GEOTECHNICAL EXPLORATION REPORT – REVISION 1
STADIUM TRAIL, SKUNK CREEK TO 75TH AVENUE – PHASE II
ADOT CONTRACT NO. 2018-006
ADOT PROJECT NO. 0000 MA PEO T0321 01C
FEDERAL AID NO. PEO-0(229)T
PEORIA, ARIZONA

Prepared for:

Jacobs

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Ethos Project No. 2022053
March 4, 2024

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Date Signed 3/4/2024
ARIZONA U.S.A.

March 4, 2024
Ethos Project No.: 2022053

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SUBJECT: Geotechnical Exploration Report – Revision 1
Stadium Trail, Skunk Creek to 75th Avenue – Phase II
ADOT Contract No. 2018-006
ADOT Project No. 0000 MA PEO T0321 01C
Federal Aid No. PEO-0(229)T
Peoria, Arizona

Dear Gary,

Ethos Engineering, LLC is pleased to present the findings of the geotechnical exploration for the proposed multi-use path, pedestrian bridge crossing over Skunk Creek, and an undercrossing of the existing 75th Avenue bridge to connect the existing Stadium Trail along the Arizona Canal Diversion Channel in Peoria, Arizona. Our services were conducted in general accordance with the scope of services presented in our proposal, dated July 28, 2022. This report provides the results of our investigation for foundation support for the proposed new single-span bridge and retaining wall. Also included are recommendations for subgrade preparation, 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

Magdaleno Meza, E.I.T.
Geotechnical Designer

Reviewed By:

Registered Professional Engineer (Civil)
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Francisco J. Garza, P.E.
Principal/Senior Geotechnical Engineer

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1.0 PROJECT DESCRIPTION

The project consists of the construction of a 0.15-mile multi-use path, pedestrian bridge over Skunk Creek, and undercrossing of the existing 75th Avenue bridge to connect the existing Stadium Trail along the Arizona Canal Diversion Channel (ACDC) in Peoria, Arizona. The project includes a 12-foot-wide concrete path with shoulders, landscaping, and lighting.

The pedestrian bridge will improve access to the Peoria Sports Complex located near 83rd Avenue and Paradise Lane . The preferred alternative is a 142-foot-long single span. We anticipate the bridge will be supported on drilled shafts. A small (up to 5 feet) in height retaining wall may be necessary for the undercrossing at 75th Avenue. The Arizona Department of Transportation (ADOT) has selected Jacobs Engineering Group (Jacobs) to complete final design for the project.

The exploration included site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. The purpose of this report is to provide information regarding the subsurface soil conditions based on the results of our field and laboratory testing and to provide geotechnical recommendations for design and construction of the planned improvements.

2.0 FIELD EXPLORATION

Prior to our field exploration, Ethos obtained a City of Peoria permit (No. E221302), dated October 18, 2022, and Flood Control District of Maricopa County permit (No. 2022P286) dated November 11, 2022. Upon receipt of permits, Ethos marked the boring locations and coordinated clearing our work areas with Arizona 811.

Drilling of the exploratory borings was performed by Geomechanics Southwest, Inc. (GSI) from November 16 to 18, 2022. The field work was supervised by Magdaleno Meza, E.I.T, of Ethos. The subsurface soil conditions at the site were explored by drilling a total of 3 borings (designated as B-1 through B-3) to approximate depths of 21.5 to 81 feet below existing site grades. A summary of the field exploration program is presented in Table 2.1. The boring locations are shown on Figure 2.

Table 2.1 – Field Exploration Program

Location ID	Project Element	Drill Method	Depth (feet)
B-1	Bridge Abutment	Tubex	81
B-2	Bridge Abutment	Tubex	81
B-3	Pathway / Retaining Wall	Tubex	21.5

The borings were drilled with a truck-mounted CME 75 drill-rig advancing 5.5-inch outside-diameter (OD) casing and a 4.5-inch downhole percussion hammer (Tubex). During the field exploration, the soils encountered were visually classified, logged, and sampled by the field engineer.

Relatively undisturbed samples of the subsurface materials were obtained using a ring sampler with a 2.42-inch inside diameter (ID) and 3-inch OD. Though numerous ring samples were attempted, recovery was limited due to the sandy and gravel nature of the subsurface materials. Disturbed samples of soils were obtained using a standard penetration test (SPT) split spoon sampler with a 1.375-inch ID and 2-inch OD. Bulk samples of drill cuttings were also collected at selected near-surface depths from the borings. 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 logs, 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) laboratory. The borings were backfilled with bentonite slurry in accordance with permit requirements. The boring logs are included in Appendix A.

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	5	Soil Classification
Atterberg Limits (ASTM D4318)	Bulk/SPT	5	Soil Classification
Moisture (ASTM D2216)	Bulk/SPT	6	Moisture Conditions
In-Situ Density (ASTM D2937)	Ring	4	Soil Density Conditions
Consolidation (ASTM D2435)	Ring	2	Swell/Consolidation Potential
Direct Shear (ASTM D3080-1)	Ring	2	Friction Angle and Cohesion
Proctor (ASTM D698)	Bulk	1	Compaction Characteristics
Expansion (ASTM D4546)	Bulk	1	Expansion Potential
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 pedestrian bridge will cross Skunk Creek at a point 400 feet south of the existing Paradise Lane traffic bridge crossing at Skunk Creek and just north of the confluence with the ACDC. This section of Skunk Creek is a trapezoidal channel with a surface of grouted stone. The channel bottom is relatively flat with an overall drainage pattern to the south with the ACDC. According to as-built drawings, the side slopes are approximately 2:1 (horizontal:vertical) with the adjacent natural ground surface approximately 20 feet higher than the channel bottom. The adjacent ground surface is relatively flat both west and east of the proposed bridge, sloping slightly towards the creek. The area west of the bridge consists of a landscaped area adjacent to a multi-family development. The area east of the bridge is mainly undeveloped and sparsely covered with native vegetation and an asphalt-paved pedestrian pathway.

4.2 REGIONAL AND LOCAL GEOLOGY

The project site is in 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 deposition of the sediments in the newly-formed basins.

Locally, Skunk Creek is part of a larger alluvial fan complex south of the Hedgpeth Hills into Deer Valley/northern Glendale. The eastern margin of the Skunk Creek deposits merge into alluvial fan deposits of Cave Creek, and farther south the western margin of Skunk Creek deposits abut New River deposits. Limited exposures of deposits in northernmost Deer Valley suggest that deposits contain abundant gravel AZGS 2016).

4.3 SITE SUBSURFACE CONDITIONS

Based on the results of the field investigation, the subgrade soils generally consist of coarse-grained non-plastic sand, gravel and cobbles (GP, GP-GM, GM, and GC) with isolated lenses of medium plasticity clayey sand (SC) and sandy clay (CL). The near-surface soils were noted to have low potential for expansion with a laboratory-tested swell value of 1.1 percent.

The relative consistency based on blow counts was generally very dense throughout the boring depths but included intermittent dense to soft zones. Refer to Appendix A for details about the conditions encountered in the borings.

Groundwater was not encountered to the depths explored. Based on index well data available on the Arizona Department of Water Resources (ADWR) website, the depth to regional groundwater was measured at approximate depth of 380 feet in December 2008 (ADWR 2022). Based on the conditions encountered in the borings, the impact to construction from groundwater appears to be negligible. However, wet ground conditions could occur due to flows within Skunk Creek/ACDC and should be considered during construction planning.

4.4 SITE SEISMICITY

The project site is in south-central Arizona which is an area of low seismic activity. Based on the conditions encountered in the borings limited by depth, it is recommended that a Site Class D be utilized for seismic design. In accordance with AASHTO (2012) 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 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 33.63319° N, Longitude 112.22223° W		
Site Class Definition	D	Table 3.10.3.1-1
Site Coefficient, F_{PGA}	1.6	Table 3.10.3.2-1
Site Coefficient, F_a	1.6	Table 3.10.3.2-2
Site Coefficient, F_v	2.4	Table 3.10.3.2-3
PGA	0.053g	
Spectral Acceleration, S_{DS}	0.194g	Equation 3.10.4.2-3
Spectral Acceleration, S_{D1}	0.096g	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 and retaining wall, site preparations and grading, moisture protection, excavations, and other construction considerations. 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 exploratory bridge borings (B-1 and B-2) and retaining wall boring (B-3).

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 very dense materials present at depth. Based on input from the hydraulic designer, there is no scour anticipated at the abutments. Alternatively, a spread footing could be considered, but was not evaluated by the design team.

In general, drilled shafts, which derive their support from both side shear and tip will provide adequate support of the abutments with limited 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 was determined using the AASHTO LRFD Bridge Design Specifications (AASHTO 2012) using both tip and side resistance. The drilled shaft foundations were designed using the Beta method as outlined for cohesionless soils based on the subsurface profile encountered in the borings. For the beta method analysis, refusal blow counts were limited to 50 in cohesionless soils (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. 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 approximately 5 feet below the existing site grades. A minimum shaft penetration of 20 feet below the top of shaft (i.e., 25 feet below existing site grades) is also recommended.

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
	3D or more	---	1.0
Single and Multiple Rows	*2D* 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

5.2.1.2 Downdrag

Our understanding is that approach embankment fills 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, downdrag loads need not 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	Below 1,205	Sand	115	32	225

NOTES:
When the ground in front of the drilled shaft is sloping, the lateral shaft resistance should 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

5.2.1.3.2. Group Effects - Lateral

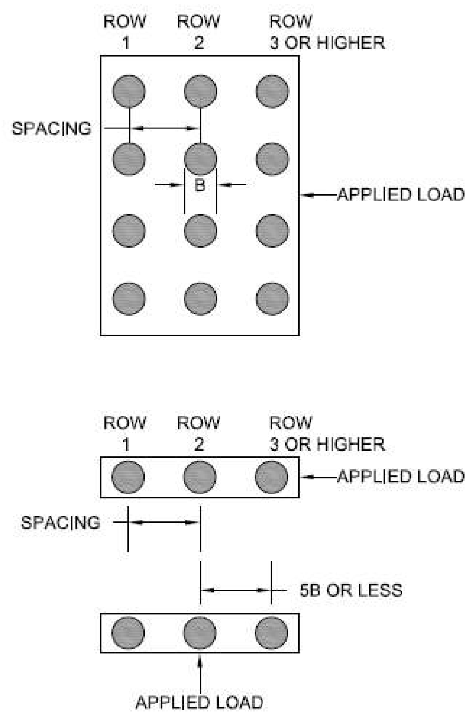
The design of laterally loaded drilled shafts must account for the influence from adjacent shafts in a group. Article 10.7.2.4 (AASHTO, 2012) defines a drilled-shaft group with respect to lateral loading as drilled shafts spaced less than five diameters center-to-center (CTC) in the direction parallel 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 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 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.

While groundwater is not 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.

5.2.2 Wall Spread Footings

5.2.2.1 Bearing Resistance

The strength and service limit state design analyses for spread footings were completed per the methods presented in Sections 10.5 and 10.6, respectively, of AASHTO (2012), and ADOT Geotechnical Design Policy SF-1 (2010a). Based on current design information provided by Jacobs, the 75th Avenue undercrossing retaining wall will likely be founded on spread footings. The following recommendations can be applied to all sections of retaining wall assuming the provisions in Section 5.2.2.5 are followed.

The factored net bearing resistance, q_{Rn}, for the strength limit state design was determined using the net nominal bearing resistance (ultimate bearing capacity), q_{nn}, calculated per Section 10.6.3.1.2a and bearing resistance factor, φ_b, from Section 10.5.5.2.2 of AASHTO (2012). The parameters presented below in Table 5.4 were assumed for the nominal resistance and strength limit state analyses.

The footing length and depth were assumed based on information provided by the project team. The resulting factored net bearing resistance, q_{Rn}, versus effective footing width, B', is shown as the "Strength Limit State" line in Figure D1.

Table 5.4
Spread Footing Analysis Parameters - Strength Limit State Design for Bearing

Parameter	Symbol	Value
Soil Angle of Internal Friction	ϕ_f	32 degrees
Soil Total Unit Weight	γ	115 pcf
Cohesion	c	0 psf
Maximum Footing Length	L	50 ft
Footing Bearing Depth	D_f	3.0 ft
Effective Footing Width	B_f	2 to 10 ft
Bearing Resistance Factor	φ_b	0.45

Per the ADOT Geotechnical Design Policy SF-1 (2010a), the modified Schmertmann method presented in Section 8.5 of the Federal Highway Administration (FHWA 2006) Soils and Foundation Reference Manual was used to calculate settlements for the service limit state analysis. The parameters assumed for this analysis are presented in Table 5.5.

Table 5.5
Spread Footing Analysis Parameters - Service Limit State Design for Bearing

Parameter	Symbol	Depth Interval (ft)[1]	
		0-10	Below 10
Soil Type	--	Sand	Sand
Soil Unit Weight (pcf)	γ	115	115
Corrected SPT N-value	N_{60}	22 to 48	50+
Elastic Modulus (ksf)	E_s	$5N_{60}$	$5N_{60}$

(1) Depth of 0 assumed to be at base of footing.

The parameters are based on the measured soil densities, distribution of N values and on the E_s-N correlations from FHWA (2006). Figure D1 presents the family of service limit state curves developed per ADOT Geotechnical Design Policy SF-1 (2010a) for design settlements of 0.25, 0.5, 0.75, 1.0, 1.5 and 2.0 inches and effective footing widths, B_f, ranging from 2 to 10 feet.

5.2.2.2 Sliding

The factored sliding resistance, R_R, for limit state design should be determined using the nominal sliding resistance between soil and foundation, R_τ, and nominal passive resistance, R_{ep}, per Section 10.6.3.4, and corresponding resistance factors, φ_τ and φ_{ep}, from Section 10.5.5.2.2 of AASHTO LRFD (2012). We recommend the parameters presented in Table 5.6 be used for analyzing sliding resistance.

Passive lateral soil resistance should typically be neglected in the upper 3 feet of finished grade due to the potential for disturbance. Below a depth of 3 feet, the nominal passive resistance can be estimated assuming a hydrostatic pressure distribution of 300 psf per foot.

Table 5.6
Spread Footing Analysis Parameters - Strength Limit State Design for Sliding

Parameter	Symbol	Value
Factored Sliding Resistance		
Resistance Factor for Shear Between Soil and Foundation	φ_τ	0.90[1]
Resistance Factor for Passive Resistance	φ_{ep}	0.50
Nominal Sliding Resistance		
Soil Angle of Internal Friction	ϕ_f	32 degrees
Soil Total Unit Weight	γ	115 pcf
Cohesion	c	0
Shear Resistance Between Soil and Foundation	δ	32 deg = ϕ_f
Passive Earth Pressure Coefficient	K_p	3.25

(1) Use resistance factor of 0.90 for soil-on-soil interface for the bottom horizontal plane of footing between toe and front of key. For remainder of footing bottom use values provided in Table 10.5.5.2.2-1 (AASHTO, 2012).

5.2.2.3 Eccentricity

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

5.2.2.4 Nominal Lateral Loads Acting on Retaining Walls

Walls retaining soils should be designed for the lateral earth pressure imposed by the soils. The magnitude of the lateral earth pressure is a function of the backfill material, imposed surcharge loads, drainage accommodations and the rigidity of the retaining structure. The recommended lateral earth pressure values presented below assume the backfill will be structure backfill comprised of granular soils which meet the requirements of Section 203 of the current ADOT Standard Specifications. 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 which are free to deflect a minimum of 0.1 percent of the wall height should be designed for the full active earth pressure condition and an active equivalent fluid unit weight on the order of 35 psf per foot of wall height. Walls which are restrained from lateral movement should be designed for the at rest condition using an equivalent fluid unit weight of 55 psf per foot of wall height. 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.

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 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 to develop passive resistance with low strains.

5.2.2.5 Foundation Subgrade Preparation

Details of the foundation elements for the retaining wall are unknown at this time, but it is assumed the retaining wall will be a standard cast-in-place cantilever constructed on native site soils at an approximate depth of 3 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.

5.2.2.6 Structure Backfill

All wall backfill placed for this project should consist of structure backfill meeting the requirements of Section 203 of the current ADOT Standard Specifications. 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.

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 not steeper than 2:1 (H:V). Permanent cut slopes, where required, should be no steeper than 3H: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 (2020). 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. The perimeter of all excavations should be protected against water runoff and infiltration near the edges to maintain stability.

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 PRELIMINARY CORROSION OR DEGRADATION POTENTIAL

5.5.1 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 two samples ranged from 8.3 to 9.5. The resistivity value of the tested samples ranged from 704 to 2,011 ohm-centimeters (ohm-cm). Pipe locations where the pH is greater than 9.0 and/or the resistivity is less than 2,000 ohm-cm require the use of special pipes and/or pipe coatings (ADOT 1996). The samples tested had a pH greater than 9.0 or resistivity values less than 2,000 ohm-cm. Therefore, it is recommended that specialized piping is utilized for metallic pipes.

5.5.2 Concrete in Contact with Soil

Two samples from the current investigation were used for soluble sulfates and chlorides (Arizona Test Method 733 and Arizona Test Method 736) to support the design of concrete structures.

The total soluble sulfate values ranged from 3 to 4 parts per million (ppm). The sulfate test measures the water-leachable or "available" sulfate content. These results were compared to Table 19.3.1.1, "Exposure Categories and Classes," in Section 19.3.1 of the American Concrete Institute's (ACI's) Building Code Requirements for Structural Concrete (ACI, 2019). The samples fall within Exposure Class S0 for water-soluble sulfate (SO_42-) in soil by percent mass (SO_4<0.1% or 1,000 ppm) and are categorized with a severity level of "not applicable" in terms of sulfate exposure. Based on Table 19.3.2.1, "Requirements for Concrete by Exposure Class," in Section 19.3.2 of ACI 318-19, there is no restriction on Portland cement type for concrete structures in contact with these materials.

The chloride test values ranged from 19 to 168 ppm. Regarding chloride attack, Section 19.3.2 of 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 of ACI (2019). The anticipated concrete exposure for this segment falls within Exposure Class C1. Table 19.3.2.1 of 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.

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.

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.

7.0 REFERENCES

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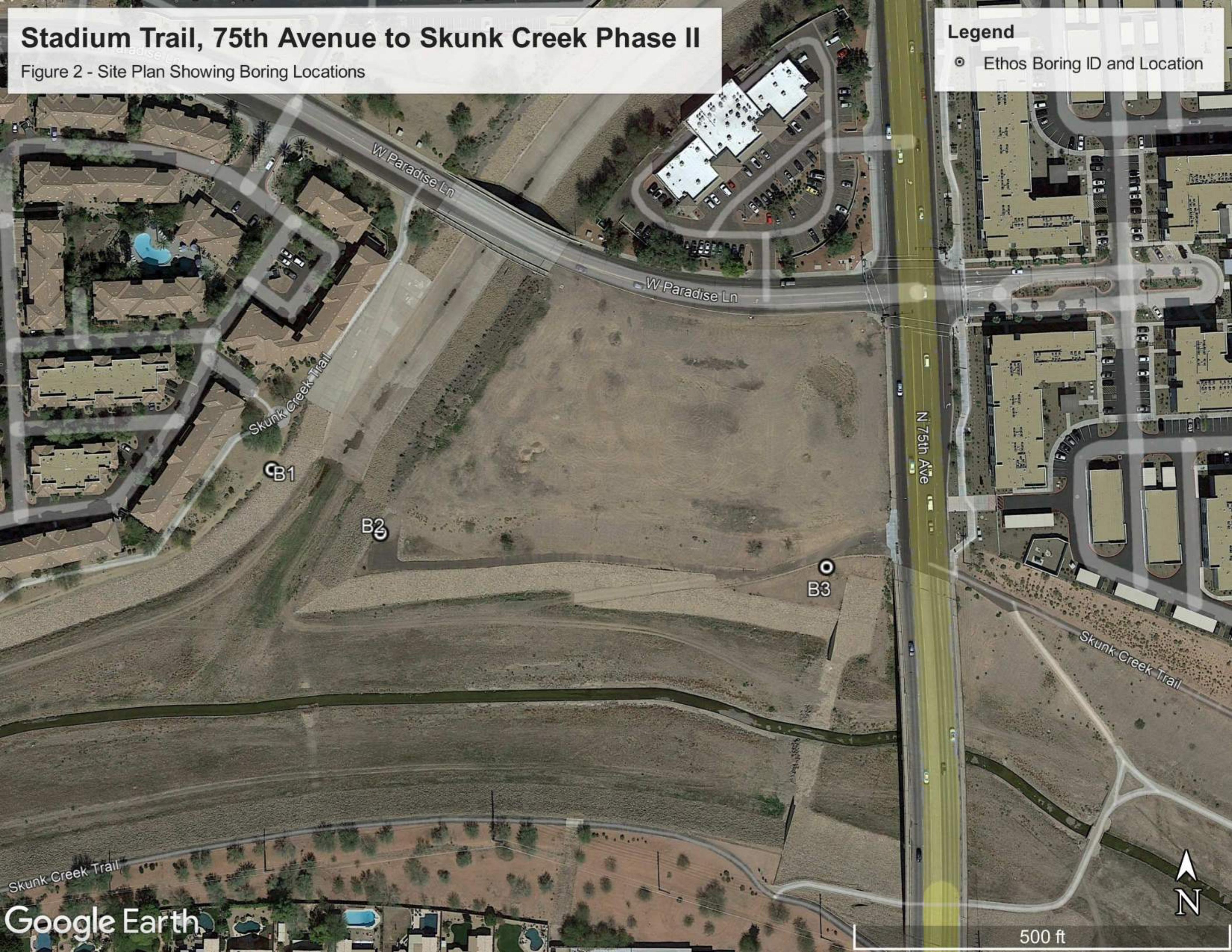
FIGURES

Stadium Trail, 75th Avenue to Skunk Creek Phase II

Figure 1 - Vicinity Map Showing Project Location



Stadium Trail, 75th Avenue to Skunk Creek Phase II

Figure 2 - Site Plan Showing Boring Locations

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.

<u>Relative Density</u>

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

<u>Relative Firmness</u>

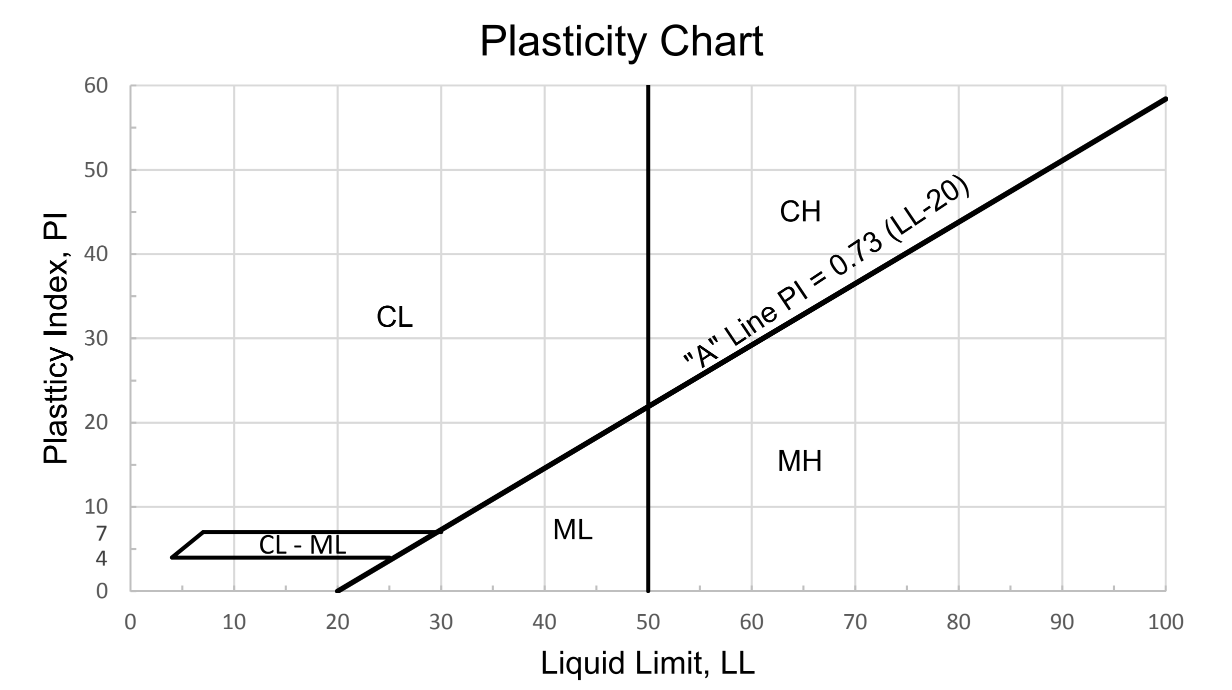
Relative Firmness for cohesive and/or cemented soils including silts, clays and silty to clayey sandy and gravelly soils is described based on the following SPT blowcounts:

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



Angularity	
Angular	
Subangular	
Subrounded	
Rounded	

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

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

Moisture
Slightly Moist
Moist
Wet
(Saturated)

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9180 S. Kyrene Road, Suite 104
Tempe, AZ 85284

BORING NUMBER B-1

CLIENT Jacobs Engineering Group, Inc.
PROJECT NAME Stadium Trail, Skunk Creek to 75th Avenue - Phase II
PROJECT NUMBER 2022 053
PROJECT LOCATION Peoria, Arizona
DATE STARTED 11/17/2022 **COMPLETED** 11/17/2022
BORING LOCATION 10+70, Skunk Creek Trail CST CL
DRILLER GSI **DRILLED BY** C. Fiesler
GPS COORDINATES 33.63354°N, -112.22288°E
DRILLING METHOD Tubex
GROUND ELEVATION 1203 ft **BOREHOLE DEPTH** 81 ft
RIG TYPE / # CME-75/109
GROUNDWATER DEPTH ---
HAMMER TYPE Auto **HAMMER EFFICIENCY** 92
LOGGED BY M. Meza **CHECKED BY** P. Garza

ELEVATION (ft)	DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
		SILTY, CLAYEY SAND WITH GRAVEL (SC-SM), subangular, firm, low plasticity, slightly moist, light brown, some gravel, predominantly fine to medium sand	SPT	6-14-14 (28)						
1200			AU							
	5	GRAVEL WITH SILT AND SAND (GP-GM), subangular, very dense, non plastic, slightly moist, brown, trace to some cobbles, some medium sand	R	50/4"						
1195	10	GRAVEL WITH SAND (GP), subangular, medium dense, non plastic, slightly moist, orangish brown to light brown, predominantly fine to coarse gravel, some to considerable medium to coarse sand	R	9-17		1.5	NP	NP	NP	4
1190		Note: Trace Subrounded Cobbles below 12'								
	15	SILTY GRAVEL WITH SAND (GM), subangular, very dense, non plastic, slightly moist, pinkish brown to light gray, trace cobbles, some medium sand	SPT	17-26-32 (58)						
1185	20	Note: Increased Cobbles below 20'	SPT	50/5"						
1180	25		SPT	30-50/4"						
1175	30	GRAVEL WITH SILTY CLAY (GC), subangular, hard, low to medium plasticity, slightly moist, orangish brown to light brown, trace cobbles, trace to some fine to medium sand	SPT	50/5"						
1170	35									

(Continued Next Page)

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BORING NUMBER B-1

CLIENT Jacobs Engineering Group, Inc. **PROJECT NAME** Stadium Trail, Skunk Creek to 75th Avenue - Phase II
PROJECT NUMBER 2022 053 **PROJECT LOCATION** Peoria, Arizona
DATE STARTED 11/17/2022 **COMPLETED** 11/17/2022 **BORING LOCATION** 10+70, Skunk Creek Trail CST CL
DRILLER GSI **DRILLED BY** C. Fiesler **GPS COORDINATES** 33.63354°N, -112.22288°E
DRILLING METHOD Tubex **GROUND ELEVATION** 1203 ft **BOREHOLE DEPTH** 81 ft
RIG TYPE / # CME-75/109 **GROUNDWATER DEPTH** ---
HAMMER TYPE Auto **HAMMER EFFICIENCY** 92 **LOGGED BY** M. Meza **CHECKED BY** P. Garza

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
			SANDY CLAY (CL), subangular, very firm, medium plasticity, moist, brown, trace fine gravel, considerable fine sand	SPT	15-19-25 (44)						
1165			CLAYEY GRAVEL WITH SAND (GC), subangular, low to medium plasticity, slightly moist, dark brown to brown, some fine sand								
	40		GRAVEL WITH SILT AND SAND (GP-GM), subangular, very dense, non plastic, slightly moist, orangish brown to light gray, trace cobbles, some medium to coarse sand	SPT	19-50/5"						
1160			SAND WITH CLAY (SP-SC), subangular, firm, low plasticity, moist, brown to orangish brown, some gravel, predominantly medium sand								
	45			R	13-29	87	17.9				
1155			SILTY GRAVEL WITH SAND (GP-GM), subangular, very dense, non plastic, slightly moist, pinkish brown to light gray, trace cobbles, some medium to coarse sand								
	50			SPT	12-50/5"						
1150											
	55			SPT	30-50/2"						
1145											
	60		SAND WITH SILT AND GRAVEL (SP-SM), subangular, very dense, non plastic, slightly moist, light brown to light gray, trace cobbles, predominantly fine to medium sand	SPT	17-35-44 (79)		3.5	NP	NP	NP	10
1140			GRAVEL WITH SAND (GP), subangular, very dense, non plastic, slightly moist, light brown to light gray, trace to some cobbles, some medium to coarse sand								
	65			SPT	27-50/3"						
1135											
	70										

(Continued Next Page)

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BORING NUMBER B-1

CLIENT Jacobs Engineering Group, Inc.
PROJECT NAME Stadium Trail, Skunk Creek to 75th Avenue - Phase II
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PROJECT LOCATION Peoria, Arizona
DATE STARTED 11/17/2022 **COMPLETED** 11/17/2022
BORING LOCATION 10+70, Skunk Creek Trail CST CL
DRILLER GSI **DRILLED BY** C. Fiesler
GPS COORDINATES 33.63354°N, -112.22288°E
DRILLING METHOD Tubex
GROUND ELEVATION 1203 ft **BOREHOLE DEPTH** 81 ft
RIG TYPE / # CME-75/109
GROUNDWATER DEPTH ---
HAMMER TYPE Auto **HAMMER EFFICIENCY** 92
LOGGED BY M. Meza **CHECKED BY** P. Garza

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
1130	75		SILTY, CLAYEY GRAVEL WITH SAND (GC), subangular, hard, low to medium plasticity, slightly moist, orangish brown to gray, trace to some cobbles, some medium to coarse sand *(continued)*	SPT	13-28-27 (55)						
			GRAVELLY SILTY CLAY (CL-ML), subangular, hard, low plasticity, slightly moist, light brown to orangish brown, some to considerable gravel, trace to some fine sand	SPT	8-14-50 (64)						
1125			GRAVEL WITH SAND (GP), subangular, very dense, non plastic, slightly moist, light brown to light gray, some to considerable medium to coarse sand								
	80		SILTY SAND (SM), subangular, medium dense, non plastic, moist, brown to orangish brown, occasional to trace fine gravel, predominantly fine to medium sand	R	14-26						

Bottom of borehole at 81.0 feet. Backfilled with 20% Bentonite Slurry.

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Tempe, AZ 85284

BORING NUMBER B-2

CLIENT Jacobs Engineering Group, Inc.
PROJECT NAME Stadium Trail, Skunk Creek to 75th Avenue - Phase II
PROJECT NUMBER 2022 053
PROJECT LOCATION Peoria, Arizona
DATE STARTED 11/16/2022 **COMPLETED** 11/16/2022
BORING LOCATION 12+30, Skunk Creek Trail CST CL
DRILLER GSI **DRILLED BY** C. Fiesler
GPS COORDINATES 33.63324°N, -112.22227°E
DRILLING METHOD Tubex
GROUND ELEVATION 1202 ft **BOREHOLE DEPTH** 81 ft
RIG TYPE / # CME-75/109
GROUNDWATER DEPTH ---
HAMMER TYPE Auto **HAMMER EFFICIENCY** 92
LOGGED BY M. Meza **CHECKED BY** P. Garza

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
1200			CLAYEY SAND (SC), subangular, firm, medium plasticity, slightly moist, brown to light brown, trace to some fine gravel, predominantly fine to medium sand	SPT	6-10-8 (18)		4.3	36	17	19	39
	5		GRAVEL WITH SILT AND SAND (GP-GM), subangular, very dense, non plastic, slightly moist, light brown to light gray, trace cobbles, some to considerable fine to medium sand	AU							
				SPT	50/2"						
1195			SAND WITH GRAVEL (SP), subangular, non plastic, slightly moist, light brown, some gravel, predominantly medium to coarse sand								
	10		GRAVEL WITH SAND (GP), subangular, medium dense, non plastic, slightly moist, light brown to light gray, trace cobbles, some medium to coarse sand	SPT	5-12-12 (24)						
1190											
	15		SILTY GRAVEL WITH SAND (GP-GM), subrounded, very dense, non plastic, slightly moist, light brown to light gray, trace cobbles, some to considerable medium to coarse sand	SPT	5-16-14 (30)						
1185											
	20		CLAYEY SAND WITH GRAVEL (SC), subrounded, very dense, medium plasticity, slightly moist, light gray to pinkish brown, trace cobbles, considerable fine gravel, predominantly medium to coarse sand	SPT	12-27-50/5"		3.3				12
1180											
	25		SILTY GRAVEL WITH SAND (GP-GM), subrounded, very dense, non plastic, slightly moist, light brown to light gray, trace cobbles, some to considerable medium to coarse sand Note: Pinkish Brown to Light Brown below 25'	SPT	14-28-50 (78)						
1175											
	30		SAND WITH CLAY (SP-SC), subangular, very firm, non plastic, moist, weak lime cementation, brown to orangish brown, trace to some gravel, predominantly medium sand	SPT	16-18-22 (40)		7.3				
1170											
	35										

(Continued Next Page)

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BORING NUMBER B-2

CLIENT Jacobs Engineering Group, Inc.		**PROJECT NAME** Stadium Trail, Skunk Creek to 75th Avenue - Phase II	
PROJECT NUMBER 2022 053		**PROJECT LOCATION** Peoria, Arizona	
DATE STARTED 11/16/2022	**COMPLETED** 11/16/2022	**BORING LOCATION** 12+30, Skunk Creek Trail CST CL	
DRILLER GSI	**DRILLED BY** C. Fiesler	**GPS COORDINATES** 33.63324°N, -112.22227°E	
DRILLING METHOD Tubex		**GROUND ELEVATION** 1202 ft	**BOREHOLE DEPTH** 81 ft
RIG TYPE / # CME-75/109		**GROUNDWATER DEPTH** ---	
HAMMER TYPE Auto	**HAMMER EFFICIENCY** 92	**LOGGED BY** M. Meza	**CHECKED BY** P. Garza

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
			GRAVEL WITH SAND (GP), subangular, very dense, non plastic, slightly moist, light brown to pinkish brown, trace to some cobbles, some medium to coarse sand *(continued)*	SPT	37-50/2"						
1165			GRAVEL WITH SILT AND SAND (GP-GM), subangular, very dense, non plastic, slightly moist, brown to reddish brown, trace to some cobbles, some medium to coarse sand								
	40			SPT	30-50/5"						
1160											
	45		Note: Orangish Brown to Light Grey below 44'	SPT	50/5"						
1155											
	50			SPT	50/5"						
1150											
	55			SPT	50/4"						
1145											
	60			SPT	31-50/5"						
1140			Note: Pinkish Brown to Light Grey below 62'								
	65			SPT	40-50/5"						
1135											
	70										

(Continued Next Page)

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BORING NUMBER B-2

CLIENT Jacobs Engineering Group, Inc.
PROJECT NAME Stadium Trail, Skunk Creek to 75th Avenue - Phase II
PROJECT NUMBER 2022 053
PROJECT LOCATION Peoria, Arizona
DATE STARTED 11/16/2022 **COMPLETED** 11/16/2022
BORING LOCATION 12+30, Skunk Creek Trail CST CL
DRILLER GSI **DRILLED BY** C. Fiesler
GPS COORDINATES 33.63324°N, -112.22227°E
DRILLING METHOD Tubex
GROUND ELEVATION 1202 ft **BOREHOLE DEPTH** 81 ft
RIG TYPE / # CME-75/109
GROUNDWATER DEPTH ---
HAMMER TYPE Auto **HAMMER EFFICIENCY** 92
LOGGED BY M. Meza **CHECKED BY** P. Garza

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
1130	75		GRAVEL WITH SILT AND SAND (GP-GM), subangular, very dense, non plastic, slightly moist, brown to reddish brown, trace to some cobbles, some medium to coarse sand *(continued)* Note: Orangish Brown to Light Grey below 70'	SPT	16-38-50/5"						
1125	80		SAND WITH SILT AND GRAVEL (SP-SM), subangular, very dense, non plastic, slightly moist, brown to orangish brown, some gravel, predominantly medium to coarse sand	SPT	18-38-41 (79)						
			SILTY SAND (SM), subangular, medium dense, non plastic, light brown, occasional to trace fine gravel, predominantly fine to medium sand	R	13-30	92	18.6				

Bottom of borehole at 81.0 feet. Backfilled with 20% Bentonite Slurry.

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Tempe, AZ 85284

BORING NUMBER B-3

CLIENT Jacobs Engineering Group, Inc.
PROJECT NAME Stadium Trail, Skunk Creek to 75th Avenue - Phase II
PROJECT NUMBER 2022 053
PROJECT LOCATION Peoria, Arizona
DATE STARTED 11/18/2022 **COMPLETED** 11/18/2022
BORING LOCATION 18+25, Skunk Creek Trail CST CL
DRILLER GSI **DRILLED BY** C. Fiesler
GPS COORDINATES 33.6331°N, -112.22048°E
DRILLING METHOD Tubex
GROUND ELEVATION 1203 ft **BOREHOLE DEPTH** 21.5 ft
RIG TYPE / # CME-85/118
GROUNDWATER DEPTH ---
HAMMER TYPE Auto **HAMMER EFFICIENCY** 91
LOGGED BY M. Meza **CHECKED BY** P. Garza

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
			SILTY, CLAYEY SAND (SC-SM), subangular, soft, low plasticity, slightly moist, brown to light brown, trace to some gravel, predominantly fine sand	R	3-6	106	3.3				
1200			SANDY SILTY CLAY (CL-ML), subangular, soft, medium plasticity, moist, brown to light brown, occasional gravel, some to considerable fine sand	AU			6.9	24	6	18	65
	5			R	3-4						
1195			GRAVEL WITH SAND (GP), subangular, medium dense, non plastic, slightly moist, grayish brown to light gray, trace cobbles, considerable medium to coarse sand								
	10			SPT	3-10-9 (19)						
1190			Note: Increased Cobbles below 13'								
	15		GRAVEL WITH SILT AND SAND (GP-GM), subangular, very dense to dense, non plastic, slightly moist, pinkish brown to light gray, trace cobbles, some to considerable medium to coarse sand	SPT	9-21-11 (32)						
1185			Note: Increased Cobbles below 18'								
	20			SPT	22-35-45 (80)						

Bottom of borehole at 21.5 feet. Backfilled with 20% Bentonite Slurry.

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)	In Place Dry Density (pcf)[1] (ASTM D2937)	Optimum Moisture Content (%) (by ASTM D698A)	Maximum Dry Density (pcf)[1] (by ASTM D698A)	Expansion % (ASTM D4546)	Consolidation% (ASTM D2435)	Direct Shear (ASTM D3080)	pH (AZ 236)	Resistivity ohm-cm (AZ 236)	Sulfates (ppm)[2] (AZ 733)	Chlorides (ppm)[2] (AZ 736)
	Begin	End															
B-1	0.5	5												8.3	704	3	198
B-1	10	11	GP	3.5	NV	NP	1.5						x				
B-1	45	46					17.9	86.6				1.8					
B-1	60	61.5	SP-SM	9.9	NV	NP	3.5										
B-1	80	81					17.1	94.4									
B-2	0	1.5	SC	39	36	17	4.3										
B-2	0.5	5												9.5	2,011	4	19
B-2	20	21.4	SC	12	26	11	3.3										
B-2	30	31.5					7.3										
B-2	80	81					18.6	92.4									
B-3	0	1					3.3	105.6					x				
B-3	2	5	CL-ML	65	24	6	6.9		12.8	117.1	1.1						
B-3	5	6										2.9					
			Average	**26.0**	**---**	**---**	**8.4**	**94.8**	**12.8**	**117.1**	**1.1**	**2.35**	**---**	**8.9**	**1,358**	**4**	**109**
			Standard Deviation	**25.8**	**---**	**---**	**6.8**	**8.0**	**#DIV/0!**	**#DIV/0!**	**#DIV/0!**	**0.8**	**---**	**0.8**	**924**	**1**	**127**
			Maximum	**65**	**36**	**17**	**18.6**	**105.6**	**12.8**	**117.1**	**1.1**	**2.90**	**0**	**9.5**	**2,011**	**4**	**198**
			Minimum	**3.5**	**NV**	**NP**	**1.5**	**86.6**	**12.8**	**117.1**	**1.1**	**1.80**	**0**	**8.3**	**704**	**3**	**19**
			Count	**5**	**5**	**5**	**10**	**4**	**1**	**1**	**1**	**2**	**2**	**2**	**2**	**2**	**2**

PROJECT: ACS Project #2202066
LOCATION: Peoria, AZ
MATERIAL: Native

JOB NO: 19-2012-2017
WORK ORDER NO: N/A
DATE ASSIGNED: 11/28/22

MECHANICAL SIEVE ANALYSIS (ASTM C136/C117) PLASTICITY INDEX (ASTM D4318)
GROUP SYMBOL, USCS (ASTM D2487)

PERCENT PASSING BY WEIGHT

				Silt or Clay	SAND								GRAVEL									COBBLES	
					Fine			Medium			Coarse		Fine				Coarse						
Location & Depth	USCS	LL	PI	#200	#100	#50	#40	#30	#16	#10	#8	#4	1/4"	3/8"	1/2"	3/4"	1"	1 1/4"	1 1/2"	2"	3"	6"	Lab #
B-1 (10.0-11.0')	GP	NV	NP	3.5	4	6	7	9	15	21	23	33	38	47	53	61	64	69	82	100	100	100	22-1797-01

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ACS SERVICES LLC

Laboratory Soil Test Results

ACS PROJECT #	2202066	Material Type:	Soil
ACS Lab #	22-5002-4	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	-
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Sampled By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Test Date:	12/2/2022
Project City	Peoria	Tested By:	Fernando Montero
Sample Location:	B-1 @ 60 - 61.5	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"	9	91	
3/4"	3	88	
1/2"	10	78	
3/8"	4	74	
1/4"	7	67	
#4	4	63	
#8	10	53	
#10	2	51	
#16	7	44	
#30	12	32	
#40	6	26	
#50	5	20	
#100	7	14	
#200	4	9.9	

Liquid Limit (AASHTO T-89)	
Plastic Limit (AASHTO T-90)	
Plasticity Index (AASHTO T-90)	NP
Moisture Content (AASHTO T-265)	3.5
USCS Soil Classification	SP-SM

Group Name (ASTM D2487)
Poorly graded SAND with silt and gravel

Dylan Ward
Laboratory Manager

Dylan Ward
Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2202066	Material Type:	Soil
ACS Lab #	22-5002-6	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	-
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Sampled By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Test Date:	12/2/2022
Project City	Peoria	Tested By:	James Karl
Sample Location:	B-2 @ 0 - 1.5	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"	2	98	
3/8"	3	95	
1/4"	5	90	
#4	3	86	
#8	8	78	
#10	2	76	
#16	8	69	
#30	9	59	
#40	4	55	
#50	4	51	
#100	6	45	
#200	7	38.8	

Liquid Limit (AASHTO T-89)	36
Plastic Limit (AASHTO T-90)	19
Plasticity Index (AASHTO T-90)	17
Moisture Content (AASHTO T-265)	4.3
USCS Soil Classification	SC

Group Name (ASTM D2487)
Clayey SAND

Dylan Ward

Laboratory Manager

Dylan Ward

Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2202066	Material Type:	Soil
ACS Lab #	22-5002-8	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	-
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Sampled By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Test Date:	12/2/2022
Project City	Peoria	Tested By:	James Karl
Sample Location:	B-2 @ 20 - 21.4	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"	5	95	
3/4"	5	90	
1/2"	7	83	
3/8"	9	74	
1/4"	12	62	
#4	5	56	
#8	11	45	
#10	2	43	
#16	8	35	
#30	9	26	
#40	4	22	
#50	3	19	
#100	4	15	
#200	3	12.2	

Liquid Limit (AASHTO T-89)	26
Plastic Limit (AASHTO T-90)	15
Plasticity Index (AASHTO T-90)	11
Moisture Content (AASHTO T-265)	3.3
USCS Soil Classification	SC

Group Name (ASTM D2487)
Clayey SAND with gravel

Dylan Ward
Laboratory Manager

Dylan Ward
Signature

ACS SERVICES LLC

Laboratory Soil Test Results

ACS PROJECT #	2202066	Material Type:	Soil
ACS Lab #	22-5002-9	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	-
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Sampled By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Test Date:	1/0/1900
Project City	Peoria	Tested By:	0
Sample Location:	B-2 @ 30 - 31.5	Reviewed By:	Dylan Ward

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

Sieve Size	% Retained	% Passed	Specs
6"			
3"			
2 1/2"			
2"			
1 1/2"			
1"			
3/4"			
1/2"			
3/8"			
1/4"			
#4			
#8			
#10			
#16			
#30			
#40			
#50			
#100			
#200			

Liquid Limit (AASHTO T-89)	
Plastic Limit (AASHTO T-90)	
Plasticity Index (AASHTO T-90)	
Moisture Content (AASHTO T-265)	7.3

USCS Soil Classification	

Group Name (ASTM D2487)

Dylan Ward
Laboratory Manager

Dylan Ward
Signature

ACS Services LLC	Laboratory Soil Test Results

ACS PROJECT #	2202066	Material Type:	Soil
ACS Lab #	22-5002-12	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	-
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Sampled By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Test Date:	12/6/2022
Project City	Peoria	Tested By:	Brian Karl
Sample Location:	B-3 @ 2 - 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"	1	99	
1/2"	1	99	
3/8"	1	98	
1/4"	1	97	
#4	1	97	
#8	3	93	
#10	1	93	
#16	2	91	
#30	3	88	
#40	2	86	
#50	2	84	
#100	7	77	
#200	11	65.4	

Liquid Limit (AASHTO T-89)	24
Plastic Limit (AASHTO T-90)	18
Plasticity Index (AASHTO T-90)	6
Moisture Content (AASHTO T-265)	6.9
USCS Soil Classification	CL-ML

Group Name (ASTM D2487)
Sandy SILTY CLAY

Dylan Ward
Laboratory Manager

Dylan Ward
Signature

ACS Services LLC

Job #	2202066	Material Type:	Soil
Lab #	22-5002	Extraction Date:	N/A
Client:	Ethos	Extracted By:	Client
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Laboratory Test Date	12/6/2022
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Laboratory Tested By:	Fernando Montero
Project City:	Peoria	Reviewed By:	Dylan Ward
Material Source:	N/A		

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)**
22-5002-3	B-1 @ 45 - 46	369.8	313.6	17.9%	3	498.1	128.3	86.6
22-5002-5	B-1 @ 80 - 81	533.8	456.0	17.1%	4	704.2	170.4	94.4
22-5002-10	B-2 @ 80 - 81	661.9	557.9	18.6%	5	885.7	223.8	92.4

PROJECT: ACS Project #2202066
LOCATION: Peoria, AZ
MATERIAL: Native

JOB NO: 19-2012-2017
WORK ORDER NO: N/A
DATE ASSIGNED: 11/28/22

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

		MOISTURE						
LAB #	**BORING**	**WET WT. (g)**	**DRY WT. (g)**	**MOISTURE CONTENT**	**NUMBER OF RINGS**			**DRY DENSITY (pcf)**
22-1797-01	B-1 (10.0-11.0')	573.1	564.9	1.5%				
22-1797-02	B-3 (0.0-1.0')	417.0	403.7	3.3%	5	872.4	213.8	105.6

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Phoenix, AZ 85040

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

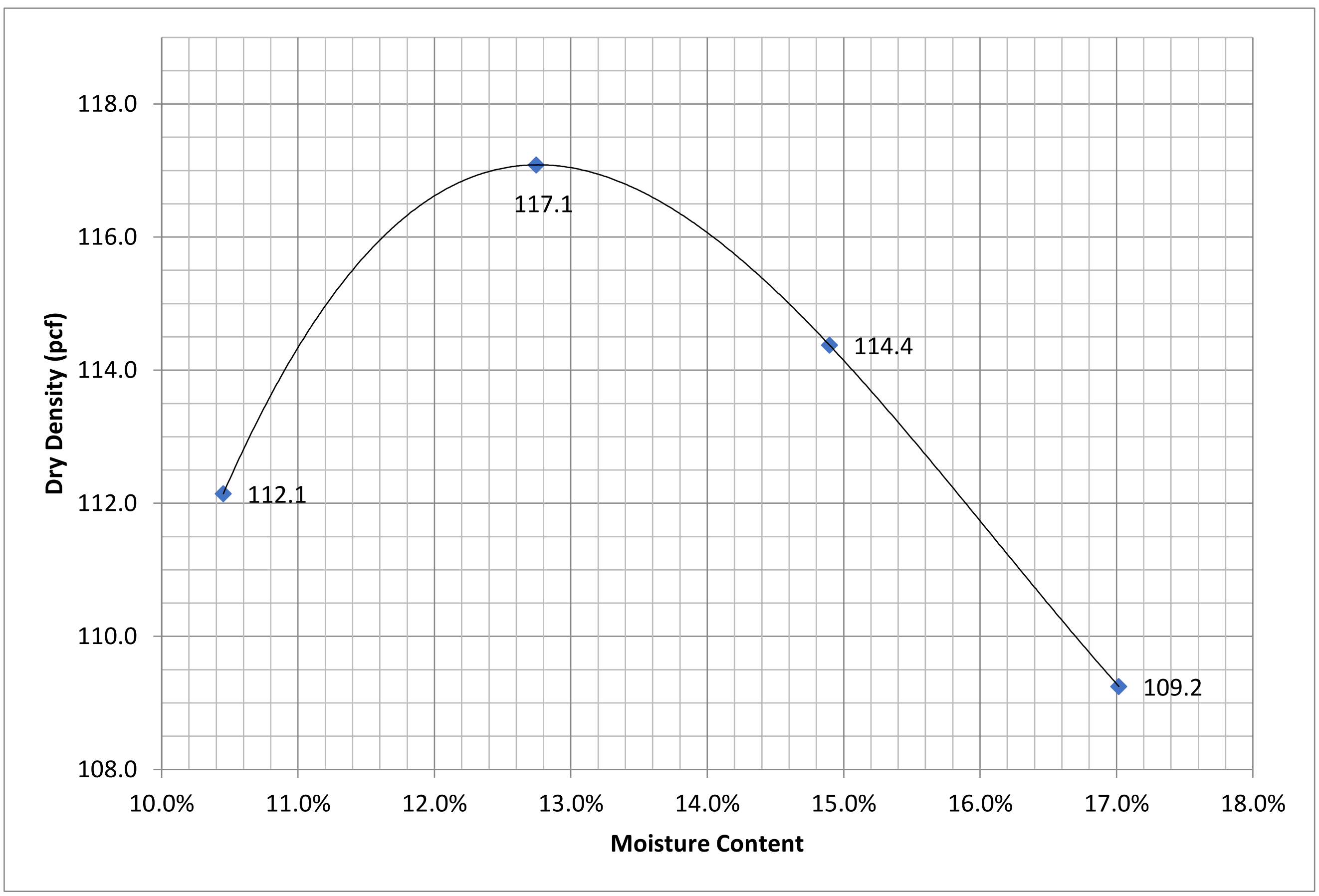
ACS Project #	2202066	Material Type:	Soil
ACS Lab #	22-5002-12	Material Supplier:	-
Client Name:	Ethos Engineering, LLC	Sample Date:	-
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Sampled By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Date Tested:	12/6/2022
Project City:	Peoria	Tested By:	Keagen Mayfield
		Reviewed By:	Dylan Ward

Sample Location: B-3 @ 2 - 5

Method: ☑ A ☐ B ☐ C ☐ D

Dry Density	112.1	117.1	114.4	109.2
Moisture Content	10.5%	12.7%	14.9%	17.0%

Uncorrected Dry Density	117.1	Uncorrected Moisture Content	12.8
% Rock	3	% Passing	97
Rock Corrected Dry Density	**117.1**	Rock Corrected Moisture Content	**12.8**
Specific Gravity of Oversize Aggregate	**2.600**		



Dylan Ward

Project Manager

ACS Services LLC	Swell / Settlement Potential of Soils ASTM D4546-03

ACS Project #	**2202066**	Material Type:	Soil
ACS Lab #	22-5002-12	Sample Date:	-
Client Name:	Ethos Engineering, LLC	Sampled By:	Client
Project Name:	ium Trail, 75th Avenue to Skunk Creek (Phas	Laboratory Test Date:	12/6/2022
Project Address:	ium Trail, 75th Avenue to Skunk Creek (Phas	Laboratory Test By:	Keagen Mayfield
Project City:	Peoria	Reviewed By:	Dylan Ward
Sample Source:	B-3 @ 2 - 5		

Test Method: ☐ A ☐ B ☒ C

Standard Proctor Information

Maximum Dry Density:	117.1	pcf
Optimum Moisture Content:	12.8	%

Specimen Preparation

95% of Maximum Dry Density:	111.2	pcf
-2% of Optimum Moisture Content:	10.8	%
Wet Density of Sample:	123.3	pcf
Test Weight (Wet):	148.8	g
Test Weight (Dry):	134.3	g
Moisture Sample (Wet):	122.5	g
Moisture Sample (Dry):	115.1	g
Sample Moisture Content:	6.4	%
Sample Weight (In situ):	143.0	g
Additional Moisture Needed:	5.9	ml
Ring Weight:	45.3	g
Total Weight Ring + Sample:	194.1	g
Initial Sample Saturation:	37	%

Specimen Test Data Post-Swell Test

Initial Specimen Height:		in
Final Specimen Height:		in
Final Specimen Weight:		g
Final Specimen Moisture Content:		%
Final Sample Saturation:		%

Final Wet Weight:

Final Dry Weight:

Swell / Settlement Potential Results

Load at Inundation (ksf)	
% Strain Before Inundation:	
% Strain After Inundation:	
Percent Heave:	

Correction Factor

Calibration Correction Factor	NONE

Corrected Swell	1.1

Ring Mold Information

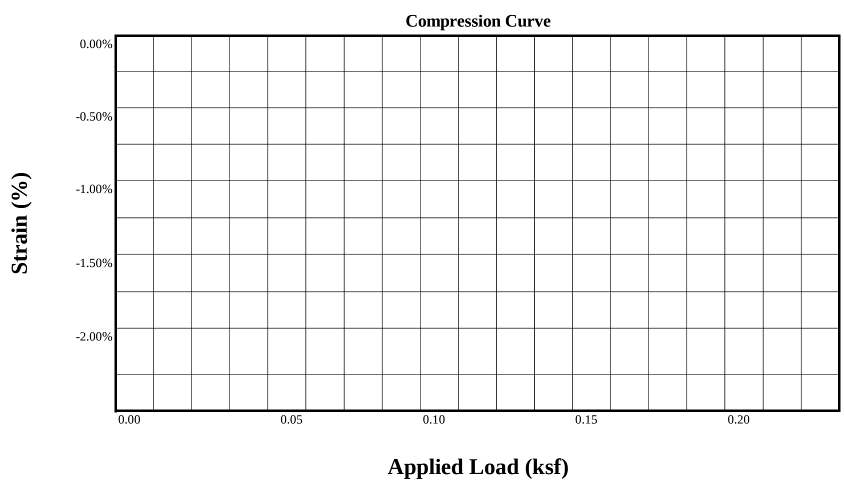
Ring Weight (g)	45.3
Ring Height (in)	1.000
Ring Diameter (in)	2.421
Ring Area (in^2)	4.604

Weight Used in Swell Test

Load Weight:	144 psf

Dial Readings During Test

Time Elapsed	Dial Reading (0.001")
0 (w/ seating pressure)	0.000
0 (w/ load applied)	0.000
30 Seconds	0.002
1 Minute	0.003
2 Minutes	0.006
4 Minutes	0.007
8 Minutes	0.009
15 Minutes	0.009
30 Minutes	0.010
1 Hour	0.010
2 Hours	0.011
4 Hours	0.011
8 Hours	0.011
15 Hours	0.011
24 Hours	0.011



Dylan Ward
Laboratory Manager

Dylan Ward
Signature

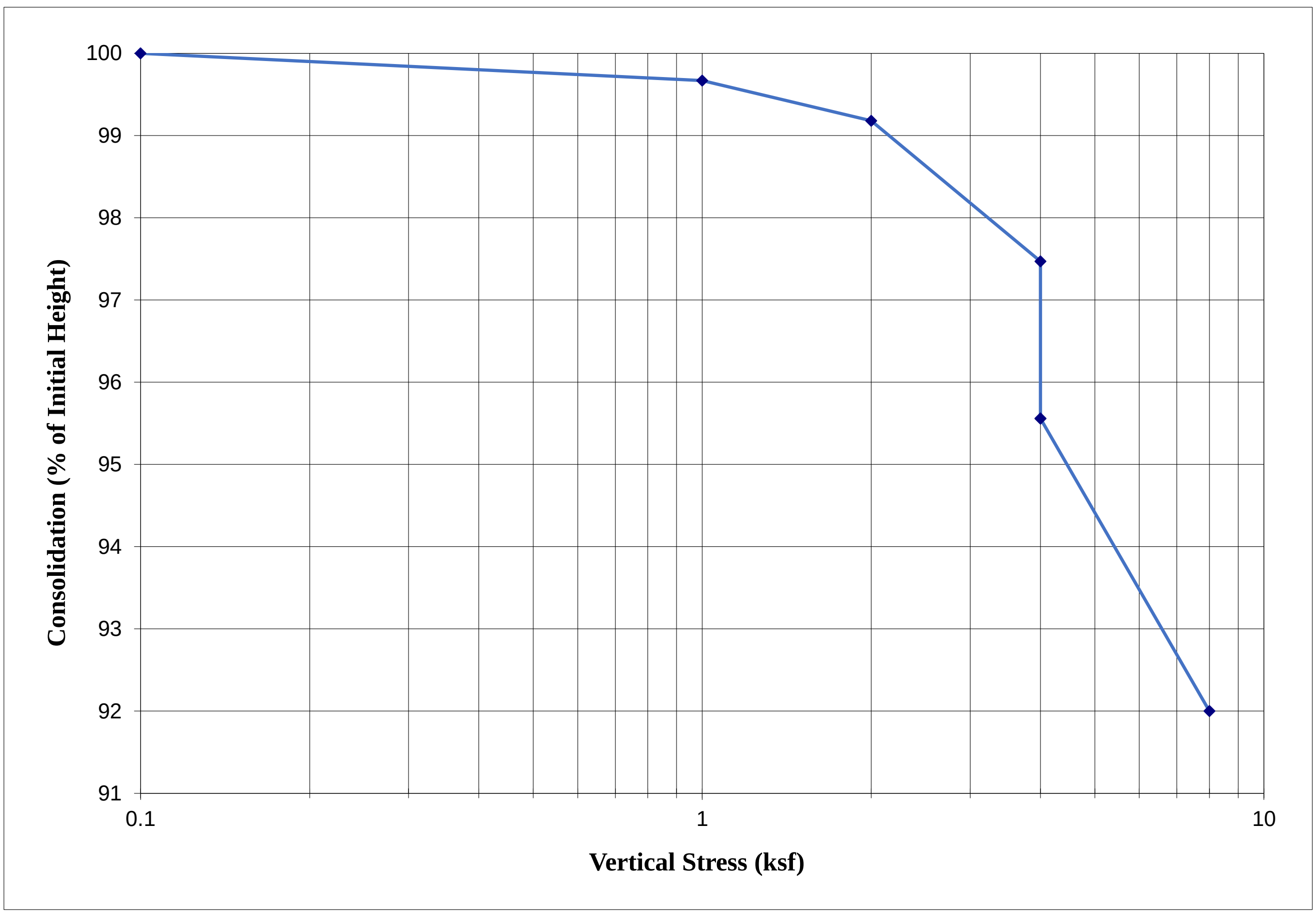
ACS Services LLC

ENGINEERING DESIGN • MATERIAL TESTING • CONSTRUCTION INSPECTION

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

ACS Project No.:	**2202066**		
Lab No.:	**22-5002-3**	Material Type:	Soil
Client:	Ethos Engineering, LLC	Date of Extraction:	-
Project Name:	Stadium Trail, 75th Avenue to Skui	Extracted By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skui	Date of Lab Test:	12/6/2022
Project City:	Peoria	Lab Tested By:	Fernando Montero
Sample Location:	B-1 @ 45 - 46	Reviewed By:	Dylan Ward

INITIAL VOLUME (cu.in)	4.60	FINAL VOLUME (cu.in)	4.40
INITIAL MOISTURE CONTENT	12.9%	FINAL MOISTURE CONTENT	18.8%
INITIAL DRY DENSITY(pcf)	96.7	FINAL DRY DENSITY(pcf)	101.2
INITIAL DEGREE OF SATURATION	48%	FINAL DEGREE OF SATURATION	79%
INITIAL VOID RATIO	0.7	FINAL VOID RATIO	0.6
ESTIMATED SPECIFIC GRAVITY	2.65	SATURATED AT	4 ksf



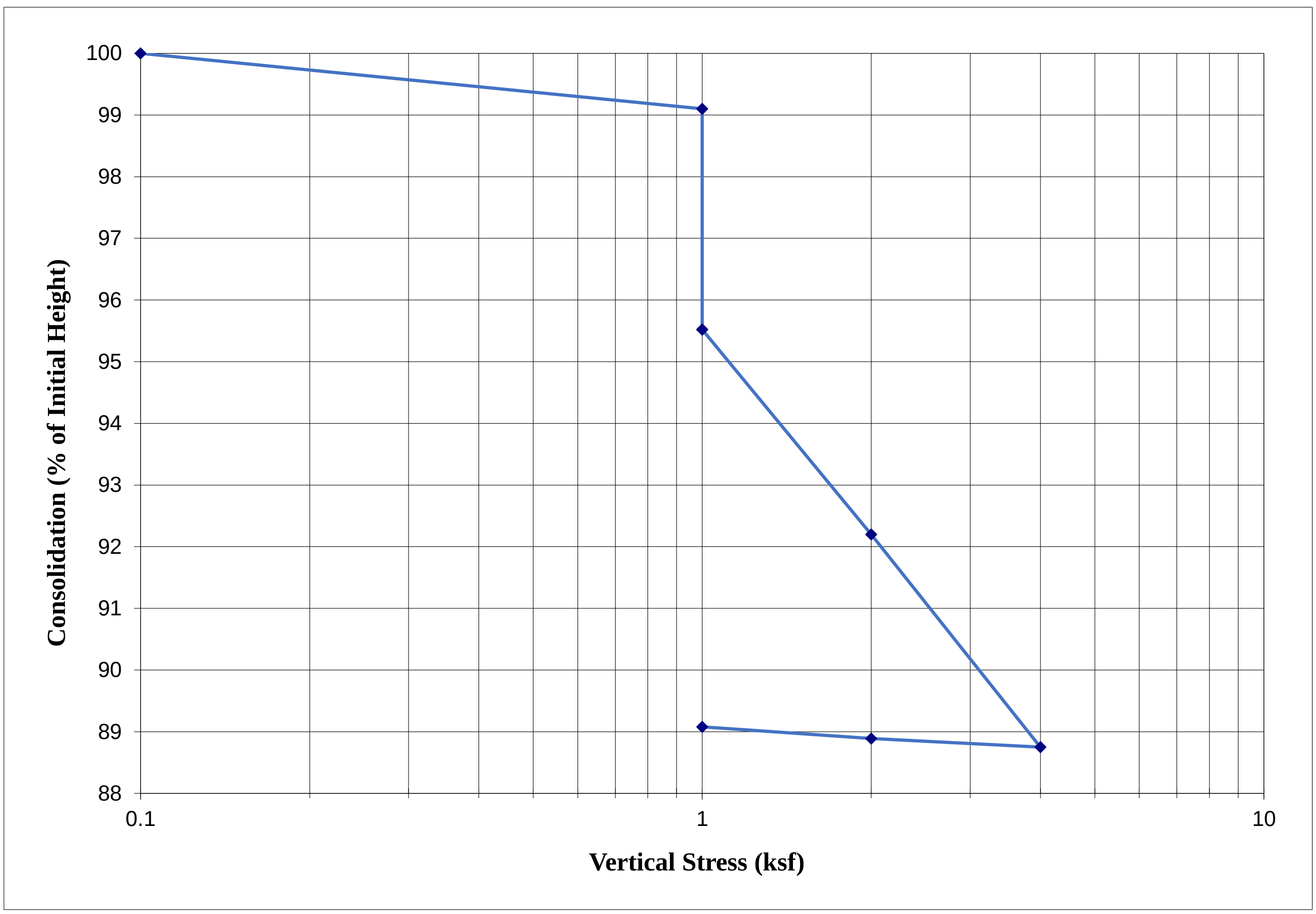
ACS Services LLC

ENGINEERING DESIGN • MATERIAL TESTING • CONSTRUCTION INSPECTION

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

ACS Project No.:	**2202066**		
Lab No.:	**22-5002-13**	Material Type:	Soil
Client:	Ethos Engineering, LLC	Date of Extraction:	-
Project Name:	Stadium Trail, 75th Avenue to Skun	Extracted By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skun	Date of Lab Test:	12/6/2022
Project City:	Peoria	Lab Tested By:	Fernando Montero
Sample Location:	B-3 @ 5 - 6	Reviewed By:	Dylan Ward

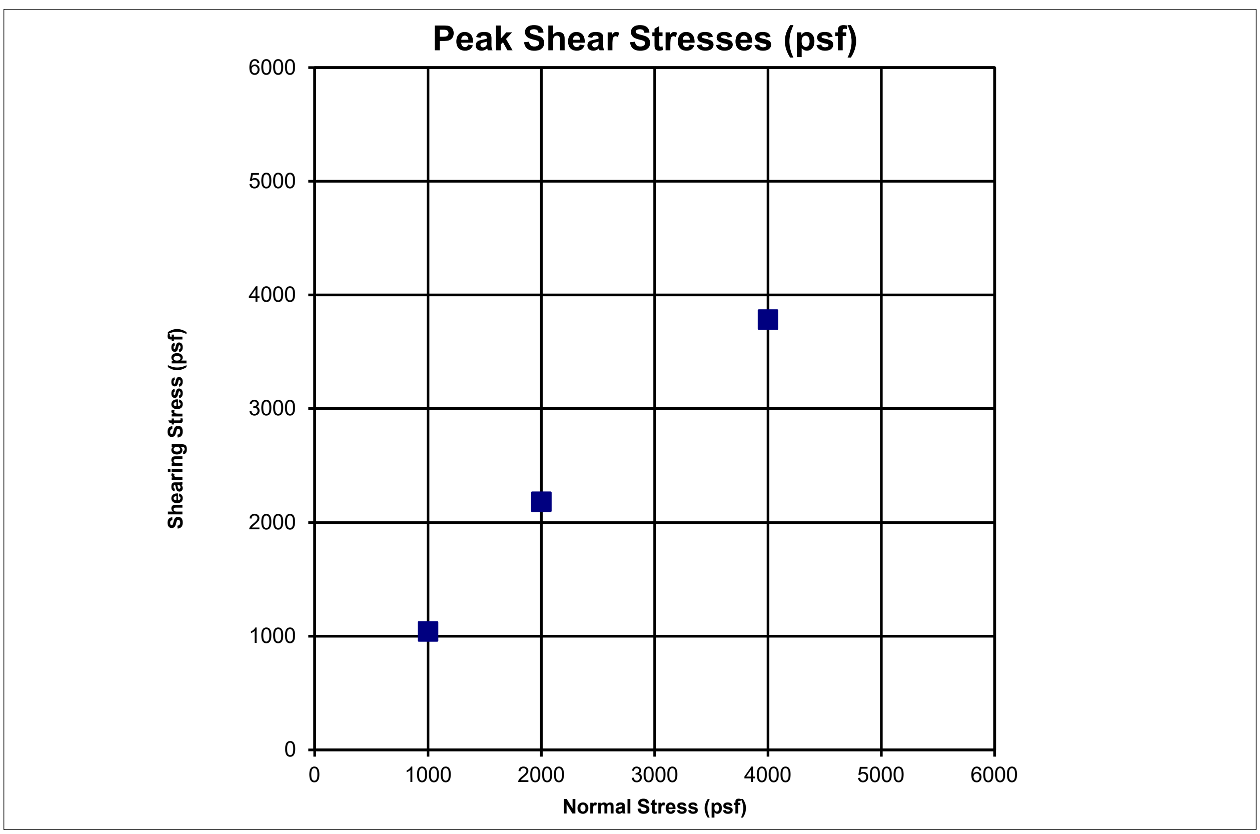
INITIAL VOLUME (cu.in)	4.60	FINAL VOLUME (cu.in)	4.09
INITIAL MOISTURE CONTENT	5.8%	FINAL MOISTURE CONTENT	21.4%
INITIAL DRY DENSITY(pcf)	93.4	FINAL DRY DENSITY(pcf)	105.2
INITIAL DEGREE OF SATURATION	20%	FINAL DEGREE OF SATURATION	99%
INITIAL VOID RATIO	0.8	FINAL VOID RATIO	0.6
ESTIMATED SPECIFIC GRAVITY	2.65	SATURATED AT	1 ksf



PROJECT: ACS Project #2202066	**JOB NO:** 19-2012-2017
LOCATION: Peoria, AZ	**WORK ORDER NO:** N/A
MATERIAL: Native	**LAB NO:** 22-1797-01
SAMPLE SOURCE: B-1 (10.0-11.0')	**DATE ASSIGNED:** 11/28/2022
SAMPLE PREPARATION: Saturated - 1, 2, and 4ksf	

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.):	0.986	0.996	0.994
Shearing device used:	Humboldt Automated Shear Test System by Trautwein Soil Testing Equipment		
Rate of deformation (in/min):	0.01	0.01	0.01
Direct shear point:	1	2	3
Dry mass of specimen (g):	121.6	129.8	133.7
Initial Moisture Content:	3.9%	3.5%	3.9%
Initial Wet Density (pcf):	104.6	111.2	115.0
Initial Dry Density (pcf):	100.7	107.5	110.7
Final Moisture Content:	15.3%	14.3%	15.5%
Final Wet Density (pcf):	117.7	123.4	128.6
Final Dry Density (pcf):	102.1	108.0	111.4
Normal Stress (psf):	1000	2000	4000
Maximum Shearing Stress (psf):	1041	2182	3785
Vertical Deformation @ Max Shear (in):	0.206	0.251	0.275
Horizontal Deformation @ Max Shear (in):	0.500	0.169	0.307



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Phoenix, AZ 85040

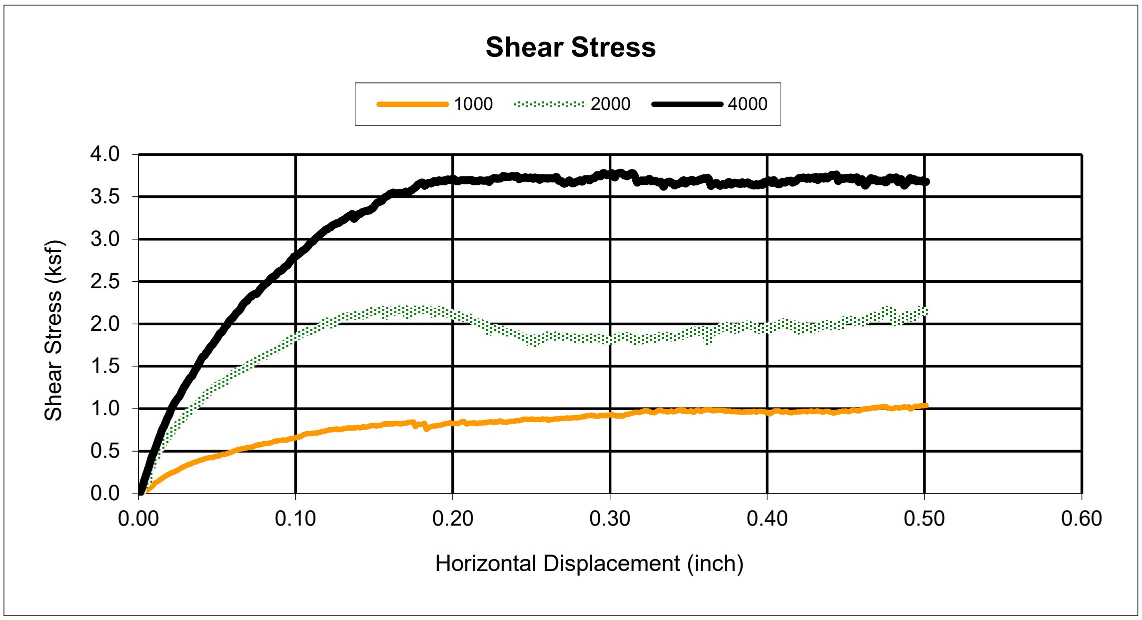
REVIEWED BY Hiram Franco

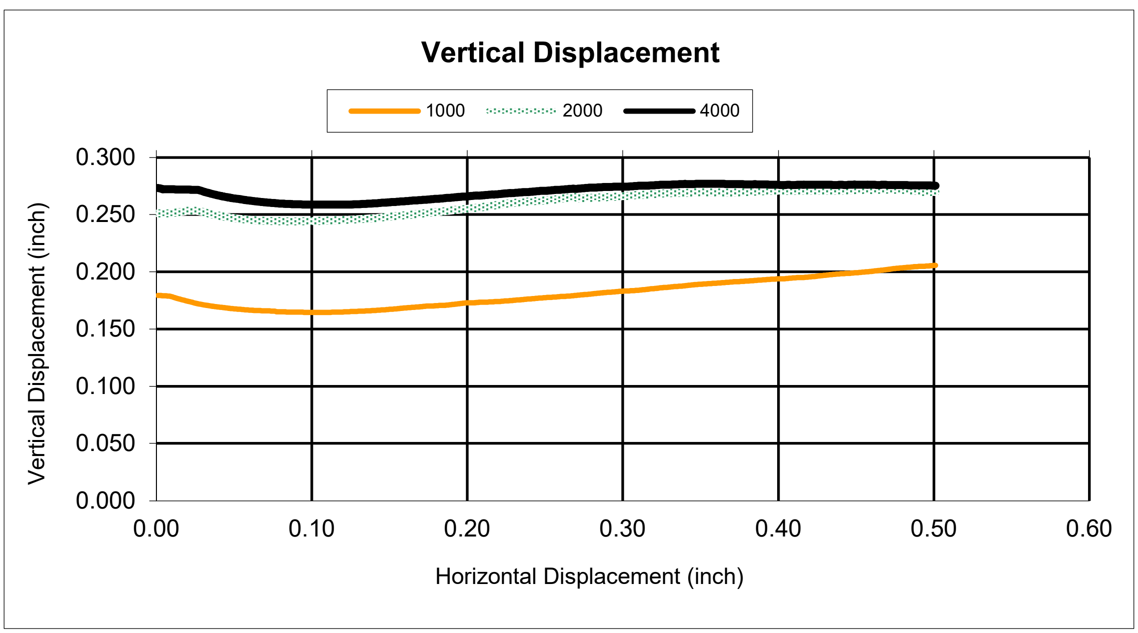
PROJECT: ACS Project #2202066
LOCATION: Peoria, AZ
MATERIAL: Native
SAMPLE SOURCE: B-1 (10.0-11.0')
SAMPLE PREPARATION: Saturated - 1, 2, and 4ksf

JOB NO: 19-2012-2017
WORK ORDER NO: N/A
LAB NO: 22-1797-01
DATE ASSIGNED: 11/28/2022

NORMAL LOADS (psf): 1000 2000 4000

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





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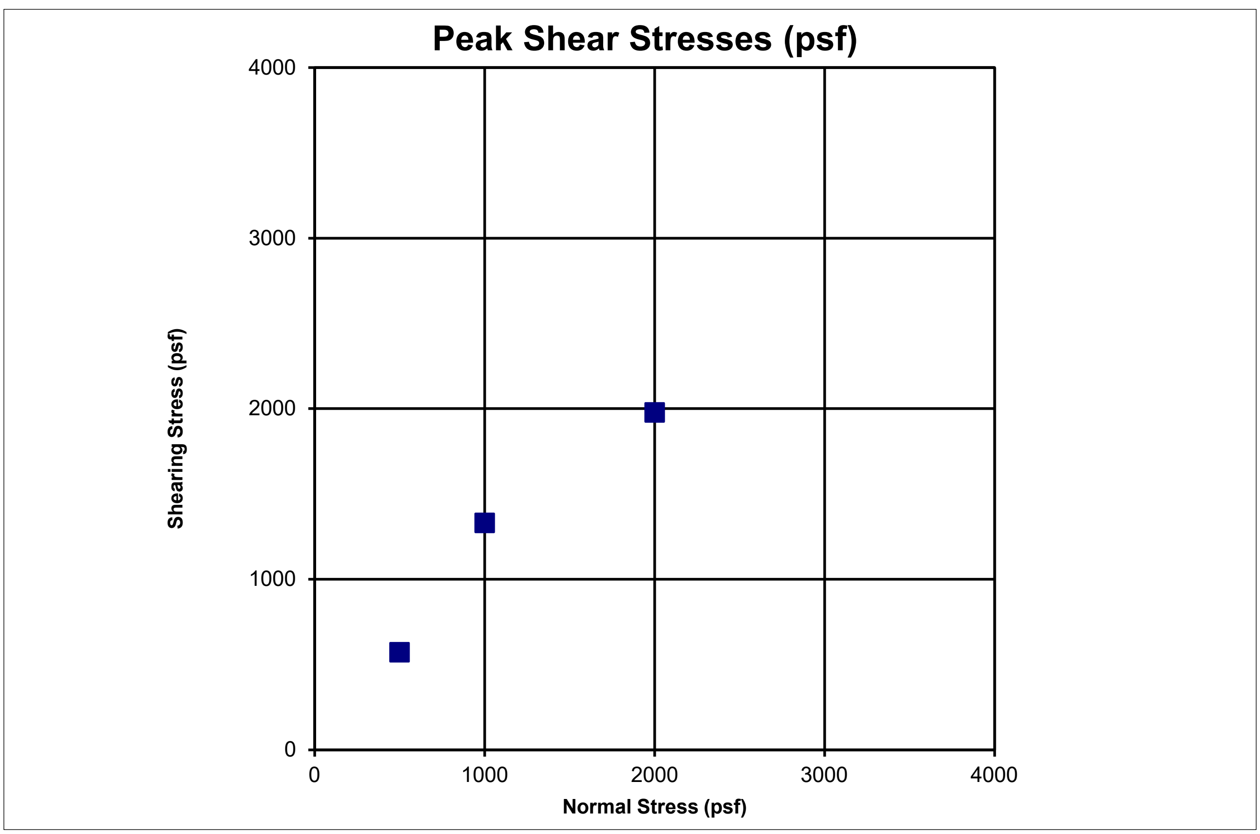
3630 E Wier Ave.
Phoenix, AZ 85040

PROJECT: ACS Project #2202066
LOCATION: Peoria, AZ
MATERIAL: Native
SAMPLE SOURCE: B-3 (0.0-1.0')
SAMPLE PREPARATION: Saturated - .5, 1, and 2ksf

JOB NO: 19-2012-2017
WORK ORDER NO: N/A
LAB NO: 22-1797-02
DATE ASSIGNED: 11/28/2022

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.):	0.989	0.987	0.984
Shearing device used:	Humboldt Automated Shear Test System by Trautwein Soil Testing Equipment		
Rate of deformation (in/min):	0.01	0.01	0.01
Direct shear point:	1	2	3
Dry mass of specimen (g):	121.2	121.0	121.2
Initial Moisture Content:	4.6%	5.0%	5.7%
Initial Wet Density (pcf):	105.0	105.2	106.1
Initial Dry Density (pcf):	100.4	100.2	100.4
Final Moisture Content:	20.7%	17.3%	16.1%
Final Wet Density (pcf):	122.5	119.1	118.5
Final Dry Density (pcf):	101.5	101.6	102.1
Normal Stress (psf):	500	1000	2000
Maximum Shearing Stress (psf):	572	1331	1978
Vertical Deformation @ Max Shear (in):	0.230	0.208	0.204
Horizontal Deformation @ Max Shear (in):	0.458	0.426	0.479



WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

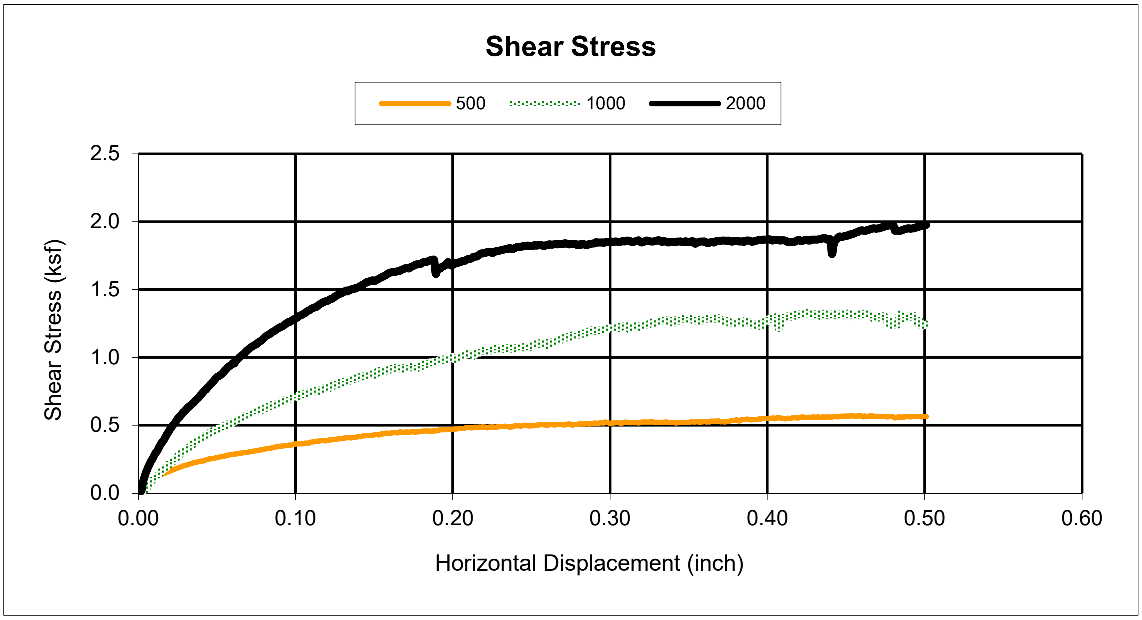
REVIEWED BY Hiram Franco

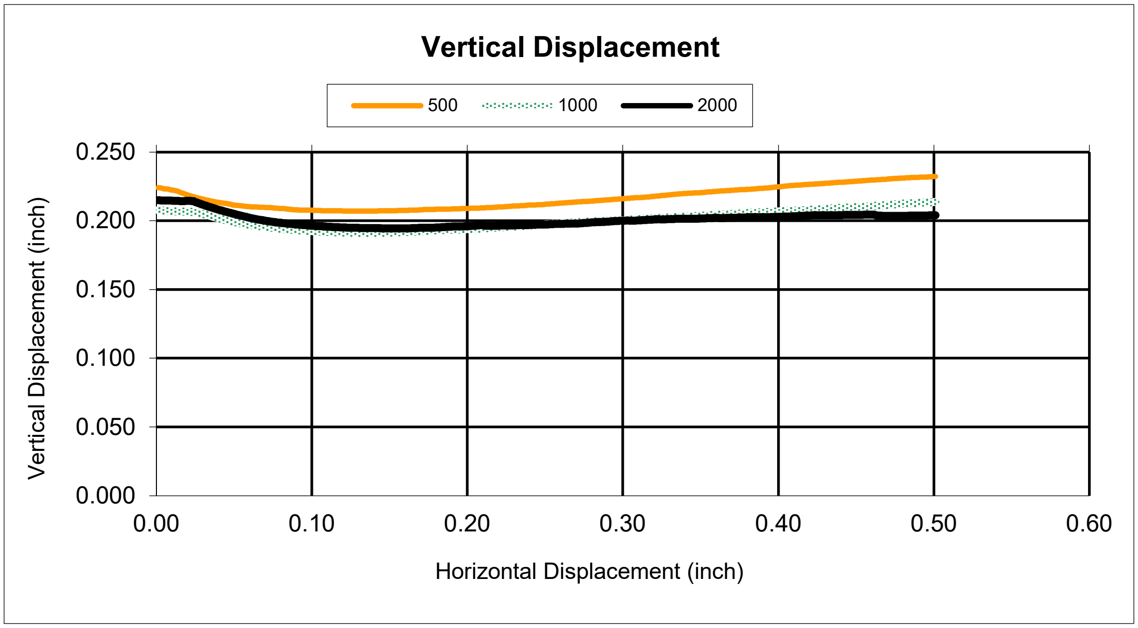
PROJECT: ACS Project #2202066
LOCATION: Peoria, AZ
MATERIAL: Native
SAMPLE SOURCE: B-3 (0.0-1.0')
SAMPLE PREPARATION: Saturated - .5, 1, and 2ksf

JOB NO: 19-2012-2017
WORK ORDER NO: N/A
LAB NO: 22-1797-02
DATE ASSIGNED: 11/28/2022

NORMAL LOADS (psf): 500 1000 2000

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





WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

ACS Services LLC

Soil pH and Resistivity Determination

AASHTO T-289 AASHTO T-288 / ARIZ 236

Project #	2202066	Material Type:	Soil
Lab #	22-5002-1	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	-
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Sampled By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Test Date:	Friday, December 2, 2022
Project City:	Peoria	Tested By:	Fernando Montero
Sample Source:	B-1 @ 0.5 - 5	Resistivity Box:	-
		Reviewed By:	Dylan Ward

pH Reading = 8.29

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 mL	6.77	147	1	995
50	6.77	112	1	758
50	6.77	110	1	745
50	6.77	104	1	704
50	6.77	106	1	718

Fernando Montero
Lab Supervisor

Dylan Ward
Laboratory Manager

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

Project #	2202066	Material Type:	Soil
Lab #	22-5002-7	Supplier:	-
Client:	Ethos Engineering, LLC	Sample Date:	-
Project Name:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Sampled By:	Client
Project Address:	Stadium Trail, 75th Avenue to Skunk Creek (Phase II)	Test Date:	Friday, December 2, 2022
Project City:	Peoria	Tested By:	Fernando Montero
Sample Source:	B-2 @ 0.5 - 5	Resistivity Box:	-
		Reviewed By:	Dylan Ward

pH Reading = 9.48

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 mL	6.77	603	1	4082
50	6.77	371	1	2512
50	6.77	301	1	2038
50	6.77	297	1	2011
50	6.77	302	1	2045

Fernando Montero
Lab Supervisor

Dylan Ward
Laboratory Manager

MOTZZ LABORATORY, INC.

Report: 944702
Reported: 12/8/2022
Received: 12/6/2022
PO: 2202066

Laboratory Analysis Report

ACS Services LLC
Dylan Ward
2235 W Broadway Road
Mesa, AZ 85202

Project: 2202066

Lab Number	Sample ID	
944702-1	22-5002-1	B-1 (0.5-5)

Test Parameter

Test	*Method*	*Result*	*Units*
Sulfate	ARIZ 733b	3	ppm
Chloride	ARIZ 736b	198	ppm

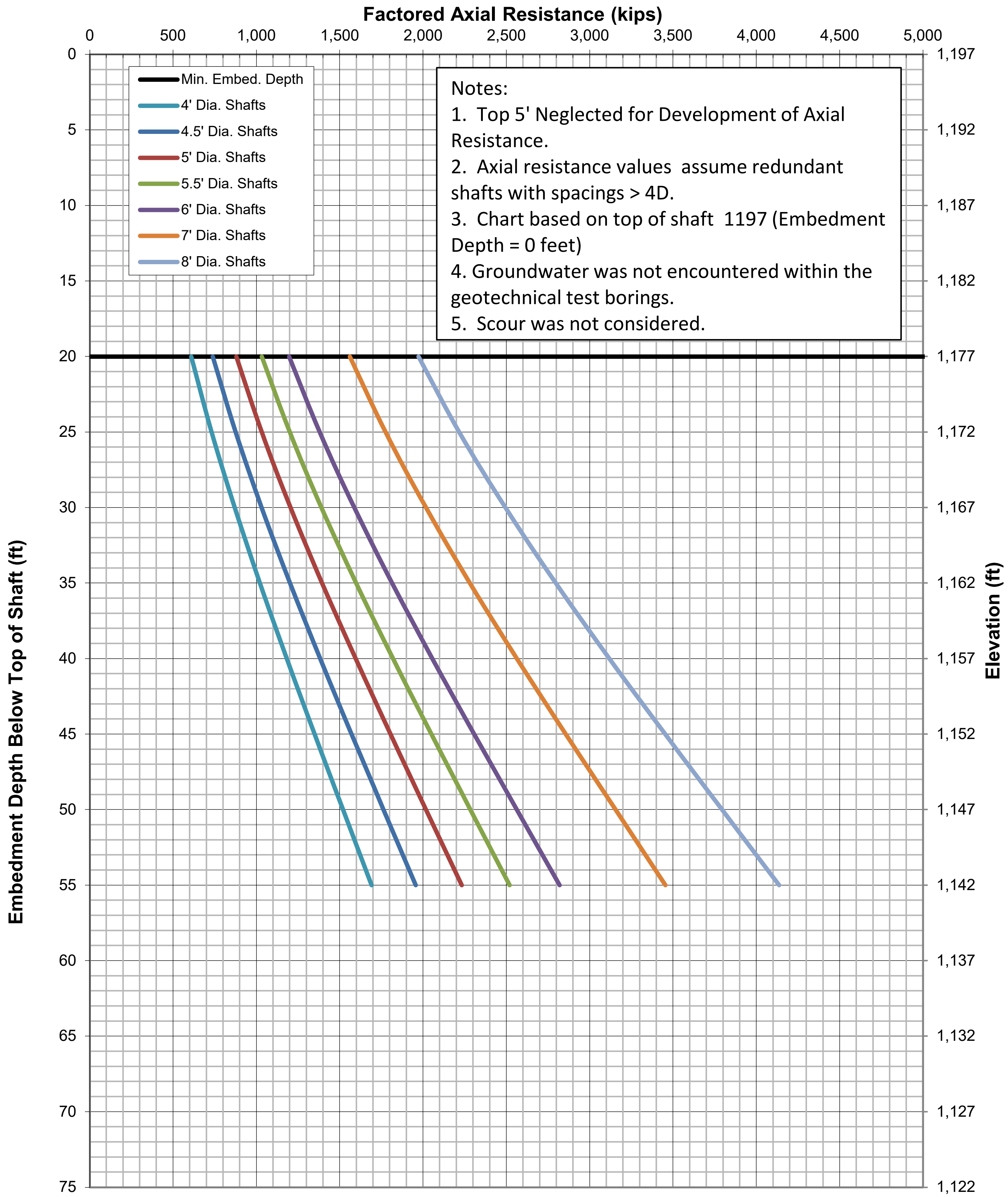
Lab Number	Sample ID	
944702-2	22-5002-7	B-2 (0.5-5)

Test Parameter

Test	*Method*	*Result*	*Units*
Sulfate	ARIZ 733b	4	ppm
Chloride	ARIZ 736b	19	ppm

APPENDIX C

Drilled Shaft Axial Resistance Charts



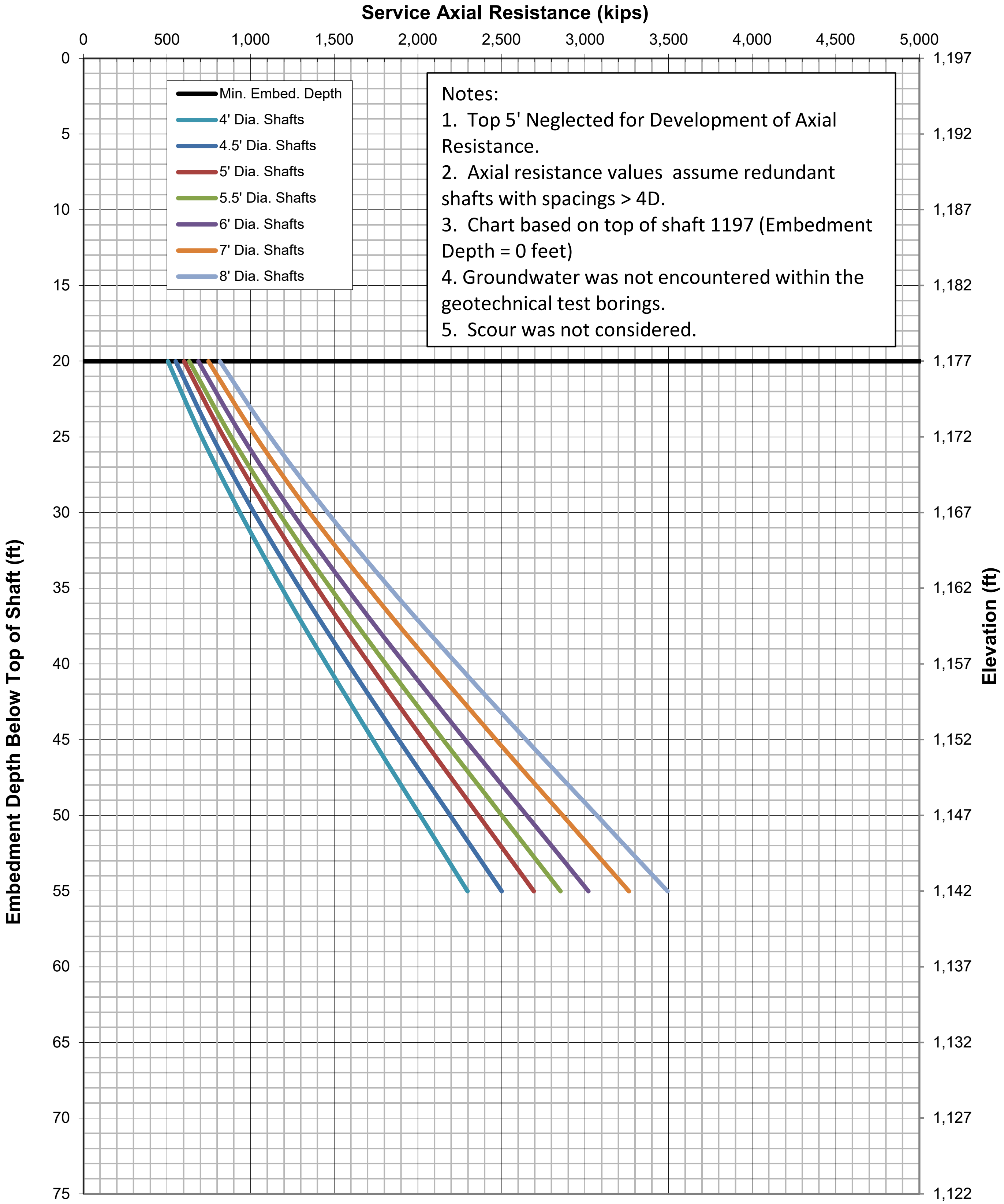
ethos ENGINEERING, LLC.

Designer:	Date:
P. Garza	12/30/2022

DRILLED SHAFT FOUNDATION DESIGN CHART
Strength Limit Axial Resistance in Kips

Stadium Trail, 75th Avenue to Skunk Creek (Phase 2)
Pedestrian Bridge

Figure
C1



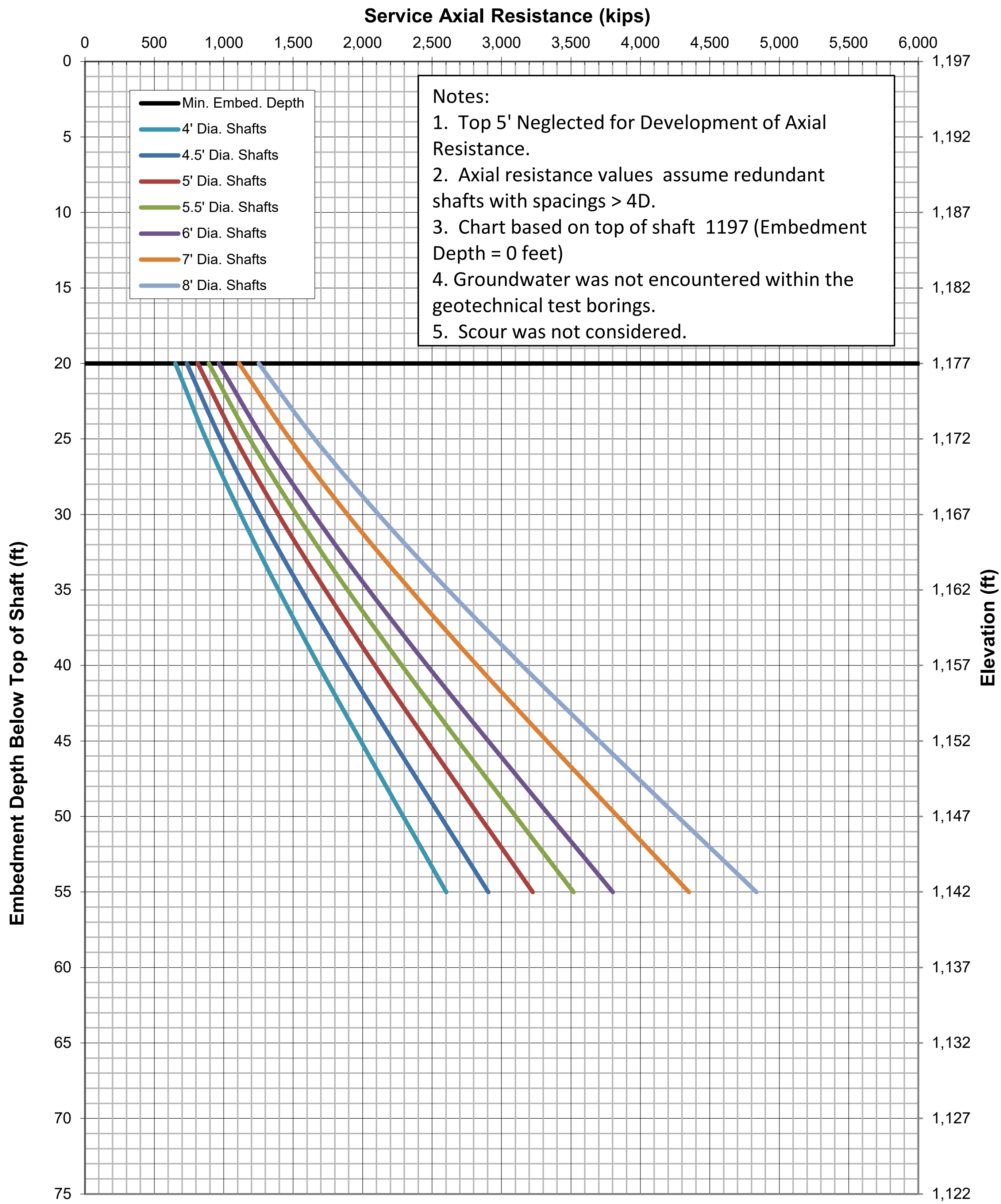
ethos ENGINEERING, LLC.

Designer:	Date:
P. Garza	12/30/2022

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

Stadium Trail, 75th Avenue to Skunk Creek (Phase 2)
Pedestrian Bridge

Figure
C2



ethos ENGINEERING, LLC.

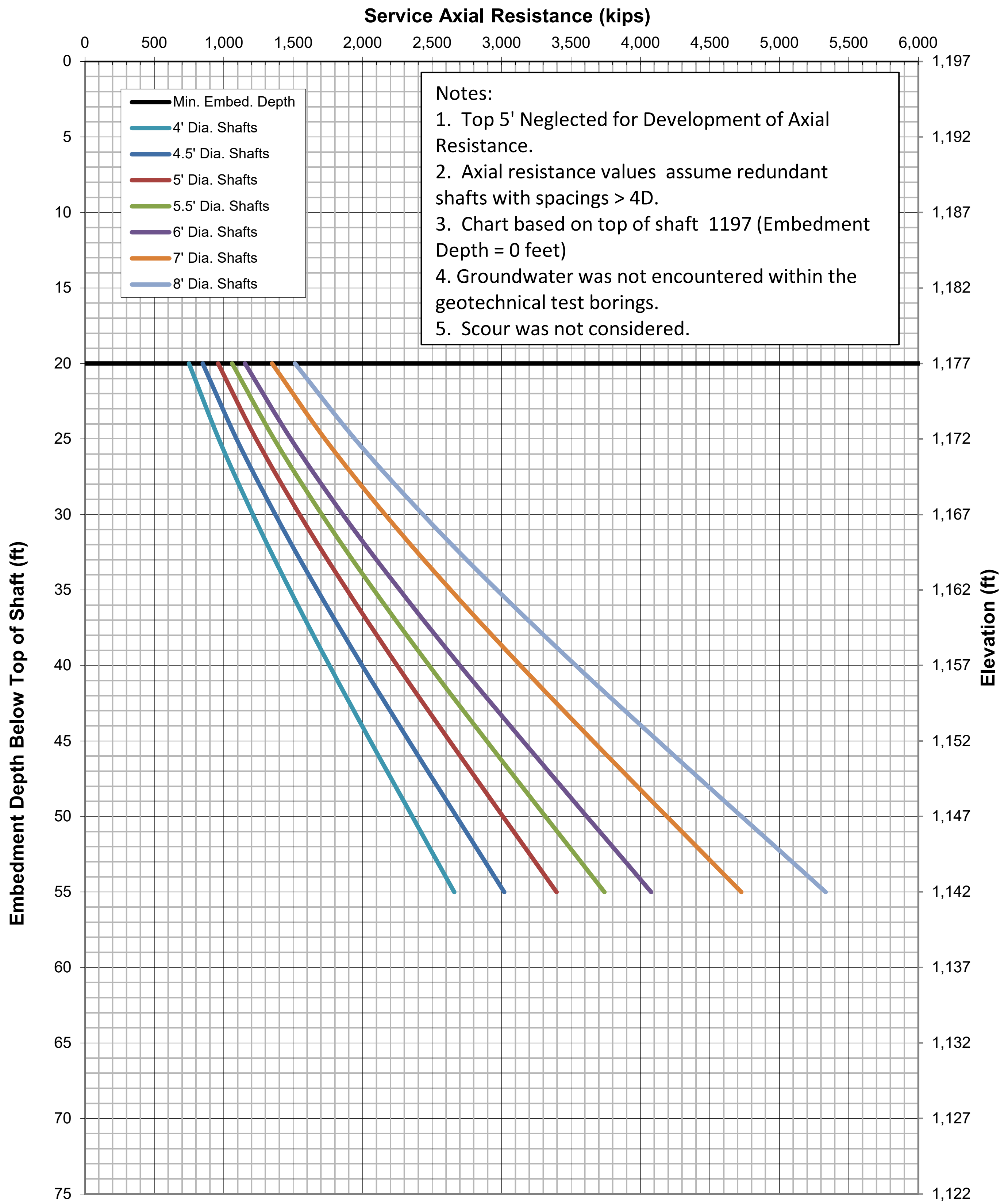
Designer:	Date:
P. Garza	12/30/2022

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

Stadium Trail, 75th Avenue to Skunk Creek (Phase 2)
Pedestrian Bridge

Figure

C3



ethos ENGINEERING, LLC.

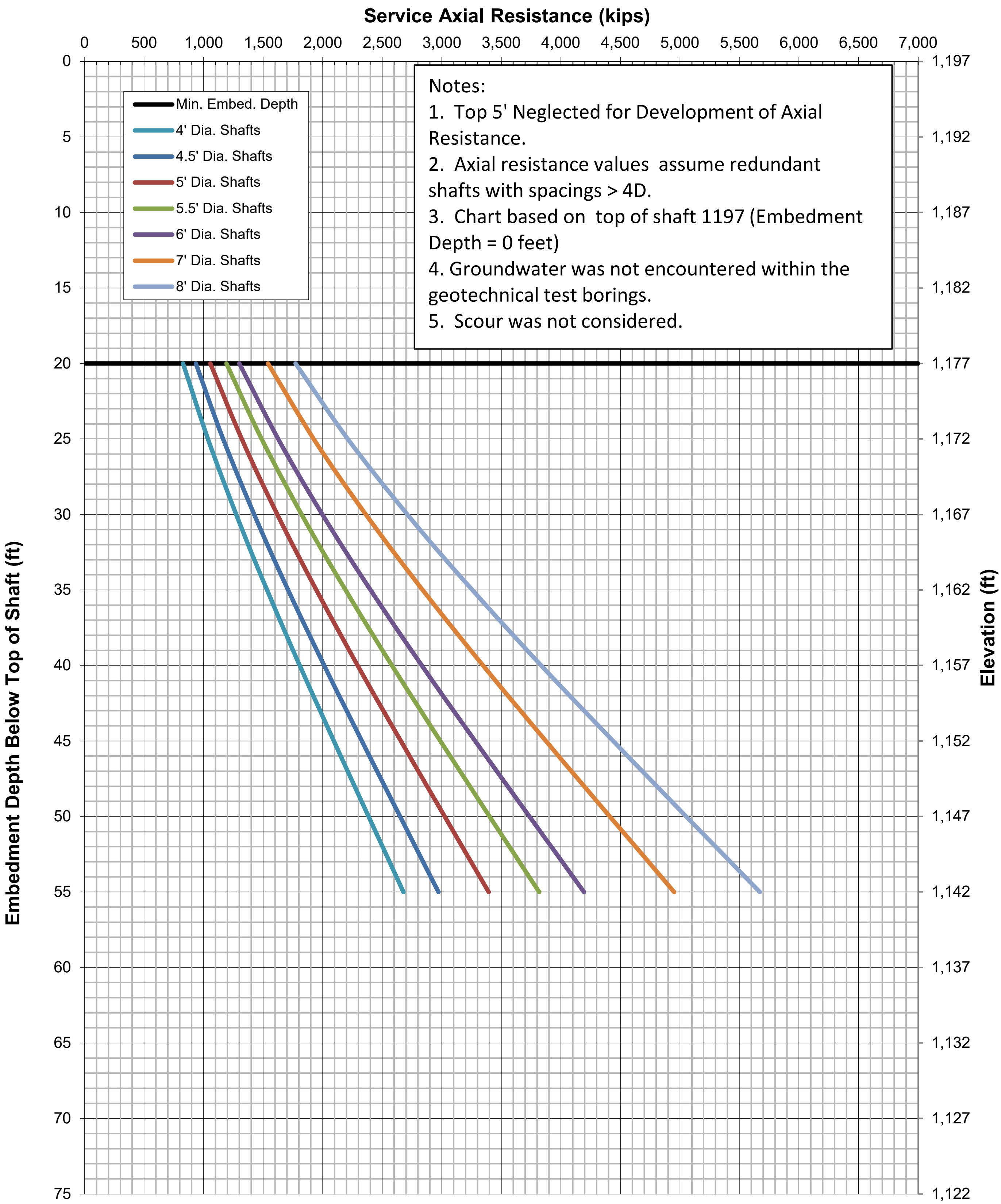
Designer:	Date:
P. Garza	12/30/2022

DRILLED SHAFT FOUNDATION DESIGN CHART
Service Limit (0.75 Inch Settlement) - Axial Resistance in Kips

Stadium Trail, 75th Avenue to Skunk Creek (Phase 2)
Pedestrian Bridge

Figure

C4



ethos ENGINEERING, LLC.

Designer:	Date:
P. Garza	12/30/2022

DRILLED SHAFT FOUNDATION DESIGN CHART
Service Limit (1 Inch Settlement) - Axial Resistance in Kips

Stadium Trail, 75th Avenue to Skunk Creek (Phase 2)
Pedestrian Bridge

Figure
C5

APPENDIX D

Factored Bearing Resistance Chart

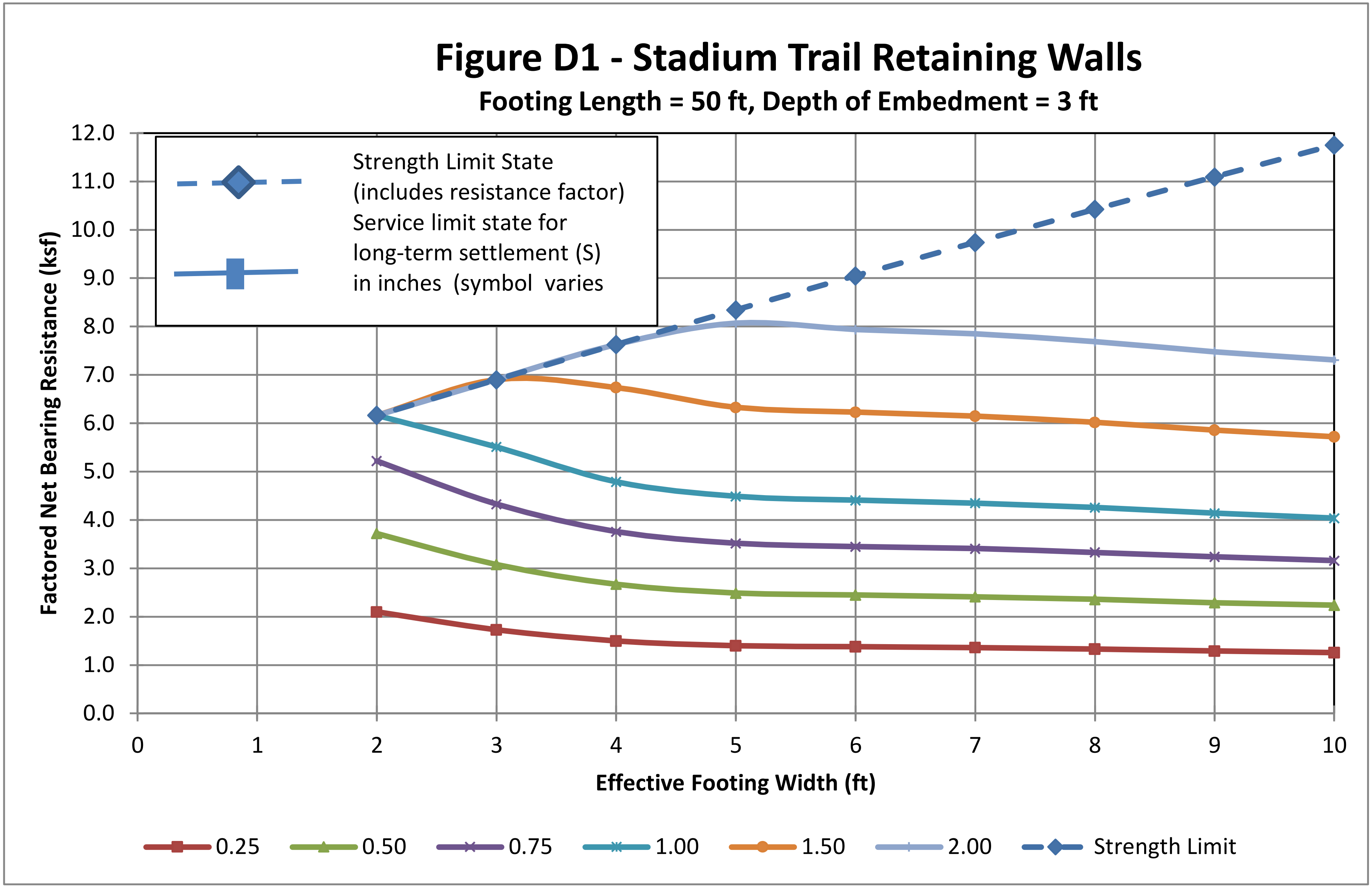


Figure D1 - Stadium Trail Retaining Walls
Footing Length = 50 ft, Depth of Embedment = 3 ft
Strength Limit State
(includes resistance factor)
Service limit state for
long-term settlement (S)
in inches (symbol varies
Factored Net Bearing Resistance (ksf)
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
1
2
3
4
5
6
7
8
9
10
Effective Footing Width (ft)
0.25
0.50
0.75
1.00
1.50
2.00
Strength Limit