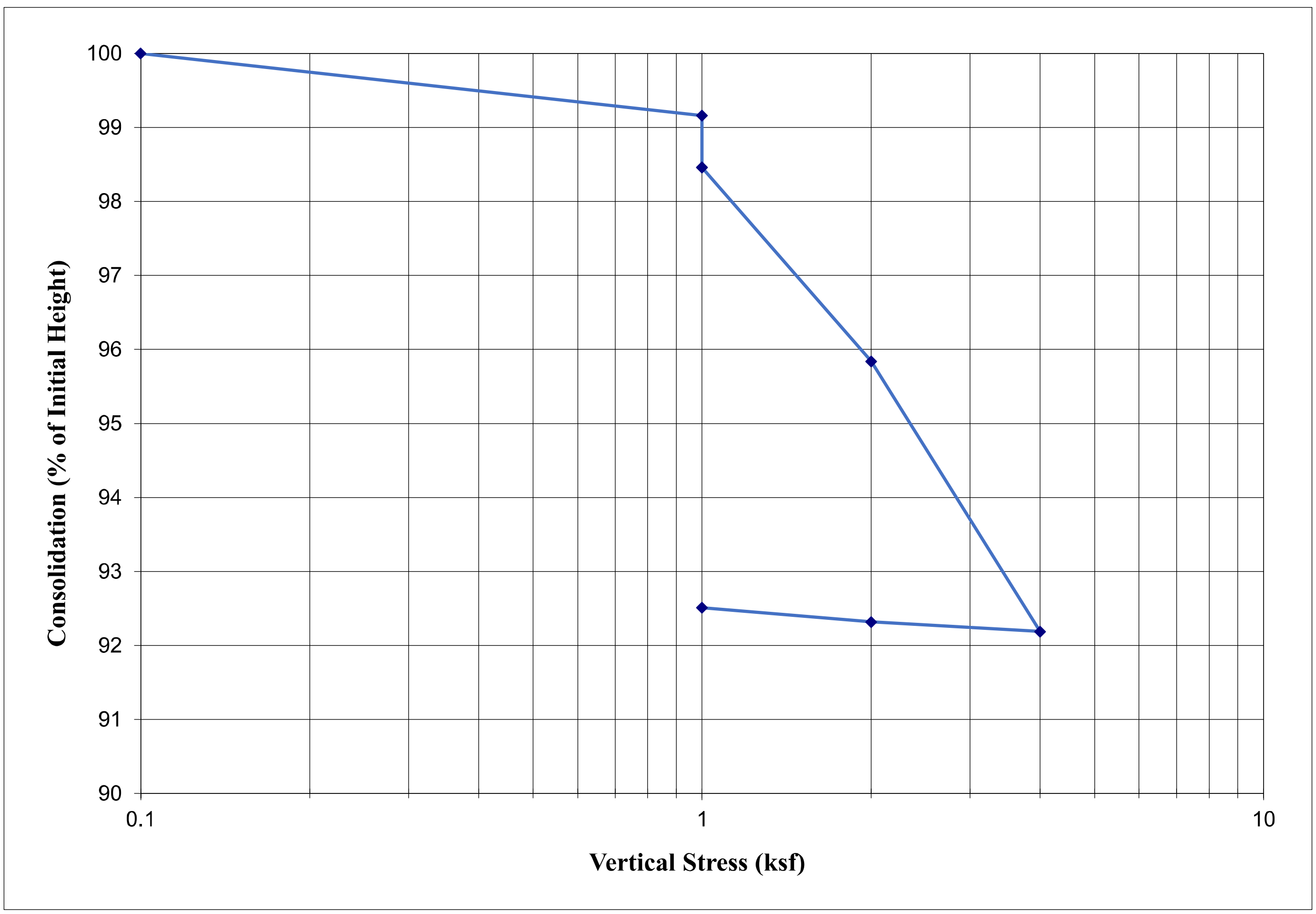
ACS Services LLC

ENGINEERING DESIGN • MATERIAL TESTING • CONSTRUCTION INSPECTION

* ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D2435)

ACS Project No.:	**2501555**		
Lab No.:	**26-1146-5**	Material Type:	Soils
Client:	Ethos Engineering, LLC	Date of Extraction:	1/16/2026
Project Name:	SR 80 Shared Use Path - West Segr	Extracted By:	Client
Project Address:	478 N. Dart Rd	Date of Lab Test:	1/20/2026
Project City:	Bisbee	Lab Tested By:	Colin Eggebrecht
Sample Location:	B-2 @ 10 - 11	Reviewed By:	Keagen Mayfield

INITIAL VOLUME (cu.in)	4.60	FINAL VOLUME (cu.in)	4.24
INITIAL MOISTURE CONTENT	16.7%	FINAL MOISTURE CONTENT	17.8%
INITIAL DRY DENSITY(pcf)	105.1	FINAL DRY DENSITY(pcf)	113.9
INITIAL DEGREE OF SATURATION	75%	FINAL DEGREE OF SATURATION	100%
INITIAL VOID RATIO	0.6	FINAL VOID RATIO	0.5
ESTIMATED SPECIFIC GRAVITY	2.70	SATURATED AT	1 ksf



PH & RESISTIVITY

ASTM AZ 236

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

Project #	2501555	Material Type:	Soils
Lab #	26-1146-3	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	1/16/2026
Project Name:	;R 80 Shared Use Path - West Segmer	Sampled By:	Client
Project Address:	478 N. Dart Rd	Test Date:	Thursday, January 22, 2026
Project City:	Bisbee	Tested By:	Austin Archibald
Sample Source:	B-2 @ 1 - 5	Resistivity Box:	
		Reviewed By:	Keagen Mayfield

pH Reading = 7.92

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	6.74	4	100	2700
50	6.74	3.1	100	2090
50	6.74	2.6	100	1750
50	6.74	2.4	100	1620
50	6.74	2.4	100	1620
50	6.74	2	100	1350
50	6.74	1.9	100	1280
50	6.74	1.7	100	1150
50	6.74	1.7	100	1150
50	6.74	1.5	100	1010
50	6.74	1.6	100	1080

Colin Eggebrecht
Lab Supervisor

Keagen Mayfield
Laboratory Manager

SULFATES & CHLORIDES

AZ 733 & AZ736

MOTZZ LABORATORY, INC.

Report: 959587
Reported: 1/25/2026
Received: 1/21/2026
PO: 2501555

Laboratory Analysis Report

ACS Services LLC
Keagen Mayfield
2235 W Broadway Road
Mesa, AZ 85202

Project: 2501555

Lab Number	Sample ID	
959587-1	26-1146-6	B-3 (1-5')

Test Parameter

Test	*Method*	*Result*	*Units*
Chloride	ARIZ 736b	48	ppm
Sulfate	ARIZ 733b	4320	ppm