FINAL GEOTECHNICAL EXPLORATION REPORT
96TH AVENUE PEDESTRIAN BRIDGE OVER WEST VAN BUREN STREET
CITY OF TOLLESON PROJECT NO. ENG 24-07
TOLLESON, ARIZONA

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Ethos Project No. 2025007
April 21, 2026

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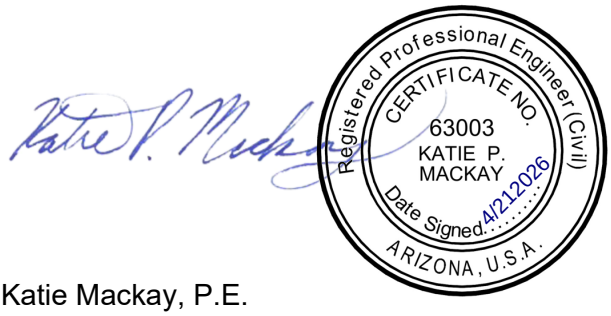
SUBJECT: Final Geotechnical Exploration Report
96th Avenue Pedestrian Bridge over West Van Buren Street
City of Tolleson Project No. ENG 24-07
Tolleson, Arizona

Dear Charlie,

Ethos Engineering LLC is pleased to present the findings of the geotechnical exploration for the proposed pedestrian bridge to span over West Van Buren Street just east of 96th Avenue in Tolleson, Arizona. Our services were conducted in general accordance with the scope of services presented in our proposal, October 17, 2024. This report provides the results of our investigation for foundation support for the proposed new single-span bridge, elevators, mechanical rooms, and staircases. Also included are recommendations for subgrade preparation, grading, 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



Katie Mackay, P.E.
Project Geotechnical Engineer

Reviewed By:



Keith Dahlen, P.E
President/Senior Geotechnical Engineer

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1.0 PROJECT DESCRIPTION

This project will consist of constructing a new pedestrian bridge over West Van Buren Street on the east side of 96th Avenue for the City of Tolleson. A single-span bridge with a length of approximately 115 feet is planned. Access to the elevated structure will be via elevators and staircases at both the north and south ends. Minor surface grading beyond the limits of the proposed improvements will be included. A vicinity map of the project is presented as Figure 1. Please refer to the project plans prepared by Kimley Horn & Associates, Inc. (KHA) for further details regarding planned construction.

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 foundations for the planned improvements.

2.0 FIELD EXPLORATION

Drilling of the exploratory borings was performed by Geomechanics Southwest, Inc. (GSI) on October 8 and October 9, 2025 using a truck-mounted CME-75 drill rig equipped with 8-inch outside diameter (OD) hollow-stem auger (HSA). The field work was supervised by Ramiro Bohon, E.I.T., staff engineer with 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 depths of 24.4 to 75.5 feet below existing site grades. A summary of the field exploration program is presented in Table 2.1 and the locations of the completed test locations are shown in Figure 2.

Table 2.1 – Field Exploration Program

Location ID	Purpose	Latitude (ºN)	Longitude (ºE)	Depth (feet)
B-1	North Abutment	33.42006	-112.07036	75.5
B-2	South Abutment	33.42003	-112.07004	74.3
B-3	South Abutment Approach	33.41999	-112.06965	24.4

Sampling was performed using a standard penetration test (SPT) split spoon sampler with a 1.375-inch ID and 2-inch 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.

The recovered soil samples were removed from the sampler, sealed to reduce moisture loss, and stored for subsequent review and laboratory testing. Upon completion, the borings were backfilled with soil cuttings. Borings B-2 and B-3 were completed within a paved parking area and were thus patched with cold patch asphalt concrete (AC).

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) and Ethos guidelines. Field direction and logging of borings were performed by Ethos personnel. Logs of the borings are presented in Appendix A.

3.0 LABORATORY TESTING

Selected laboratory tests were assigned by Ethos and performed by ACS Services. 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 completed 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	8	Soil Classification
Atterberg Limits (ASTM D4318)	Bulk/SPT	8	Soil Classification
Moisture Content (ASTM D2216)	SPT/Ring	3	Soil Moisture Conditions
Density of Rings (ASTM D7263)	Ring	3	Soil Density & Moisture Content
Direct Shear (ASTM D3080)	Ring	2	Soil Strength Parameters
pH & Resistivity (ARIZ 236)	Bulk/SPT	2	Soil Corrosion Potential
Sulfates and Chlorides (ARIZ 733/736)	Bulk/SPT	2	Soil Degradation Potential

The “B-2 at 10 feet” ring sample for the 1,000 psf load direct shear test is believed to have had gravel in the sample which resulted in questionable test results. Therefore, only the test results for the 2,000 psf and 4,000 psf loads were used to interpret soil strength parameters.

4.0 GENERAL SITE CONDITIONS

4.1 SURFACE CONDITIONS

The location of the planned pedestrian bridge is developed land. The new bridge will cross over Van Buren Street which is paved with asphalt concrete. The north end of the bridge currently includes an asphalt concrete (AC) drive, Portland cement concrete, and a depressed stormwater retention area with grass. The improvements are associated with a Sonic fast-food restaurant. The south end of the bridge includes a school paved AC parking lot, concrete sidewalk, and landscaping with trees and gravel.

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 alluvial deposition of the sediments in the newly-formed basins. The entirety of this site is located within the alluvial basin as generally confirmed by the field investigation.

Published geologic mapping indicates that surficial geologic units at the project site include Holocene-age surficial deposits consisting of unconsolidated deposits associated with modern fluvial systems (Richard et al 2000). The surficial deposits consist primarily of fine-grained, poorly graded sediment on alluvial plains, but also includes gravelly channel, terrace, and alluvial fan deposits on middle and upper piedmonts. The conditions encountered in the borings are consistent with the mapped geology.

4.2.1 Ground Subsidence, Earth Fissures and Groundwater

The project site lies in an area subjected to groundwater withdrawal (50 to 100 feet) due mainly to irrigation demands throughout the 1900's that has caused land subsidence (Schumann & Genualdi, 1986). Earth fissures, known to have been caused by land subsidence, are present in the region, however the closest reported fissure is approximately 7 miles to the northwest located near Camelback Road and North Dysart Road, and the nearest confirmed fissures are located about 7.2 miles to the northwest near Glendale Avenue and North Dysart Road. No earth fissures

are known to have been mapped within the project limits upon review of fissure maps provided by the Arizona Geological Survey (AZGS) (AZGS, 2024).

The site is shown to be in the southern portion of the West Valley Land Subsidence Feature. Ground subsidence of the site has been measured to range from 0.0 to 0.4 inches by the Arizona Department of Water Resources (ADWR) (ADWR, 2024) over the last decade. Neither earth fissures nor ground subsidence should impact this project.

Groundwater was not encountered during the explorations. The depth to groundwater measured in four wells located within one mile of the site varied from approximately 103 feet to 165 feet below grade, based on information provided in the Arizona Department of Water Resources (ADWR) Groundwater Site Inventory (Local ID: A-01-01 09BBB2). Groundwater is not expected to impact this project.

4.3 SITE SUBSURFACE CONDITIONS

The subsurface profile generally consisted of very soft to firm, high to medium plasticity sandy clay (CH, CL) in the upper 15 feet which was underlain by medium dense to very dense sand (SP) to about 30 feet. The sand layer was underlain by a 10-foot layer of firm to hard silt with sand (ML) with low plasticity which was further underlain by medium firm to hard clay and clayey gravel (CL, GC) with high plasticity to about 70 feet. Below 70 feet, the soil consisted of sand and clay with varying densities and plasticity to the maximum depth explored of 75.5 feet. Refer to Appendix A for details about the conditions encountered in the borings.

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 (2017) the project site has the Horizontal Spectral Response Acceleration Coefficients with a 7 percent probability of exceedance in 75 years. The probabilistic horizontal spectral acceleration values for the designated return period and corresponding horizontal peak ground acceleration (PGA) were obtained from the United States Geological Survey (USGS) seismic hazards program website (USGS 2002). The values obtained from the website are based on 2009 AASHTO Guide Specifications for 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 33.45048° N, Longitude 112.26592° 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	0.05g	
Spectral Acceleration, S_{DS}	0.179g	Equation 3.10.4.2-3
Spectral Acceleration, S_{D1}	0.092g	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 pedestrian bridge, elevator shafts, mechanical rooms, staircases, site preparations and grading, moisture protection, excavations, and other construction considerations. These recommendations are based on our understanding of the project, 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 6th Edition (AASHTO, 2012), our completed test borings, and laboratory analyses. The axial compression capacities of the drilled-shaft foundations were determined using ADOT's Geotechnical Design Policy DS-1 memorandum (ADOT, 2010a). The ADOT memorandum outlines the development of drilled-shaft axial resistance charts based on methods specified in AASHTO LRFD Bridge Design Specifications (AASHTO, 2010). We also reference the Arizona Department of Transportation (ADOT) Standard Specifications for Road and Bridge Construction (ADOT, 2021) as the majority of municipal transportation departments default to ADOT for drilled shaft and foundation design and construction.

The drilled shaft design charts are presented in Appendix C. Development of strength and service limit charts is further discussed in the following sections. 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 (Section 5.2) are drilled shaft recommendations for the bridge foundations.

Construction should be performed in accordance with the current requirements of the MAG Uniform Standard Specifications and Details for Public Works Construction, 2025 Edition (MAG 2025), or the most currently published edition.

The bearing resistance design charts for shallow footings are presented in Appendix D. Expansive clay soils are present at the project site at and below the level of planned shallow foundations. Within the footing footprints, we recommend that the upper 2 feet below footings be removed and replaced with engineered fill meeting the specifications as outlined in Section 5.2.4. Existing fill beneath the pavement on the south side of the proposed bridge (Boring B-2) consisting of silty, clayey sand appears to meet these specifications. The engineered fill should extend laterally at a 1H:1V ratio from the bottom of footings. Additionally, we recommend that engineered fill extend to a minimum depth of 12 inches below slabs. For the elevator shafts, over-excavation and replacement with engineered fill may be omitted with the approval of an Ethos engineer based on results of a manual probe test of the prepared foundation footprint. Recommendations for spread footings are provided in Sections 5.2.2 through 5.2.5.

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 and drained cohesive soils based on the subsurface profile encountered in the borings. For the beta method analysis, N_{60}-values were limited to 50 (AASHTO 2012).

The axial resistance design charts presented in Appendix C are applicable for non-redundant conditions, assuming only one shaft per abutment. Per AASHTO specifications, the resistance was reduced by 20 percent. The provided design charts in Appendix C can be used for redundant conditions by decreasing the applied loads by the same factor of 20 percent.

The following sections provide design recommendations for strength and service limit states for drilled shaft foundations. 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 is recommended. The strength and service limit axial resistance curves showing the factored axial resistance versus depth for drilled, cast-in-place shaft diameters of 2 to 6 feet are presented in Appendix C.

5.2.1.1.1. Strength Limit State

The Strength limit axial resistance curves showing the factored axial resistance versus depth for drilled, cast-in-place shaft diameters of 2 to 6 feet are presented in Figure C1. 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 0.44 and 0.4 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 non-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. The service limit state charts are presented as Figures C2 through C6.

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 (2017) 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 (2017) 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. Based on the design plans, only single shafts are anticipated. Therefore, no reduction for group effects is anticipated.

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	2*D*	---	0.90
	3*D* or more	---	1.0
Single and Multiple Rows	2*D* or more	Shaft group cap in intimate contact with ground consisting of medium dense or denser soil, and no scour below the shaft cap is anticipated	1.0

5.2.1.2 Downdrag

No significant amounts of fill are planned for this project. Therefore, the effects of downdrag are considered negligible and not worthy of further consideration.

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)	Effective Unit Weight (pcf)	Friction Angle (deg)	Cohesion (psf)	Secant Modulus, ε_{50} (dim)	Horizontal Subgrade Modulus, k (pci)	LPILE Model Type
1	997+	120	--	1,500	0.008	--	Stiff Clay w/o Free Water
2	997 – 985	115	30	--	--	50	Sand
3	985 – 973	120	35	--	--	140	Sand
4	Below 973	120	--	2,900	0.005	--	Stiff Clay w/o Free Water

NOTES:
When the ground in front of the drilled shaft is sloping, the lateral shaft resistance should be ignored to a depth where 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 2017) 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 (2017). Recommendations for P_m are shown in Table 5.3, based on AASHTO Table 10.7.2.4 1 (AASHTO 2017) 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.

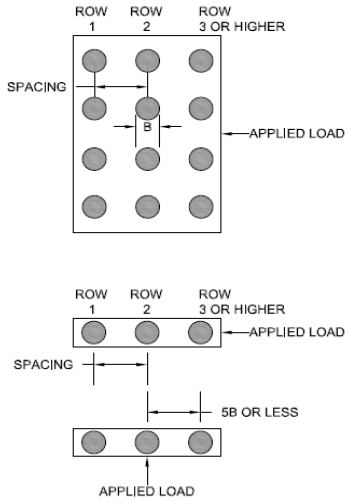
Based on the design plans, only single shafts are anticipated. Therefore, no reduction for group effects are anticipated.

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

The subsurface soils included clean, uncemented, coarse-grained sand and gravel soils with depth. Drilled-shaft excavations will likely encounter caving and/or sloughing conditions in these soils. Casing and/or slurry will be needed to advance drilled shafts. Additionally, very dense sand and gravel and cemented fine-grained soils were also encountered. These soil conditions may require increased excavation effort and/or specialized drilling equipment. 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 Section 609 of the ADOT Standard Specifications (ADOT 2021). A quality control report should be submitted for each shaft, stating that all details were monitored and meet the project requirements.

Straight, drilled shaft excavations will likely be advanced with single-flight-auger or bucket auger bits to the recommended depth. Cleaning of the drilled-shaft 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.

5.2.2 Recommendations for 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, (AASHTO 2018), and ADOT Geotechnical Design Policy SF-1 (2010a). Based on current design information provided by KHA, the staircases, elevator shafts, and mechanical rooms will likely be supported on spread footings bearing within natural ground. The following recommendations can be applied assuming the site grading provisions in Section 5.2.3 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 (AASHTO 2018). The parameters presented below in Table 5.4 were assumed for the nominal resistance and strength limit state analyses. The parameters used, particularly angle of internal friction and cohesion, are considered to be conservative for the soil conditions encountered.

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

Parameter	Symbol	Elevator Shafts	All Other Footings
Soil Angle of Internal Friction, degrees	ϕ_f	23	23
Soil Total Unit Weight, pcf	γ	120	120
Cohesion, psf	c	250	250
Assumed Footing Length, feet	L	11	6
Footing Bearing Depth (Min.), feet	D_f	6	2 to 3
Effective Footing Width, feet	B_f	5 to 20	2 to 15
Bearing Resistance Factor	φ_b	0.45	0.45

The footing lengths and depths were assumed based on information provided by KHA. The resulting factored net bearing resistance, q_{Rn}, versus effective footing width, B', is shown as the "Strength Limit State" line in Figures D1 through D3. Per the ADOT Geotechnical Design Policy SF-1 (2010a), the modified Schmertmann method presented in Section 8.5 of the FHWA Soils

and Foundation Reference Manual (FHWA 2006) 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 (feet)		
		0-15	15-40	Below 40
Soil Type	--	Clay	Sand	Clay
Soil Unit Weight (pcf)	γ	120	115	120
Corrected SPT N-value	N_{60}	13-40	24-68	33-92
Elastic Modulus (ksf)	E_s	$5N_{60}$	$20N_{60}$	$10N_{60}$

The parameters are based on the measured soil densities, distribution of N values and on the E_s-N correlations (FHWA 2006). Figures D1 through D3 present the family of service limit state curves developed per ADOT Geotechnical Design Policy SF-1 (2010a) for design settlements of 0.25, 0.5, 0.75, and 1.0 inches.

In Addition to Figures D1 through D3, Table 5.6 may be used for footing dimensions that do not fit within the dimensions of Figures D1 through D3.

Table 5.6 – Recommended Spread Footing Soil Bearing Pressure

Embedment Depth	Recommended Soil Bearing Pressure (psf) per Allowable Settlement			
	0.25-inch	0.5-inch	0.75-inch	1-inch
Near Surface (1.5 to 3' embedment)	900	1,600	2,300	3,000
3 to 4 feet	1,100	2,000	2,700	3,400
Basement (6 feet min)	1,500	2,500	3,700	4,500

Given the unsaturated support conditions, we anticipate settlements within the upper few feet of native soils to be primarily elastic, occurring during construction. In general, we anticipate total settlements to not exceed 3/4 inch. Differential settlements are anticipated to be no greater than 50 percent of total settlements if proper drainage conditions are maintained.

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 (AASHTO 2017). We recommend the parameters presented in Table 5.7 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 270 pounds per square foot (psf) per foot.

Table 5.7 - 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	0.80
Resistance Factor for Passive Resistance	φ_{ep}	0.50
Nominal Sliding Resistance		
Soil Angle of Internal Friction	ϕ_f	23 degrees
Soil Total Unit Weight	γ	120 pcf
Cohesion	C	250 psf
Shear Resistance Between Soil and Foundation	δ	23 deg = ϕ_f
Passive Earth Pressure Coefficient	K_p	2.25

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 Below Ground Retaining Walls

Retaining walls are not anticipated above existing grade. Based on the structural drawings, the elevator shafts and machine rooms will need retaining walls below grade. 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 current requirements of MAG Section 206. The limits of structure backfill placement are assumed to be the entire limits of excavations for the building footprints, 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 45 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 65 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 in order to develop passive resistance with low strains.

5.2.3 Foundation Subgrade Preparation

As mentioned previously, expansive clay soils are present at the project site. The use of expansive clay soils are not recommended as engineered fill beneath structural portions of the site but could be used in general grading areas. Care should be taken to isolate these clay soils during the grading of the site.

Trash, debris, vegetation (including roots) and other organics, any existing 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 prior to construction. Exposed areas which will receive fill, once properly cleared and benched where necessary, should be scarified to a minimum depth of 10 inches, moisture conditioned, and compacted. Exposed surfaces should be free of mounds and depressions which could prevent uniform compaction.

Within the footing footprints, we recommend that the upper 2 feet below footings be removed and replaced with engineered fill meeting the specifications as outlined in Section 5.2.4. The engineered fill should extend laterally at a 1H:1V ratio from the bottom of footings. For the elevator shafts, over-excavation and replacement with engineered fill may be omitted with the approval of an Ethos engineer based on results of a manual probe test of the prepared foundation footprint.

We recommend that engineered fill extend to a minimum depth of 12 inches below slabs. Subgrade soils which will receive fill for exterior slabs should be scarified, moisture conditioned and compacted to a minimum depth of 10 inches. The moisture content and compaction of subgrade soils should be maintained until slab construction.

5.2.4 Structure Backfill and Engineered Fill

Any engineered fill placed for this project should consist of structure backfill meeting the requirements of Section 206 of the current MAG Standard Specifications. All structure backfill should be moisture conditioned to within minus 1 to plus 3 percent (-1% to +3%) of the optimum moisture content and compacted to a minimum of 95 percent 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 the buildings and foundations.

It appears the on-site clay soils would not meet the requirements for structure backfill, but they may be used in general grading areas. Imported soils for use as engineered fill on the site should conform to the specifications in Table 5.8 for structure backfill:

Table 5.8 – Structure Backfill Specifications

Gradation	Value
4 Inch (% Finer by Weight)	100
No. 4 Sieve (% Finer by Weight)	50 - 90
No. 200 Sieve (% Finer by Weight)	10 - 50
Liquid Limit	30 (max)
Plasticity Index	10 (max)
Maximum Expansive Potential (%)[(1)]	1.0

NOTES:

[(1)] Measured on a sample compacted to approximately 95 percent of the ASTM D698 maximum dry density at about 3 percent below optimum water content. The sample is confined under a 100 psf surcharge and submerged/inundated.

Engineered fill should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended moisture contents and densities throughout the lift. Fill placed on slopes greater than 5:1 (horizontal:vertical) should be benched to allow for the horizontal placement of fill lifts. Fill lifts should not exceed 10 inches loose thickness.

5.2.5 Temporary Slopes

No permanent slopes are planned for this project. Temporary excavations for construction of footings and elevator shafts 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 13 feet are considered to be Type B soils. For excavations less than 13 feet in such soils, Subpart P, Appendix B indicates a maximum allowable unshored slope of 1H:1V for Type B 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.

5.3 SURFACE DRAINAGE

The subsurface in the immediate project vicinity consists of medium to high plasticity, moisture-sensitive clay soils. 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 shallow 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. Where possible, concrete pavement or slabs, or AC should abut with the structures.

5.4 PRELIMINARY SOIL CORROSION OR DEGRADATION POTENTIAL

5.4.1 Metal in Contact with Soil

The corrosion potential of near surface soils on corrugated metal pipes was characterized using laboratory pH and electrical resistivity testing, performed on 2 samples (Borings B-1 and B-2) in accordance with Arizona Test Method 236. The laboratory pH values were 8.56 and 8.61 at Borings B-1 and B-2, respectively. The resistivity values were 400 and 4,120 ohm-centimeters (ohm-cm) at Borings B-1 and B-2, respectively. 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).

5.4.2 Concrete in Contact with Soil

Two samples from the project site were tested for soluble sulfates and chlorides (Arizona Test Method 733 and Arizona Test Method 736) to support design of concrete structures. The results of these laboratory tests are included in Appendix B and summarized in Table B-1.

Total soluble sulfate values ranged from 28 to 136 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). Both samples fall within Exposure Class S0 for water-soluble sulfate (SO_4^{2-}) in soil by percent mass (SO_4 < 0.1% or 1,000 ppm) and are categorized with a severity level of “not applicable” in terms of sulfate exposure. Based on ACI Table 19.3.2.1, “Requirements for Concrete by Exposure Class,” in Section 19.3.2 (ACI 2019), there is no restriction on Portland cement type for concrete structures in contact with these materials.

Chloride values ranged from 13 to 496 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 project 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 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.

6.0 CLOSURE

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

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

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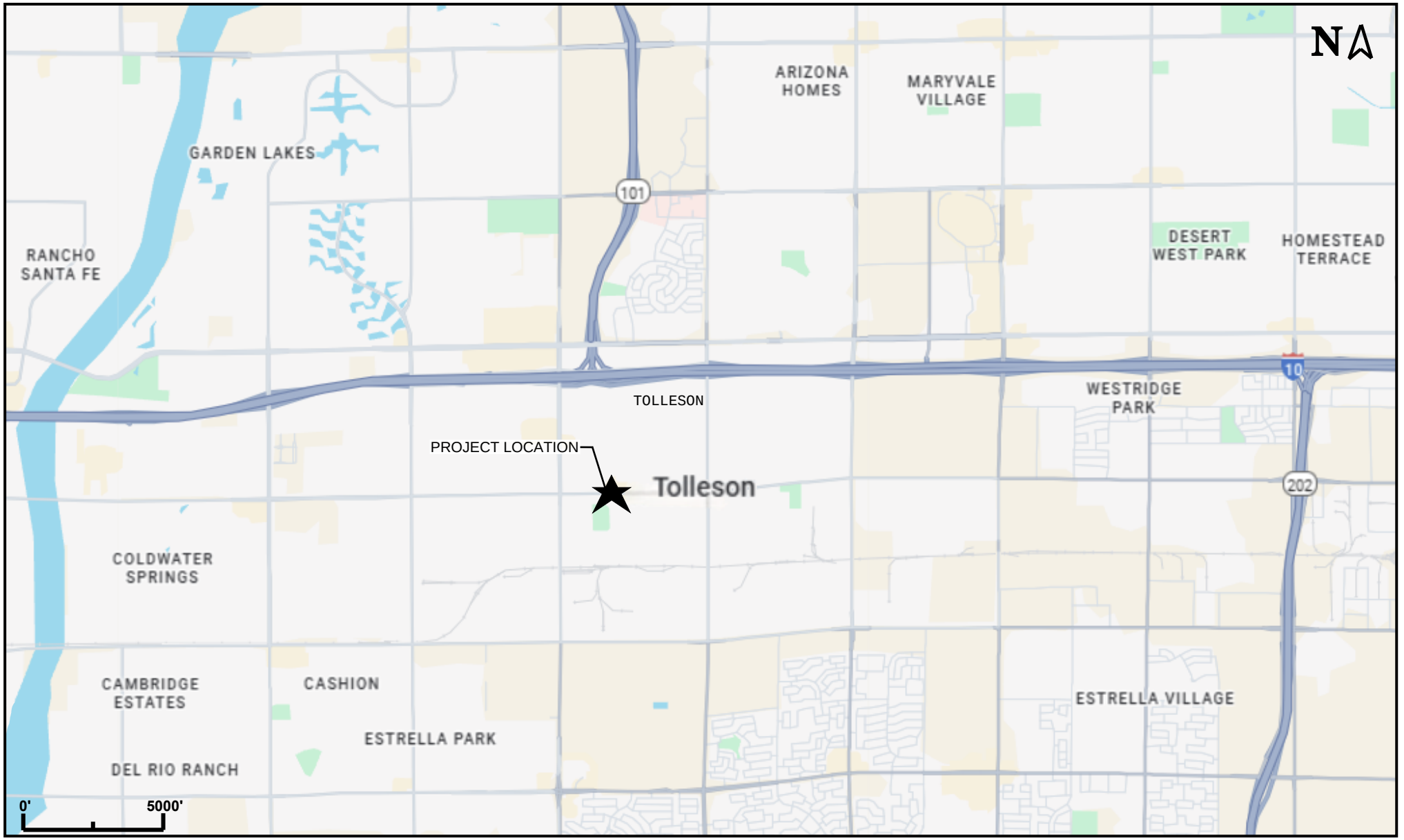
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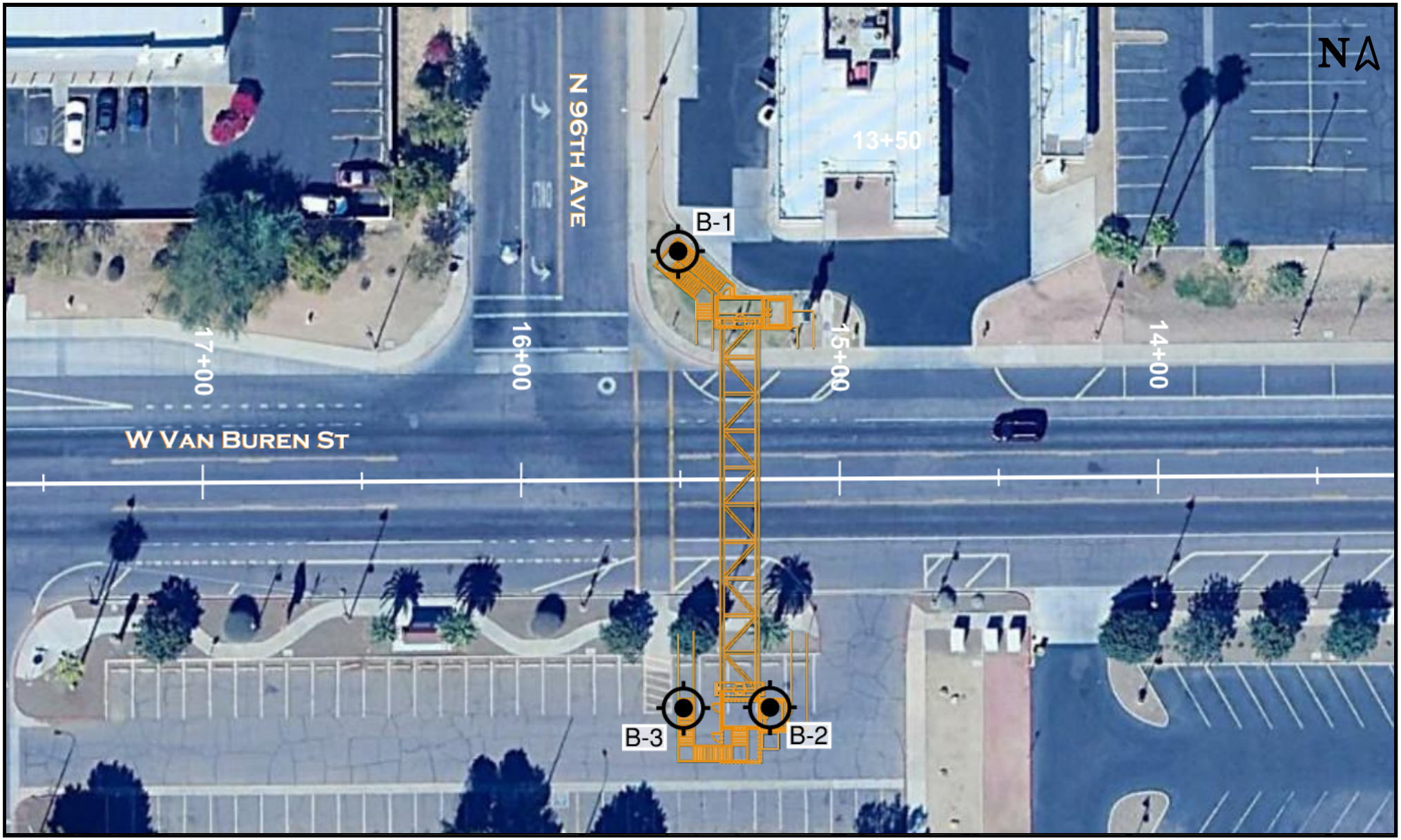
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FIGURES



ethos ENGINEERING, LLC.	**PREPARED BY** Ethos Engineering, LLC Tempe, AZ	**PROJECT** Name: 96th Ave Ped Bridge over Van Buren Number: 2025007	**LOCATION** Van Buren St and 96th Ave Tolleson, AZ	**FIGURE 1**



ethos ENGINEERING, LLC.	**PREPARED BY** Ethos Engineering, LLC Tempe, AZ	**PROJECT** Name: 96th Ave Ped Bridge over Van Buren Number: 2025007	**LOCATION** Van Buren St and 96th Ave Tolleson, AZ	**SYMBOL KEY** Boring Bridge Linework **FIGURE 2**

96th Avenue Pedestrian Bridge over Van Buren Street

USCS PROFILE

LEGEND		USCS	General Description
	=	CH, MH, CL/CH, SC/CH, SM/MH, ML/MH	High PI Silts and Clays
	=	CL, SC/CL, CL-ML	Low to Medium PI Clays
	=	ML, SM/ML	Low to Medium PI Silts
	=	SC-SM, SC	Clayey Sands
	=	SP-SC, SM	Silty Sands and Sands w/ Clay
	=	SP, SP-SM, SW, SW-SM	NP Sands
	=	GC-GM, GC, GC/SC	Clayey Gravels
	=	GP-GC, GM	Silty Gravels and Gravels w/ Clay
	=	GP, GP-GM, GW/SW, GW-GM, GP/SP	NP Gravels

Boring ID	B-1	B-2	B-3
Elevation	1010	1011	1011
Depth (ft)	76	74	24
1015			
1010	CH	SC-SM	CL
1005	CH	CL	CL
1000	CH	CL	CL
995	SP	SP	SP
990	SP	SP	SP
985	GP	SP	SP
980	ML	ML	
975	ML	ML	
970	CL	CL	
965	CL	CL	
960	GC	CL	
955	GC	CL	
950	GC	SP	
945	GC	CL	
940	SP	SM	
935	CL	SM	
930			

N60-VALUE PROFILE

LEGEND		
	=	0 to ≤ 10
	=	10 < to ≤ 30
	=	30 < to ≤ 50
	=	< 50
		Ring

Note: N60-values are corrected for hammer efficiency

Note: Correction factor of (Ring sampler blowcount)*(0.65) = SPT N60-Value used for relative firmness color coding.

Boring ID	B-1	B-2	B-3
Elevation	1010	1011	1011
Depth (ft)	76	74	24
1015			
1010	12	15	11
1005	8	17	20
1000	34	35	51
995	23	26	21
990	74	31	31
985	100	48	100
980	43	81	
975	100	37	
970	100	21	
965	74	55	
960	64	52	
955	41	100	
950	23	43	
945	40	38	
940	100	63	
935	31	100	
930			

FIGURE 3

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.

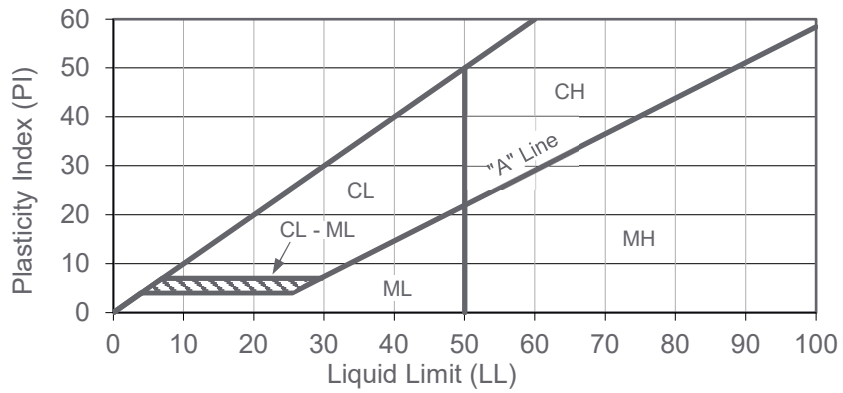
Cohesive Soils - Saturated or Near Saturated			Cohesive Soils – Unsaturated or Cemented Soils			Cohesionless Coarse-Grained Soils		
SPT/ N-Value (blows/ft)	Ring Sampler (blows/ft)	Consistency	SPT/ N-Value (blows/ft)	Ring Sampler (blows/ft)	Firmness	SPT/ N-Value (blows/ft)	Ring Sampler (blows/ft)	Relative 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 ≥ 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 ≤ LL < 50
High	50 ≤ 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: 96th Ave Ped Bridge Over Van Buren		**BORING ID: B-1**
	PROJECT LOCATION: Tolleson, AZ		AGENCY PROJECT NUMBER:
PROJECT NUMBER: 2025007	CLIENT NAME: Kimley Horn & Associates, Inc.		DATE STARTED: 10/09/2025 / DATE COMPLETED: 10/09/2025
DRILLING METHOD(S): Hollow Stem Auger	TOOLING: 6-1/4" ID Hollow Stem Auger		HAMMER TYPE: Auto / HAMMER EFFICIENCY: 92%
SURFACE ELEVATION: 1010'	STATION: ---	OFFSET AND OFFSET DIRECTION: --- ---	-
DRILLING FIRM: Geomechanincs Southwest Inc.	DRILLER: Tyler	FIELD ENGINEER: R. Bohon	RIG TYPE: CME-75 / RIG NUMBER: 109
REPORTED DEPTH: 75.5'	GROUNDWATER DEPTH: N/A	REVIEWED BY: K. Mackay	LATITUDE: 33.45048 / LONGITUDE: -112.26592

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 (%)
			CLAY WITH SAND (CH), some fine subrounded sand, rare fine subrounded gravel, high plasticity, brown, moist, soft, moderate reaction with HCl			3-4-4 (8)	14.1			83.0
									52-20-3 2	
1005	5					5-4-3 (7)				
			Note: Increased sand at 5 feet			3-2-3 (5)				
1000	10									
						9-10-12 (22)	14.3		58-20-3 8	59.0
			13.0							
995	15		**SAND** (SP), occasional fine subangular to subrounded gravel, predominantly fine to medium subangular sand, nonplastic, light brown, moist, medium dense			7-6-9 (15)				
990	20									
						10-19-29 (48)				
985	25		25.0							
			GRAVEL WITH SAND (GP), some to considerable fine to coarse subangular sand, fine to coarse subangular to subrounded gravel, nonplastic, light brown to gray, slightly moist, very dense			50/5"				
			28.0							
980	30		**SILT WITH SAND** (ML), some fine sand, rare fine subangular to subrounded gravel, low plasticity, light brown, moist, firm, moderate reaction with HCl			14-29	24.8	99.0	29-24-5	81.0
975	35									
			Note: Hard below 35 feet			50/5"				
			37.0							
			SANDY CLAY (CL), rare fine subangular to subrounded gravel, weak to moderate lime cementation, medium plasticity, brown, moist, very firm to hard							

ethos ENGINEERING, LLC.	PROJECT NAME: 96th Ave Ped Bridge Over Van Buren		BORING ID: B-1	
	PROJECT LOCATION: Tolleson, AZ		AGENCY PROJECT NUMBER:	
PROJECT NUMBER: 2025007	CLIENT NAME: Kimley Horn & Associates, Inc.		DATE STARTED: 10/09/2025	DATE COMPLETED: 10/09/2025
DRILLING METHOD(S): Hollow Stem Auger	TOOLING: 6-1/4" ID Hollow Stem Auger		HAMMER TYPE: Auto	HAMMER EFFICIENCY: 92%
SURFACE ELEVATION: 1010'	STATION: ---	OFFSET AND OFFSET DIRECTION: --- ---		-
DRILLING FIRM: Geomechanincs Southwest Inc.	DRILLER: Tyler	FIELD ENGINEER: R. Bohon	RIG TYPE: CME-75	RIG NUMBER: 109
REPORTED DEPTH: 75.5'	GROUNDWATER DEPTH: N/A	REVIEWED BY: K. Mackay	LATITUDE: 33.45048	LONGITUDE: -112.26592

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 (%)
			... continued from previous page			10-27-50/2"				
965	45									
						12-18-30 (48)				
960	50		50.0							
			CLAYEY GRAVEL WITH SAND (GC), considerable fine to coarse sand, fine gravel, weak lime cementation, high plasticity, brown, moist, moderately firm to firm			46-18	17.9	104.0	51-24-27	29.0
955	55									
						15-11-16 (27)				
950	60									
						19-8-7 (15)				
945	65									
						12-13-13 (26)				
940	70		70.0							
			SAND (SP), rare fine subangular gravel, predominantly medium subangular sand, nonplastic, brown, moist, very dense			50/5"				
			73.0							
935	75		**SANDY CLAY** (CL), considerable fine sand, rare fine subangular to subrounded gravel, medium plasticity, brown, moist, firm			4-4-16 (20)				
			75.5							

Auger stopped at 74 feet. Sampler stopped at 75.5 feet.
Backfilled with soil cuttings.

ethos ENGINEERING, LLC.	**PROJECT NAME:** 96th Ave Ped Bridge Over Van Buren		**BORING ID: B-2**
	PROJECT LOCATION: Tolleson, AZ		AGENCY PROJECT NUMBER:
PROJECT NUMBER: 2025007	CLIENT NAME: Kimley Horn & Associates, Inc.		DATE STARTED: 10/08/2025 / DATE COMPLETED: 10/08/2025
DRILLING METHOD(S): Hollow Stem Auger	TOOLING: 6-1/4" ID Hollow Stem Auger		HAMMER TYPE: Auto / HAMMER EFFICIENCY: 92%
SURFACE ELEVATION: 1011'	STATION: ---	OFFSET AND OFFSET DIRECTION: --- ---	-
DRILLING FIRM: Geomechanincs Southwest Inc.	DRILLER: Tyler	FIELD ENGINEER: R. Bohon	RIG TYPE: CME-75 / RIG NUMBER: 109
REPORTED DEPTH: 74.5'	GROUNDWATER DEPTH: N/A	REVIEWED BY: K. Mackay	LATITUDE: 33.45009 / LONGITUDE: -112.26591

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 (%)
1010			**ASPHALT CONCRETE**, 1.75" AC over 3.5" AB 0.4							
			FILL: **SILTY, CLAYEY SAND WITH GRAVEL** (SC-SM), some fine to coarse subrounded gravel, fine to medium subangular sand, low plasticity, brown to gray, slightly moist, loose 2.0			10-6-4 (10)	3.3		25-19-6	16.0
						3-3-5 (8)				
	5		**SANDY CLAY** (CL), rare fine subrounded gravel, some to considerable fine sand, medium plasticity, reddish brown, moist, soft to moderately firm							
1005						3-5-6 (11)				
	10									
1000						12-23			37-17-20	53.0
			12.5							
	15		**SAND** (SP), rare fine subangular to subrounded gravel, predominantly fine sand, nonplastic, light brown to tan, slightly moist, dense							
995						7-8-9 (17)				
	20									
990						8-10-10 (20)				
	25									
985						12-13-18 (31)				
			27.0							
	30		**GRAVEL** (GP), some fine to coarse subangular sand, predominantly fine subangular to subrounded gravel, nonplastic to low plasticity, light brown, slightly moist, hard 29.0							
980			**SILT WITH SAND** (ML), some fine subangular to subrounded sand, rare fine subangular to subrounded gravel, nonplastic to low plasticity, light brown to tan, slightly moist to moist, hard, weak reaction with HCl			19-23-30 (53)				
	35									
975			Note: Firm below 35 feet			7-11-13 (24)				

ethos ENGINEERING, LLC.	PROJECT NAME: 96th Ave Ped Bridge Over Van Buren		BORING ID: B-2	
	PROJECT LOCATION: Tolleson, AZ		AGENCY PROJECT NUMBER:	
PROJECT NUMBER: 2025007	CLIENT NAME: Kimley Horn & Associates, Inc.		DATE STARTED: 10/08/2025	DATE COMPLETED: 10/08/2025
DRILLING METHOD(S): Hollow Stem Auger	TOOLING: 6-1/4" ID Hollow Stem Auger		HAMMER TYPE: Auto	HAMMER EFFICIENCY: 92%
SURFACE ELEVATION: 1011'	STATION: ---	OFFSET AND OFFSET DIRECTION: --- ---		-
DRILLING FIRM: Geomechanincs Southwest Inc.	DRILLER: Tyler	FIELD ENGINEER: R. Bohon	RIG TYPE: CME-75	RIG NUMBER: 109
REPORTED DEPTH: 74.5'	GROUNDWATER DEPTH: N/A	REVIEWED BY: K. Mackay	LATITUDE: 33.45009	LONGITUDE: -112.26591

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 (%)
	40		**CLAY WITH SAND** (CL), considerable fine to medium sand, rare fine subangular to subrounded gravel, medium plasticity, brown, moist, moderately firm, weak reaction with HCl							
970						7-14	26.4	95.4	37-22-15	70.0
	45		Note: Firm below 45 feet			13-15-21 (36)				
965										
	50					8-15-19 (34)				
960										
	55									
955			Note: Hard below 55 feet			9-50/4"				
	60		60.0							
950			**SAND** (SP), rare fine subangular to subrounded gravel, predominantly fine to medium subangular sand, nonplastic, light brown, moist, medium dense			12-13-15 (28)				
	65		65.0							
945			**SANDY CLAY** (CL), rare fine subangular to subrounded gravel, fine clay, medium to high plasticity, brown, moist, firm			19-10-15 (25)				
	70		70.0							
940			**SILTY SAND** (SM), rare fine subangular to subrounded gravel, predominantly fine sand, nonplastic to low plasticity, brown, moist, dense to very dense, weak to moderate reaction with HCl			12-17-24 (41)				
			74.3			50/4"				

Auger stopped at 74 feet. Sampler refused at 74.3 feet.
Backfilled with soil cuttings.

ethos ENGINEERING, LLC.	PROJECT NAME: 96th Ave Ped Bridge Over Van Buren		**BORING ID: B-3**
	PROJECT LOCATION: Tolleson, AZ		AGENCY PROJECT NUMBER:
PROJECT NUMBER: 2025007	CLIENT NAME: Kimley Horn & Associates, Inc.		DATE STARTED: 10/08/2025 / DATE COMPLETED: 10/08/2025
DRILLING METHOD(S): Hollow Stem Auger	TOOLING: 6-1/4" ID Hollow Stem Auger		HAMMER TYPE: Auto / HAMMER EFFICIENCY: 92%
SURFACE ELEVATION: 1011'	STATION: ---	OFFSET AND OFFSET DIRECTION: --- ---	-
DRILLING FIRM: Geomechanincs Southwest Inc.	DRILLER: Tyler	FIELD ENGINEER: R. Bohon	RIG TYPE: CME-75 / RIG NUMBER: 109
REPORTED DEPTH: 24.5'	GROUNDWATER DEPTH: N/A	REVIEWED BY: K. Mackay	LATITUDE: 33.45009 / LONGITUDE: -112.26579

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 (%)
1010			**ASPHALT CONCRETE**, 2 inches AC over 4.5 inches AB 0.5							
			FILL: **SANDY CLAY WITH GRAVEL** (CL), some to considerable fine to coarse subrounded gravel, trace to some predominantly fine to medium subangular sand, medium plasticity, reddish brown, moist, soft, weak reaction with HCl 2.5			7-4-3 (7)				
						4-5-6 (11)				
	5									
1005			**SANDY CLAY** (CL), rare fine subrounded gravel, some fine sand, medium plasticity, reddish brown, slightly moist to moist, moderately firm			9-11			38-17-21	71.0
	10									
1000			Note: Very firm, weak lime cementation at 10 feet			10-14-19 (33)				
			13.0							
	15		**SAND** (SP), rare fine subangular to subrounded gravel, predominantly fine to medium subangular sand, nonplastic, light brown to gray, slightly moist, medium dense, strong reaction with HCl			7-6-8 (14)				
995										
	20									
990						5-8-12 (20)				
			Note: Very dense at 24 feet 24.4			50/5"				

Auger stopped at 24 feet. Sampler refused at 24.4 feet. Backfilled with soil cuttings and capped with cold patch asphalt

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)	Direct Shear (ASTM D3080)	pH (AZ 236)	Resistivity [ohm-cm] (AZ 236)	Sulfates [ppm] (AZ 733)	Chlorides [ppm] (AZ 736)
	Begin	End											
B-1	0.0	5.0	CH	83	52	32	14.1			8.56	400	136	496
B-1	10.0	11.0	CH	59	58	38	14.3						
B-1	30.0	31.0	ML	81	29	5	24.8	99.0					
B-1	50.0	51.0	GC	29	51	27	17.9	104.0					
B-2	0.5	2.0	SC-SM	16	25	6	3.3			8.61	4,120	28	13
B-2	10.0	11.0	CL	53	37	20			X				
B-2	40.0	41.0	CL	70	37	15	26.4	95.4					
B-3	5.0	6.0	CL	71	38	21			X				
		Average		**57.8**	**40.9**	**20.5**	**16.8**	**99.5**	**--**	**8.6**	**2260**	**82**	**255**
		Standard Deviation		**24.2**	**11.7**	**11.7**	**8.4**	**4.3**	**--**	**0.0**	**2630**	**76**	**342**
		Maximum		**83.0**	**58.0**	**38.0**	**26.4**	**104.0**	**--**	**8.6**	**4120**	**136**	**496**
		Minimum		**16.0**	**25.0**	**5.0**	**3.3**	**95.4**	**--**	**8.6**	**400**	**28**	**13**
		Count		**8**	**8**	**8**	**6**	**3**	**2**	**2**	**2**	**2**	**2**

Notes: pcf = pounds per cubic foot; ohm-cm = ohm-centimeters; ppm = parts per million

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2501640	Material Type:	Soils
ACS Lab #	25-4277-1	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	10/10/2025
Project Name:	Ethos Engineering	Sampled By:	Client
Project Address:	96th Ave Pedestrian Bridge over W Van Buren Street	Test Date:	10/13/2025
Project City	Tolleson	Tested By:	Austin Archibald
Sample Location:	B-1 @ 0 -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"	1	99	
1/2"	0	99	
3/8"	0	98	
1/4"	0	98	
#4	0	98	
#8	0	98	
#10	0	97	
#16	0	97	
#30	1	96	
#40	1	95	
#50	2	93	
#100	4	89	
#200	6	83	

Liquid Limit (ASTM D4318)	52
Plastic Limit (ASTM D4318)	20
Plasticity Index (ASTM D4318)	32
Moisture Content (ASTM D2216)	14.1
USCS Soil Classification	CH

Group Name (ASTM D2487)
Fat CLAY with sand

Keagen Mayfield
Laboratory Manager

Keagen Mayfield
Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2501640	Material Type:	Soils
ACS Lab #	25-4277-2	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	10/10/2025
Project Name:	Ethos Engineering	Sampled By:	Client
Project Address:	96th Ave Pedestrian Bridge over W Van Buren Street	Test Date:	10/13/2025
Project City	Tolleson	Tested By:	Austin Archibald
Sample Location:	B-1 @ 10 - 11	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"	0	100	
1/2"	0	100	
3/8"	1	99	
1/4"	0	99	
#4	0	99	
#8	0	99	
#10	0	98	
#16	2	96	
#30	3	94	
#40	3	91	
#50	5	86	
#100	14	72	
#200	12	59	

Liquid Limit (ASTM D4318)	58
Plastic Limit (ASTM D4318)	20
Plasticity Index (ASTM D4318)	38
Moisture Content (ASTM D2216)	14.3
USCS Soil Classification	CH

Group Name (ASTM D2487)
Sandy fat CLAY

Testing sizes reduced from standard minimums due to lack of material

Keagen Mayfield
Laboratory Manager

Keagen Mayfield
Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2501640	Material Type:	Soils
ACS Lab #	25-4277-3	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	10/10/2025
Project Name:	Ethos Engineering	Sampled By:	Client
Project Address:	96th Ave Pedestrian Bridge over W Van Buren Street	Test Date:	10/14/2025
Project City	Tolleson	Tested By:	Austin Archibald
Sample Location:	B-1 @ 30 - 31	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"	0	100	
1/2"	0	100	
3/8"	0	100	
1/4"	0	100	
#4	0	100	
#8	0	100	
#10	0	100	
#16	0	100	
#30	0	99	
#40	0	99	
#50	0	99	
#100	4	95	
#200	14	81	

Liquid Limit (ASTM D4318)	29
Plastic Limit (ASTM D4318)	24
Plasticity Index (ASTM D4318)	5
Moisture Content (ASTM D2216)	0.0
USCS Soil Classification	ML

Group Name (ASTM D2487)
SILT with sand

Keagen Mayfield
Laboratory Manager

Keagen Mayfield
Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2501640	Material Type:	Soils
ACS Lab #	25-4277-4	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	10/10/2025
Project Name:	Ethos Engineering	Sampled By:	Client
Project Address:	96th Ave Pedestrian Bridge over W Van Buren Street	Test Date:	10/14/2025
Project City	Tolleson	Tested By:	Austin Archibald
Sample Location:	B-1 @ 50 - 51	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"	7	93	
1/2"	13	80	
3/8"	7	73	
1/4"	8	64	
#4	5	60	
#8	11	49	
#10	2	47	
#16	4	43	
#30	5	38	
#40	4	35	
#50	1	34	
#100	2	32	
#200	3	29	

Liquid Limit (ASTM D4318)	51
Plastic Limit (ASTM D4318)	24
Plasticity Index (ASTM D4318)	27
Moisture Content (ASTM D2216)	0.0
USCS Soil Classification	GC

Group Name (ASTM D2487)
Clayey GRAVEL with sand

Testing sizes reduced from standard minimums due to lack of material

Keagen Mayfield
Laboratory Manager

Keagen Mayfield
Signature

ACS Services LLC

Laboratory Soil Test Results

ACS PROJECT #	2501640	Material Type:	Soils
ACS Lab #	25-4277-5	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	10/10/2025
Project Name:	Ethos Engineering	Sampled By:	Client
Project Address:	96th Ave Pedestrian Bridge over W Van Buren Street	Test Date:	10/13/2025
Project City	Tolleson	Tested By:	Austin Archibald
Sample Location:	B-2 @ 0.5 - 2	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"	2	98	
3/4"	4	95	
1/2"	5	90	
3/8"	4	85	
1/4"	7	78	
#4	3	75	
#8	15	60	
#10	3	57	
#16	8	49	
#30	12	38	
#40	6	31	
#50	5	26	
#100	7	20	
#200	4	16	

Liquid Limit (ASTM D4318)	25
Plastic Limit (ASTM D4318)	19
Plasticity Index (ASTM D4318)	6
Moisture Content (ASTM D2216)	3.3
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 #	2501640	Material Type:	Soils
ACS Lab #	25-4277-7	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	10/10/2025
Project Name:	Ethos Engineering	Sampled By:	Client
Project Address:	96th Ave Pedestrian Bridge over W Van Buren Street	Test Date:	10/14/2025
Project City	Tolleson	Tested By:	Mahalia Davis
Sample Location:	B-2 @ 40 - 41	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"	2	96	
3/8"	0	96	
1/4"	1	96	
#4	0	95	
#8	1	94	
#10	0	94	
#16	1	93	
#30	3	89	
#40	3	86	
#50	3	84	
#100	6	78	
#200	8	70	

Liquid Limit (ASTM D4318)	37
Plastic Limit (ASTM D4318)	22
Plasticity Index (ASTM D4318)	15
Moisture Content (ASTM D2216)	0.0
USCS Soil Classification	CL

Group Name (ASTM D2487)
Lean CLAY with sand

Keagen Mayfield
Laboratory Manager

Keagen Mayfield
Signature

PROJECT: ACS Services Project#2501640
LOCATION: Mesa, AZ
MATERIAL: See Boring Logs

JOB NO: US-EI-1720234239
WORK ORDER NO: 137
DATE ASSIGNED: 10/14/25

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

PERCENT PASSING BY WEIGHT

				Silt or	SAND								GRAVEL									COBBLES	
				Clay	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-2 @ 10-11	CL	37	20	53	66	80	87	92	96	98	98	99	100	100	100	100	100	100	100	100	100	100	25-1587
B-3 @ 5-6	CL	38	21	71	81	90	93	96	99	99	100	100	100	100	100	100	100	100	100	100	100	100	25-1588

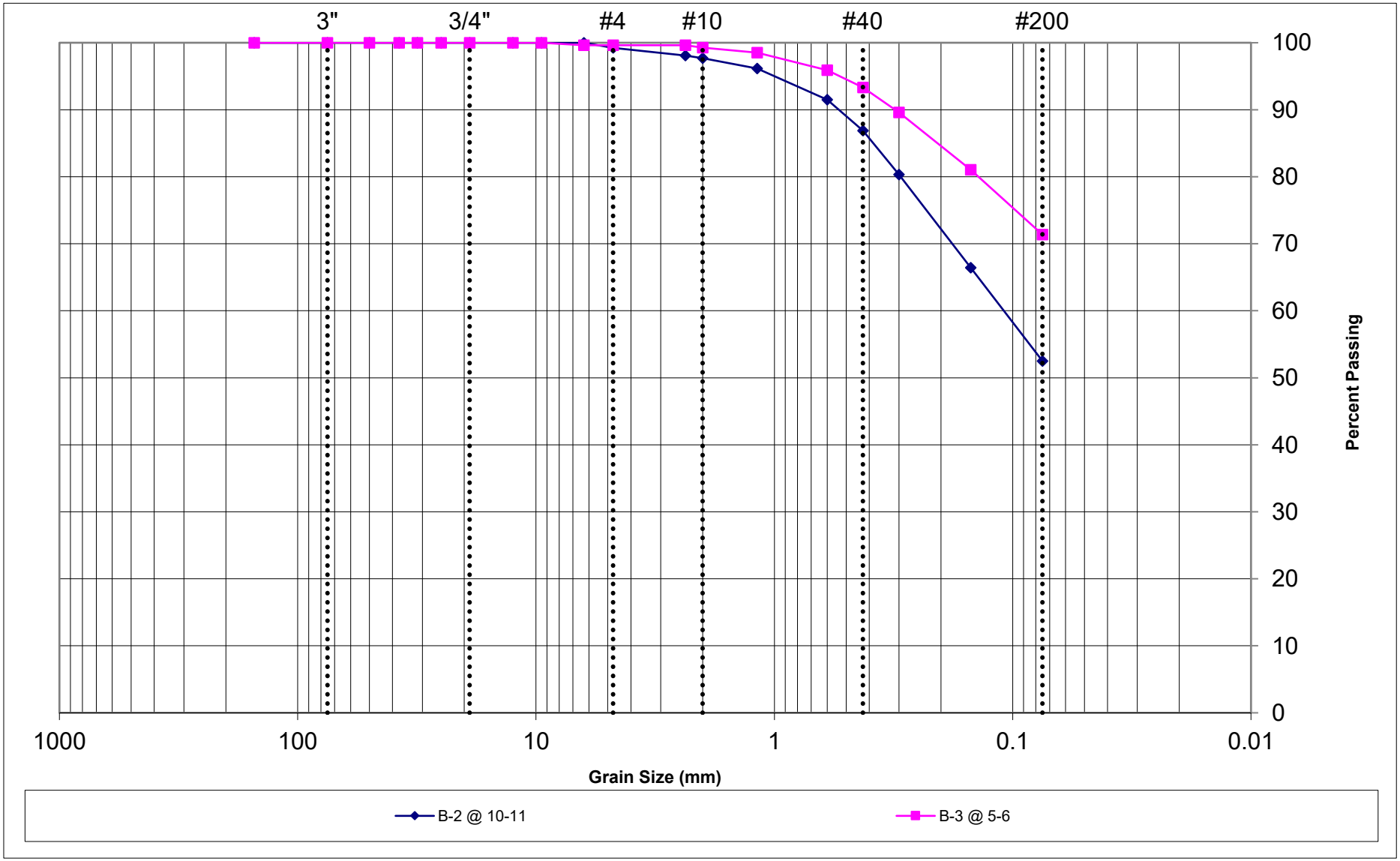
WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

REVIEWED BY _____

PROJECT: ACS Services Project#2501640
LOCATION: Mesa, AZ
MATERIAL: See Boring Logs

JOB NO: US-EI-1720234239
WORK ORDER NO: 137
DATE ASSIGNED: 10/14/25

MECHANICAL SIEVE ANALYSIS GRAPHS



ACS Services LLC

Job #	2501640	Material Type:	Soils
Lab #	25-4277	Extraction Date:	10/10/2025
Client:	Ethos Engineering, LLC	Extracted By:	Client
Project Name:	Ethos Engineering	Laboratory Test Date	10/13/2025
Project Address:	96th Ave Pedestrian Bridge over W Van Buren Street	Laboratory Tested By:	Colin Eggebrecht
Project City:	Tolleson	Reviewed By:	Keagen Mayfield
Material Source:	-		

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

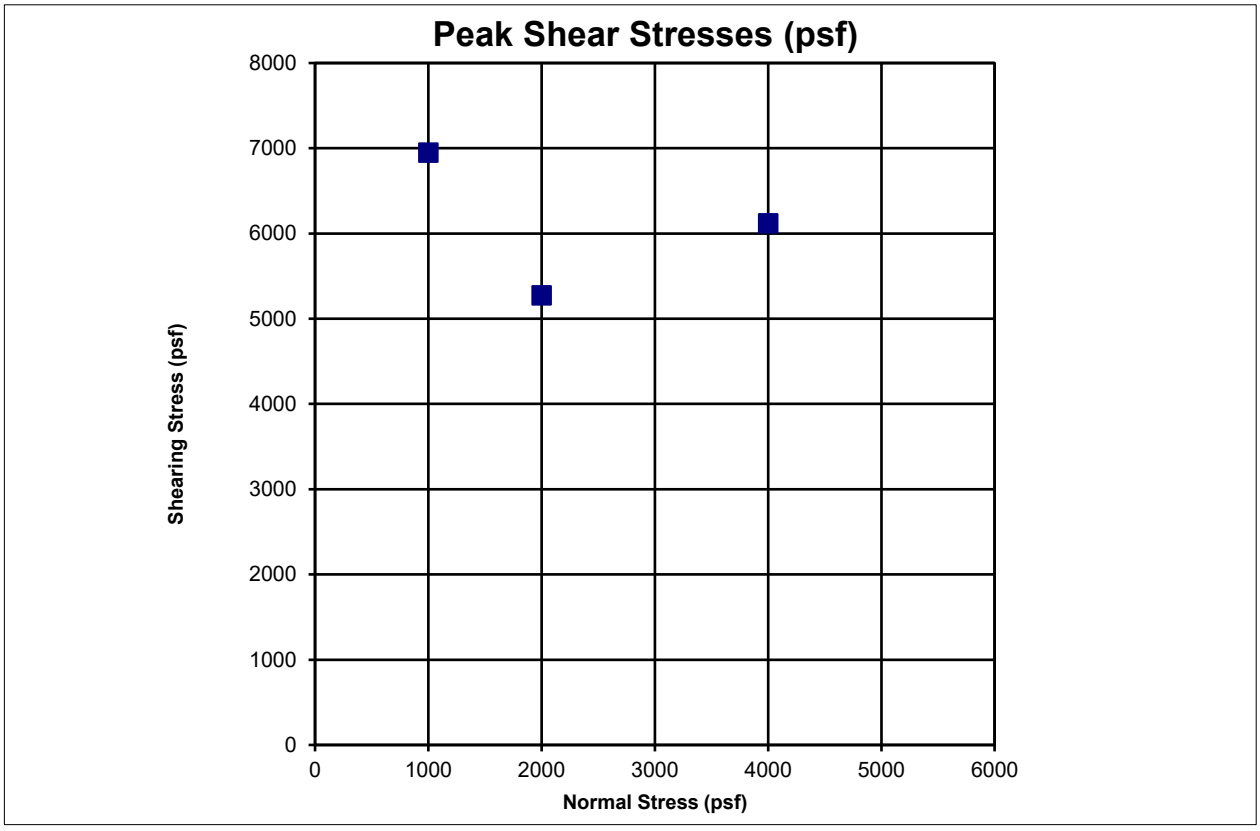
ID #	Sample Location	Wet Wt. (g)	Moisture Dry Wt. (g)	**Moist Content**	# Of Rings	Wet Wt'+Rings (g)	Wt. of Rings (g)	**Dry Density (pcf)**
25-4277-3	B-1 @ 30 - 31	597.3	478.5	24.8%	4	770.0	172.7	99.0
25-4277-4	B-1 @ 50 - 51	444.3	376.9	17.9%	3	578.1	133.8	104.0
25-4277-7	B-2 @ 40 - 41	582.7	461.1	26.4%	4	760.3	177.6	95.4

PROJECT: ACS Services Project#2501640
LOCATION: Mesa, AZ
MATERIAL: See Boring Logs
SAMPLE SOURCE: B-2 @ 10-11 / ACS Lab# 25-4277-6
SAMPLE PREPARATION: Insitu - 1, 2, and 4ksf

JOB NO: US-EI-1720234239
WORK ORDER NO: 137
LAB NO: 25-1587
DATE ASSIGNED: 10/14/2025

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.):	0.998	0.996	0.997
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):	134.7	138.8	139.0
Initial Moisture Content:	10.4%	10.4%	10.5%
Initial Wet Density (pcf):	123.2	127.0	127.2
Initial Dry Density (pcf):	111.6	115.0	115.1
Final Moisture Content:	4.2%	4.3%	5.5%
Final Wet Density (pcf):	116.5	120.4	121.9
Final Dry Density (pcf):	111.8	115.4	115.5
Normal Stress (psf):	1000	2000	4000
Maximum Shearing Stress (psf):	6949	5275	6119
Vertical Deformation @ Max Shear (in):	0.325	0.319	0.288
Horizontal Deformation @ Max Shear (in):	0.501	0.480	0.067



WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

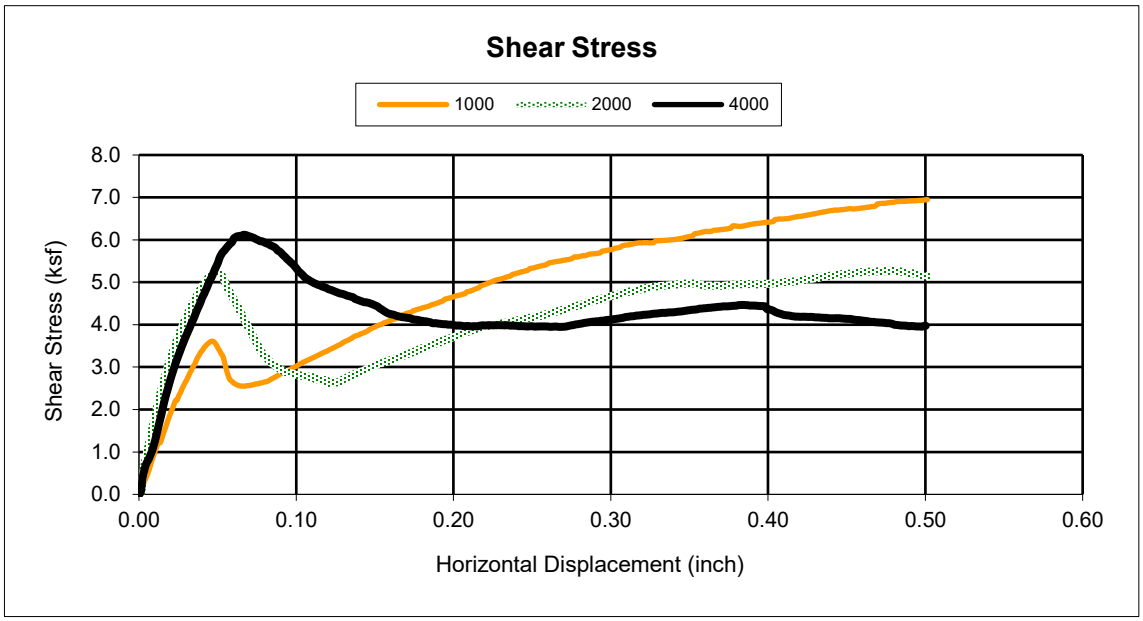
REVIEWED BY _____

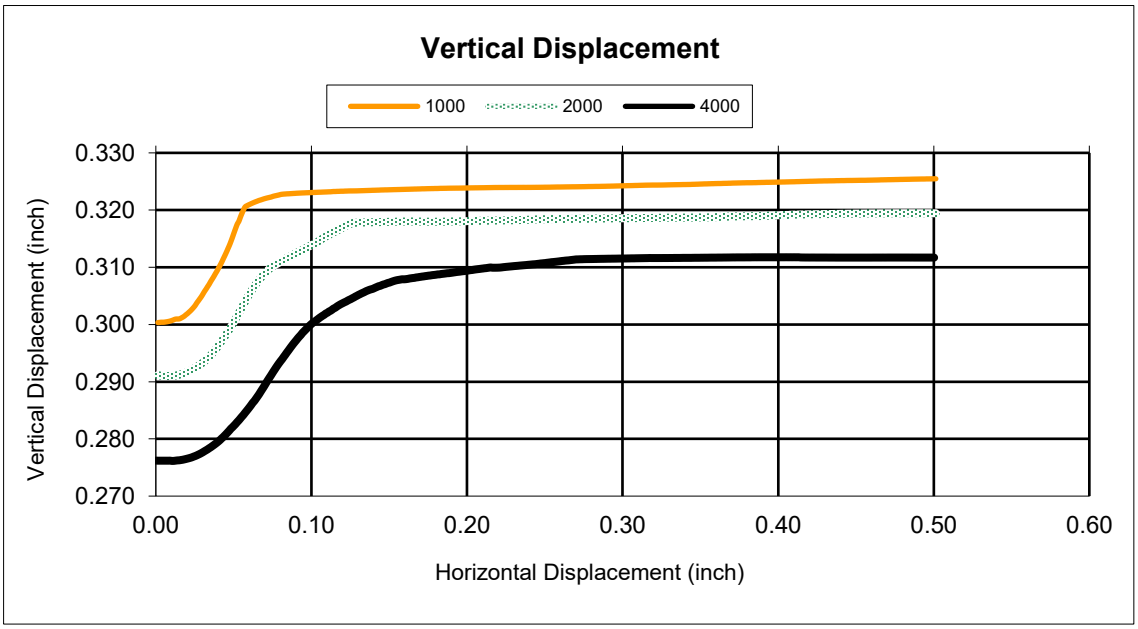
PROJECT: ACS Services Project#2501640
LOCATION: Mesa, AZ
MATERIAL: See Boring Logs
SAMPLE SOURCE: B-2 @ 10-11 / ACS Lab# 25-4277-6
SAMPLE PREPARATION: Insitu - 1, 2, and 4ksf

JOB NO: US-EI-1720234239
WORK ORDER NO: 137
LAB NO: 25-1587
DATE ASSIGNED: 10/14/2025

NORMAL LOADS (psf): 1000 2000 4000

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





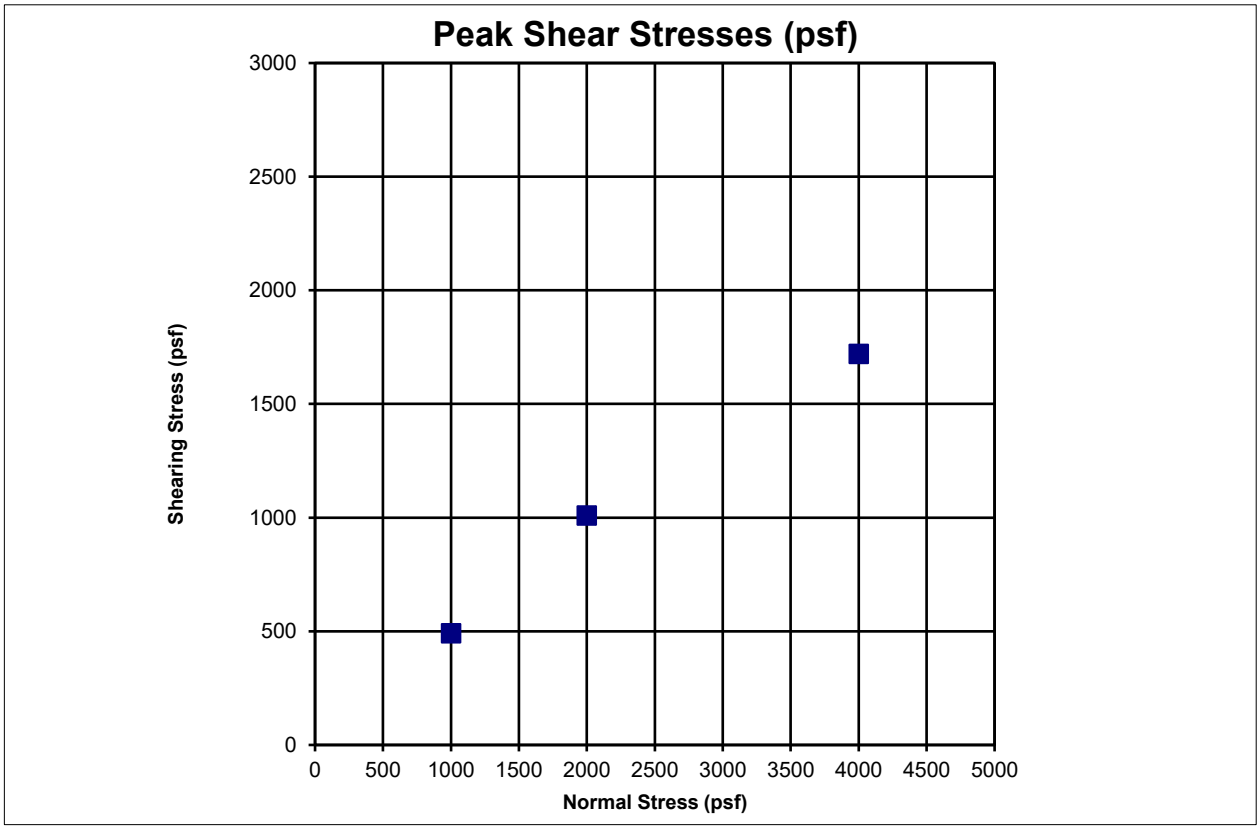
WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

PROJECT: ACS Services Project#2501640
LOCATION: Mesa, AZ
MATERIAL: See Boring Logs
SAMPLE SOURCE: B-3 @ 5-6 / ACS Lab# 25-4277-8
SAMPLE PREPARATION: Saturated - 1, 2, and 4ksf

JOB NO: US-EI-1720234239
WORK ORDER NO: 137
LAB NO: 25-1588
DATE ASSIGNED: 10/14/2025

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.):	0.999	1.000	0.999
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):	129.9	136.2	136.8
Initial Moisture Content:	9.3%	9.8%	11.5%
Initial Wet Density (pcf):	117.6	123.8	126.4
Initial Dry Density (pcf):	107.6	112.8	113.3
Final Moisture Content:	20.4%	19.7%	19.3%
Final Wet Density (pcf):	129.7	135.0	135.3
Final Dry Density (pcf):	107.7	112.8	113.5
Normal Stress (psf):	1000	2000	4000
Maximum Shearing Stress (psf):	492	1009	1721
Vertical Deformation @ Max Shear (in):	0.323	0.289	0.300
Horizontal Deformation @ Max Shear (in):	0.332	0.077	0.041



WSP USA
3630 E Wier Ave.
Phoenix, AZ 85040

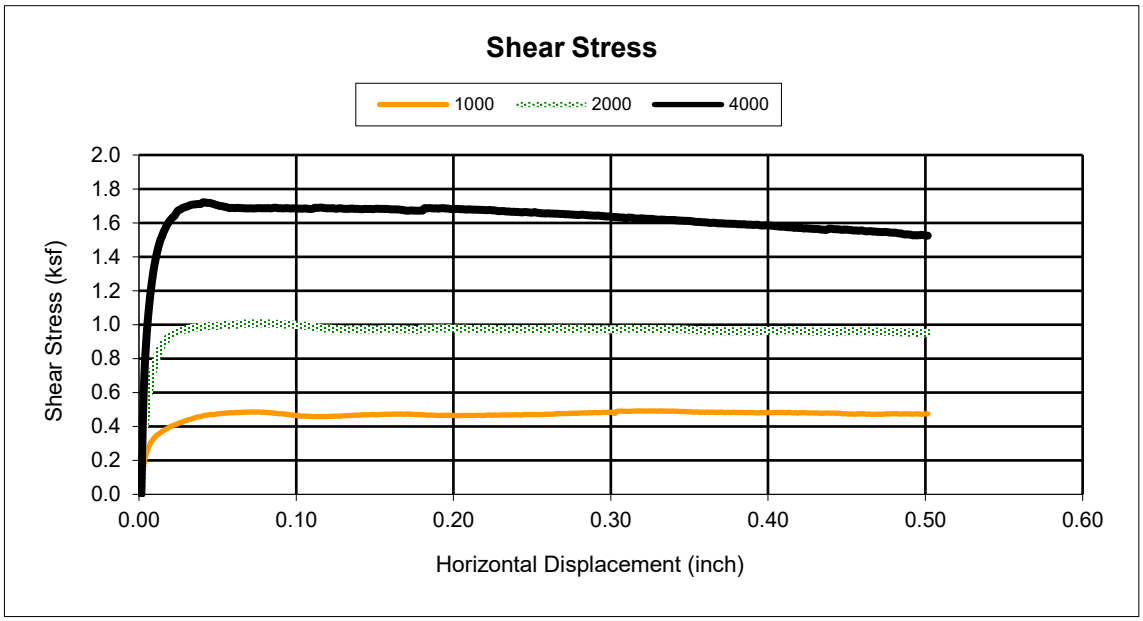
REVIEWED BY _____

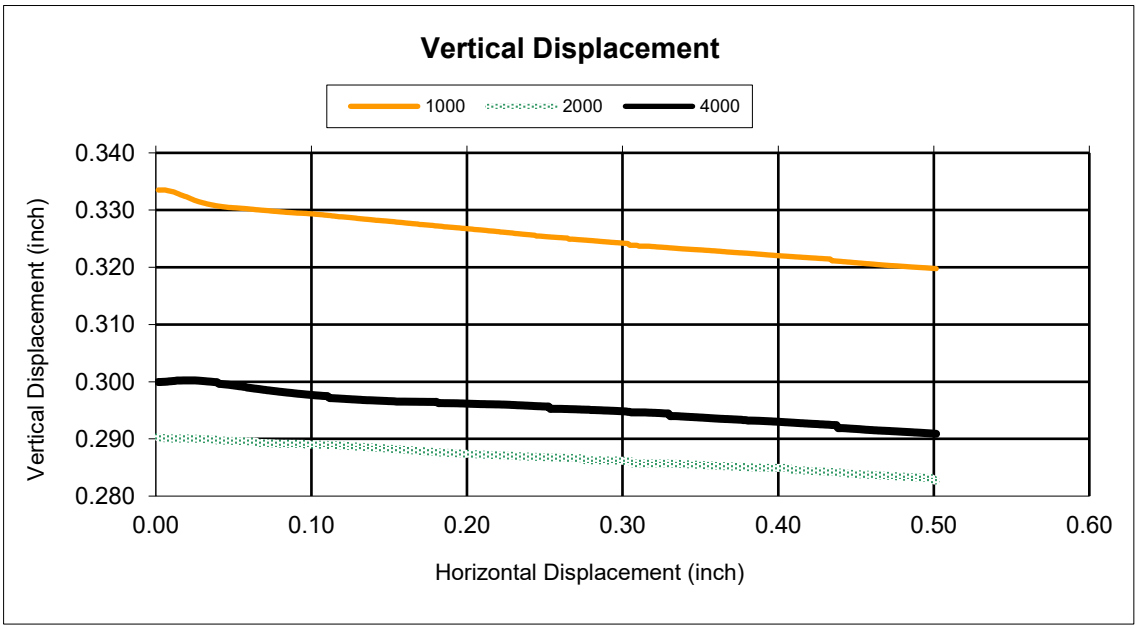
PROJECT: ACS Services Project#2501640
LOCATION: Mesa, AZ
MATERIAL: See Boring Logs
SAMPLE SOURCE: B-3 @ 5-6 / ACS Lab# 25-4277-8
SAMPLE PREPARATION: Saturated - 1, 2, and 4ksf

JOB NO: US-EI-1720234239
WORK ORDER NO: 137
LAB NO: 25-1588
DATE ASSIGNED: 10/14/2025

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

ACS Services LLC

Soil pH and Resistivity Determination

AASHTO T-289 AASHTO T-288 / ARIZ 236

Project #	2501640	Material Type:	Soils
Lab #	25-4277-1	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	10/10/2025
Project Name:	Ethos Engineering	Sampled By:	Client
Project Address:	ve Pedestrian Bridge over W Van Burer	Test Date:	Tuesday, October 14, 2025
Project City:	Tolleson	Tested By:	Mahalia Davis
Sample Source:	B-1 @ 0 -5	Resistivity Box:	
		Reviewed By:	Keagen Mayfield

pH Reading = 8.56

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	1.7	100	1150
50	6.74	1.1	100	740
50	6.74	6.3	10	420
50	6.74	5.9	10	400
50	6.74	6	10	400

Colin Eggebrecht
Lab Supervisor

Keagen Mayfield
Laboratory Manager

ACS Services LLC

Soil pH and Resistivity Determination

AASHTO T-289 AASHTO T-288 / ARIZ 236

Project #	2501640	Material Type:	Soils
Lab #	25-4277-5	Supplier:	Client
Client:	Ethos Engineering, LLC	Sample Date:	10/10/2025
Project Name:	Ethos Engineering	Sampled By:	Client
Project Address:	ve Pedestrian Bridge over W Van Burer	Test Date:	Wednesday, October 15, 2025
Project City:	Tolleson	Tested By:	Mahalia Davis
Sample Source:	B-2 @ 0.5 - 2	Resistivity Box:	
		Reviewed By:	Keagen Mayfield

pH Reading = 8.61

P = (SBF) x R x M

Where:

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

Water Added	SBF (cm)	Dial Reading (OHMS)	Multiplier	**P** (OHM-cm)
200	7.22	5.7	100	4120
50	7.22	5.7	100	4120
50	7.22	6.1	100	4400

Colin Eggebrecht
Lab Supervisor

Keagen Mayfield
Laboratory Manager

MOTZZ LABORATORY, INC.

Report: 958173
Reported: 10/16/2025
Received: 10/14/2025
PO: 2501640

Laboratory Analysis Report

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

Project: 2501640

Lab Number	Sample ID	
958173-1	25-4277-1	B-1 (0-5')

Test Parameter

Test	*Method*	*Result*	*Units*
Chloride	ARIZ 736b	496	ppm
Sulfate	ARIZ 733b	136	ppm

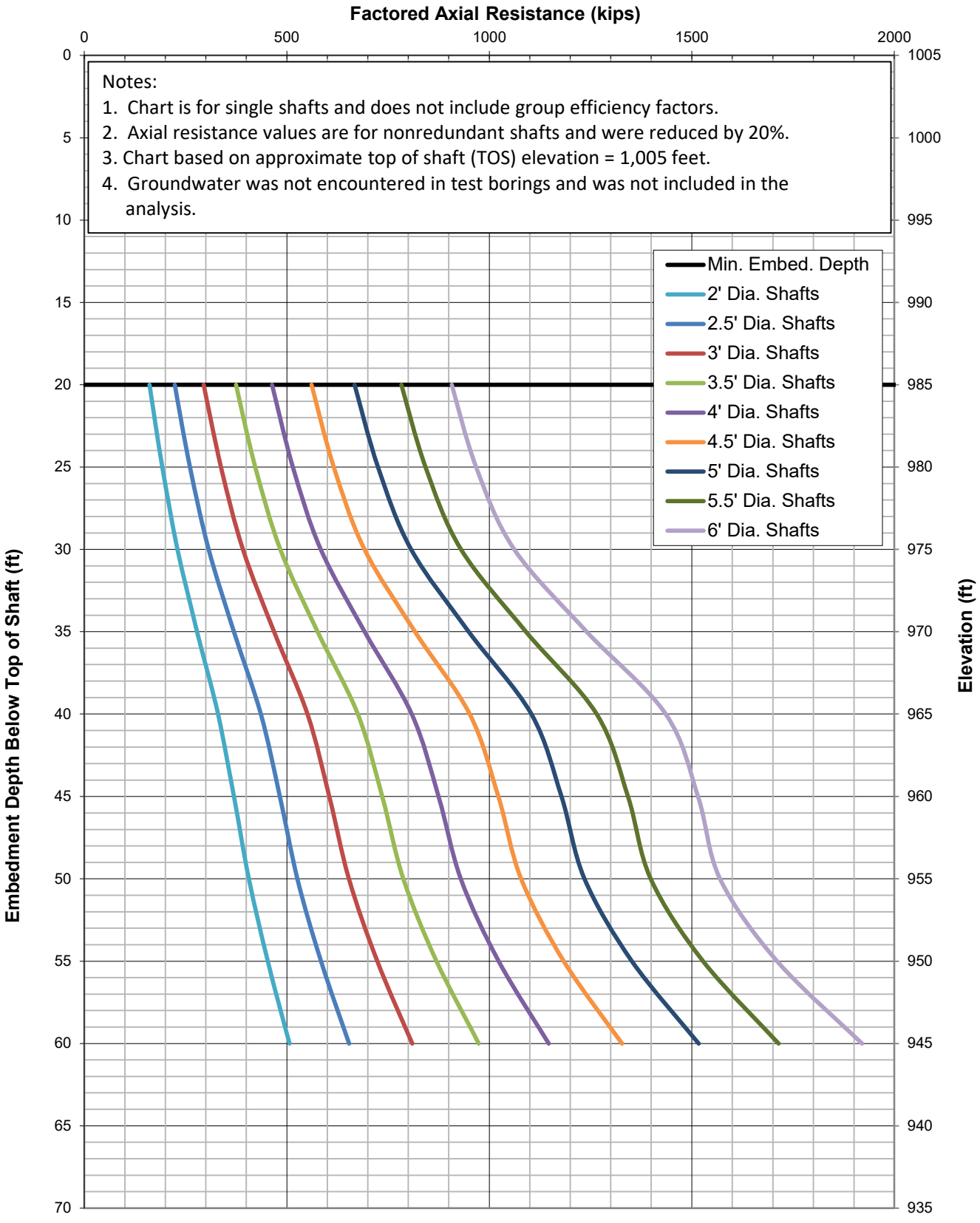
Lab Number	Sample ID	
958173-2	25-4277-5	B-2 (0.5-2')

Test Parameter

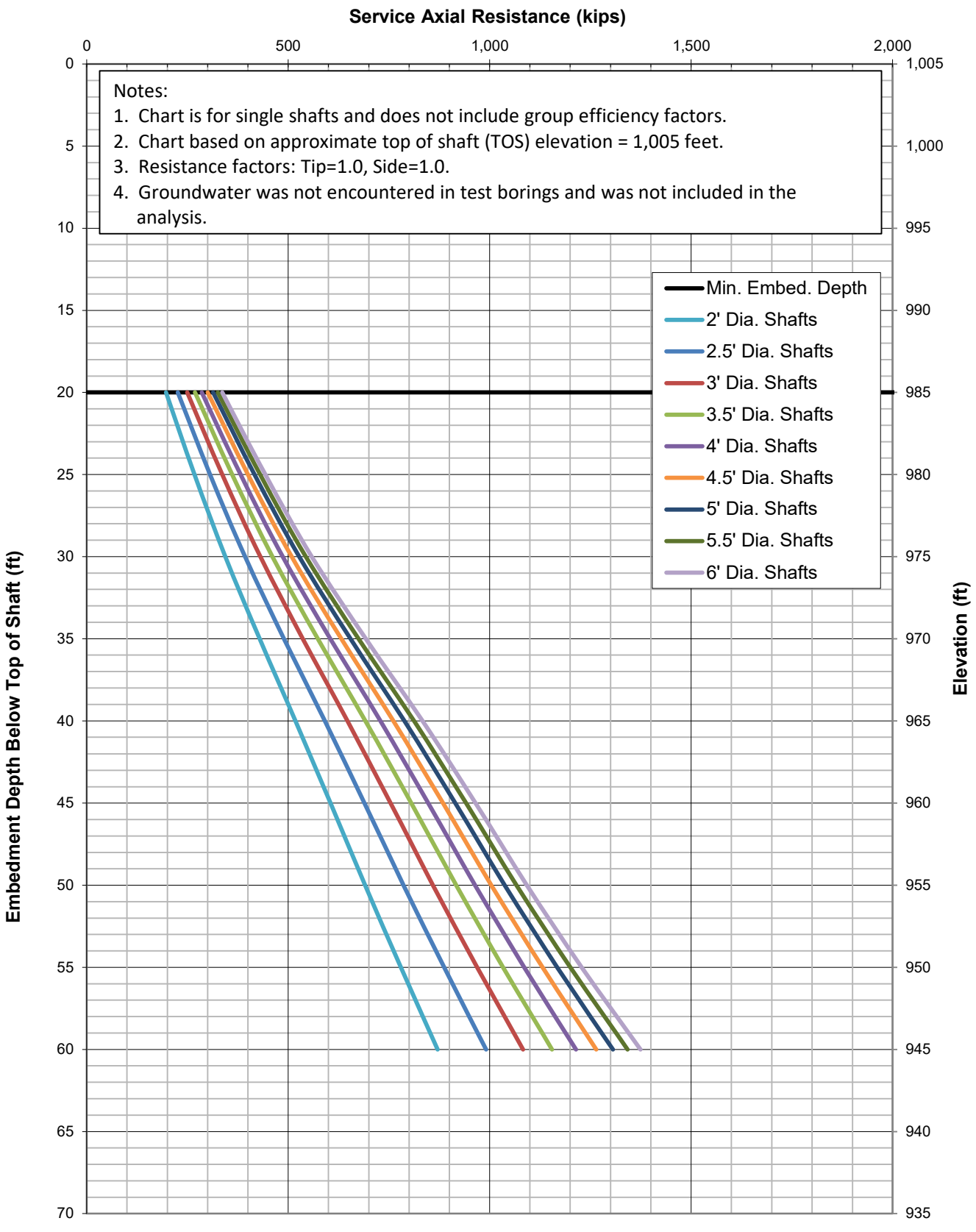
Test	*Method*	*Result*	*Units*
Chloride	ARIZ 736b	13	ppm
Sulfate	ARIZ 733b	28	ppm

APPENDIX C

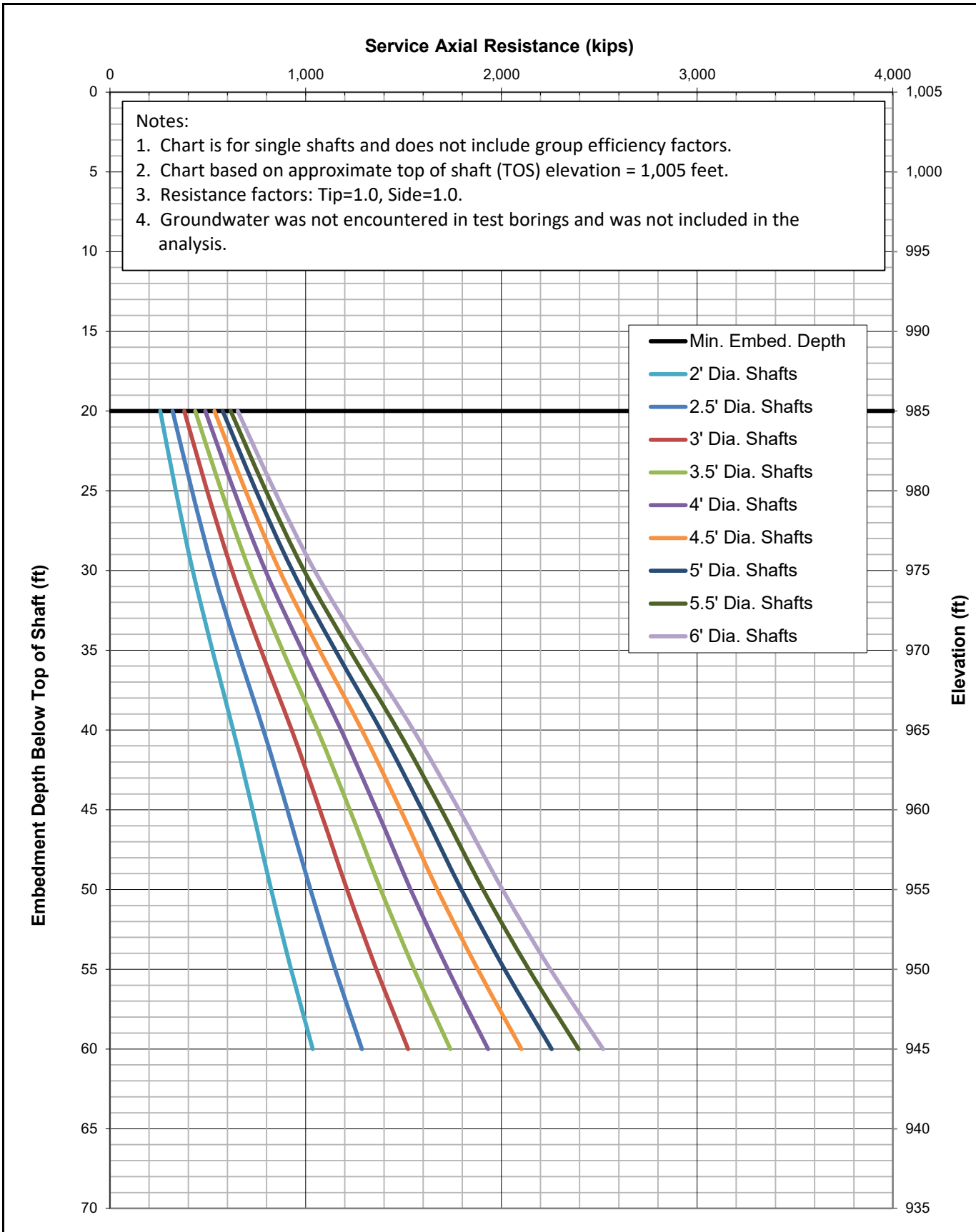
Drilled Shaft Axial Resistance Charts



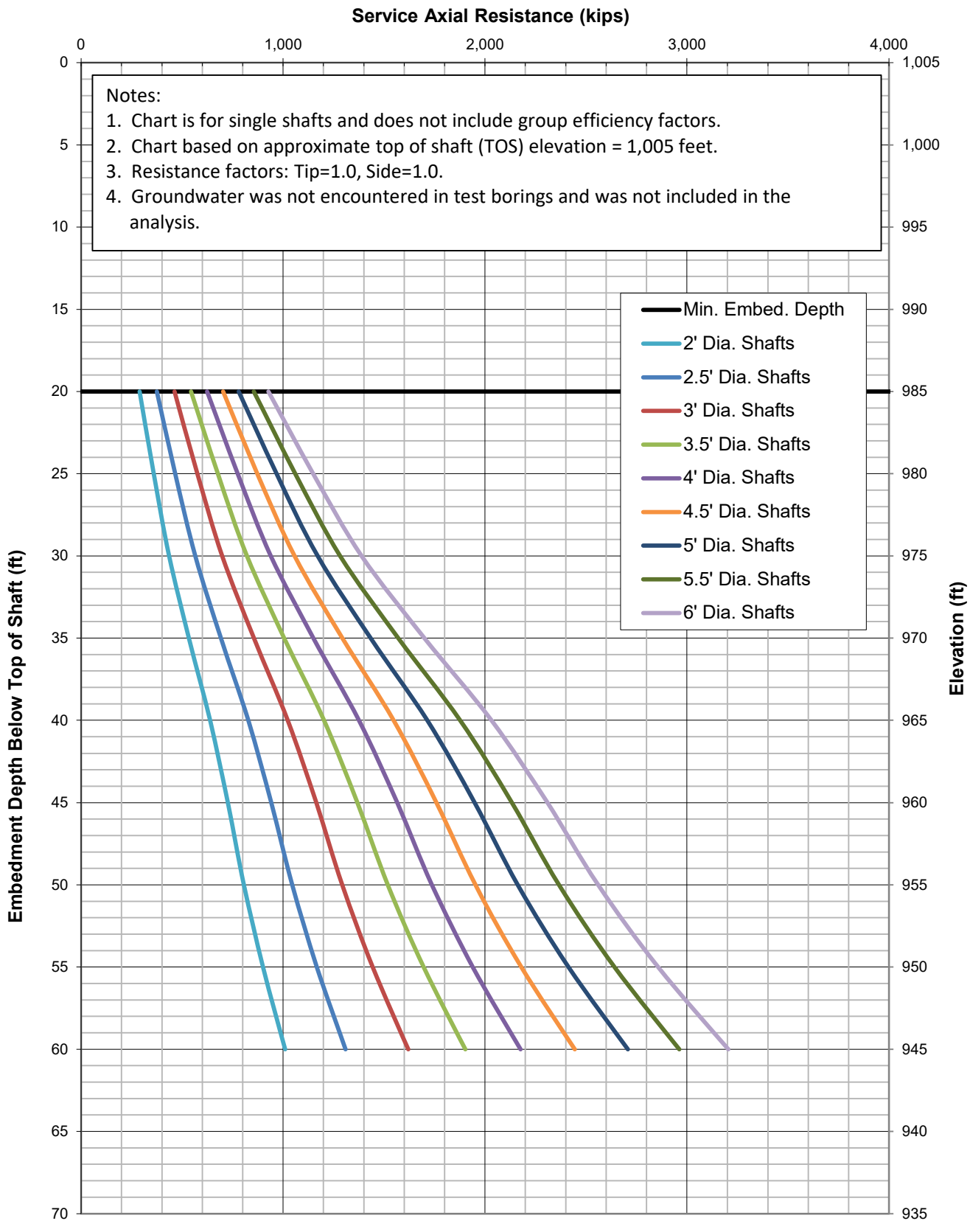
ethos ENGINEERING, LLC.		**DRILLED SHAFT FOUNDATION DESIGN CHART** **Strength Limit Axial Resistance in Kips** **96th Avenue Pedestrian Bridge over West Van Buren Street** **North and South Abutments**	Figure C1
Designer:	Date:		
Katie Mackay	11/13/2025		



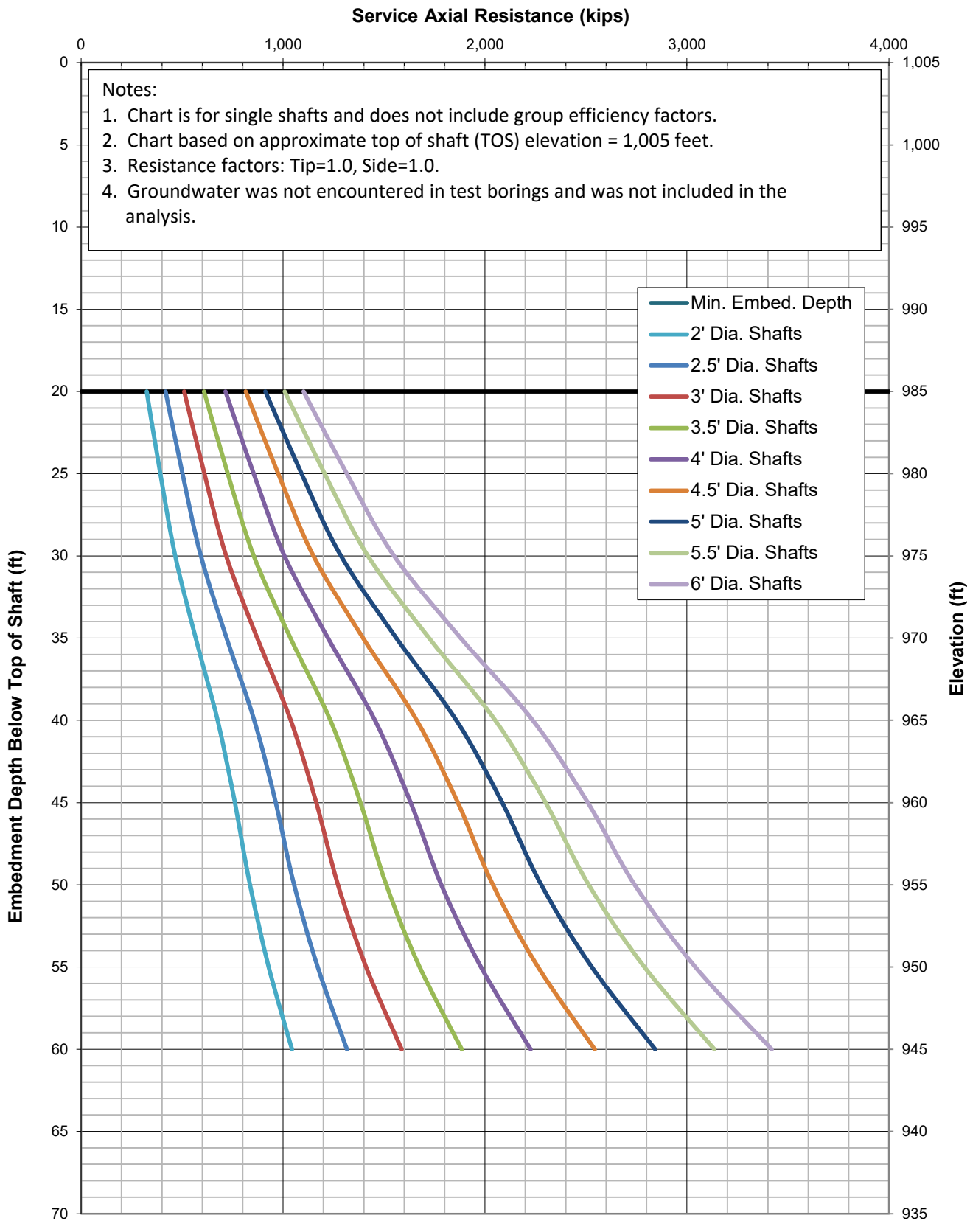
ethos ENGINEERING, LLC.		**DRILLED SHAFT FOUNDATION DESIGN CHART** **Service Limit (0.10-Inch Settlement) - Axial Resistance in Kips** **96th Avenue Pedestrian Bridge over West Van Buren Street** **North and South Abutments**	Figure C2
Designer:	Date:		
Katie Mackay	11/13/2025		



ethos ENGINEERING, LLC.		**DRILLED SHAFT FOUNDATION DESIGN CHART** **Service Limit (0.25-Inch Settlement) - Axial Resistance in Kips** **96th Avenue Pedestrian Bridge over West Van Buren Street** **North and South Abutments**	Figure C3
Designer:	Date:		
Katie Mackay	11/13/2025		



ethos ENGINEERING, LLC.		**DRILLED SHAFT FOUNDATION DESIGN CHART** **Service Limit (0.5-Inch Settlement) - Axial Resistance in Kips** **96th Avenue Pedestrian Bridge over West Van Buren Street** **North and South Abutments**	Figure C4
Designer:	Date:		
Katie Mackay	11/13/2025		



ethos ENGINEERING, LLC.

Designer:	Date:
Katie Mackay	11/13/2025

DRILLED SHAFT FOUNDATION DESIGN CHART

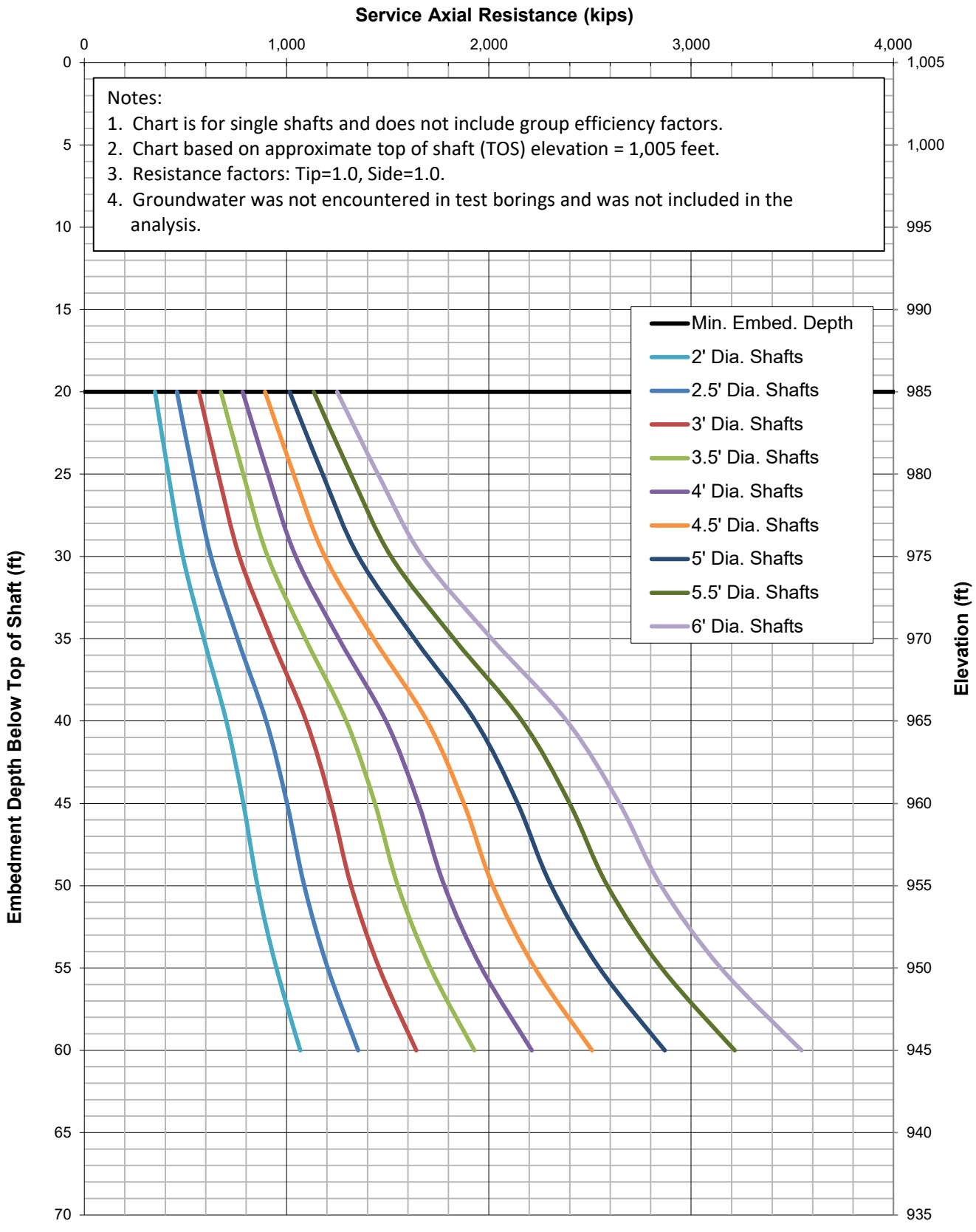
Service Limit (0.75-Inch Settlement) - Axial Resistance in Kips

96th Avenue Pedestrian Bridge over West Van Buren Street

North and South Abutments

Figure

C5



ethos ENGINEERING, LLC.

Designer:	Date:
Katie Mackay	11/13/2025

DRILLED SHAFT FOUNDATION DESIGN CHART

Service Limit (1 Inch Settlement) - Axial Resistance in Kips

96th Avenue Pedestrian Bridge over West Van Buren Street

North and South Abutments

Figure

C6

APPENDIX D

Retaining Wall Bearing Resistance Chart

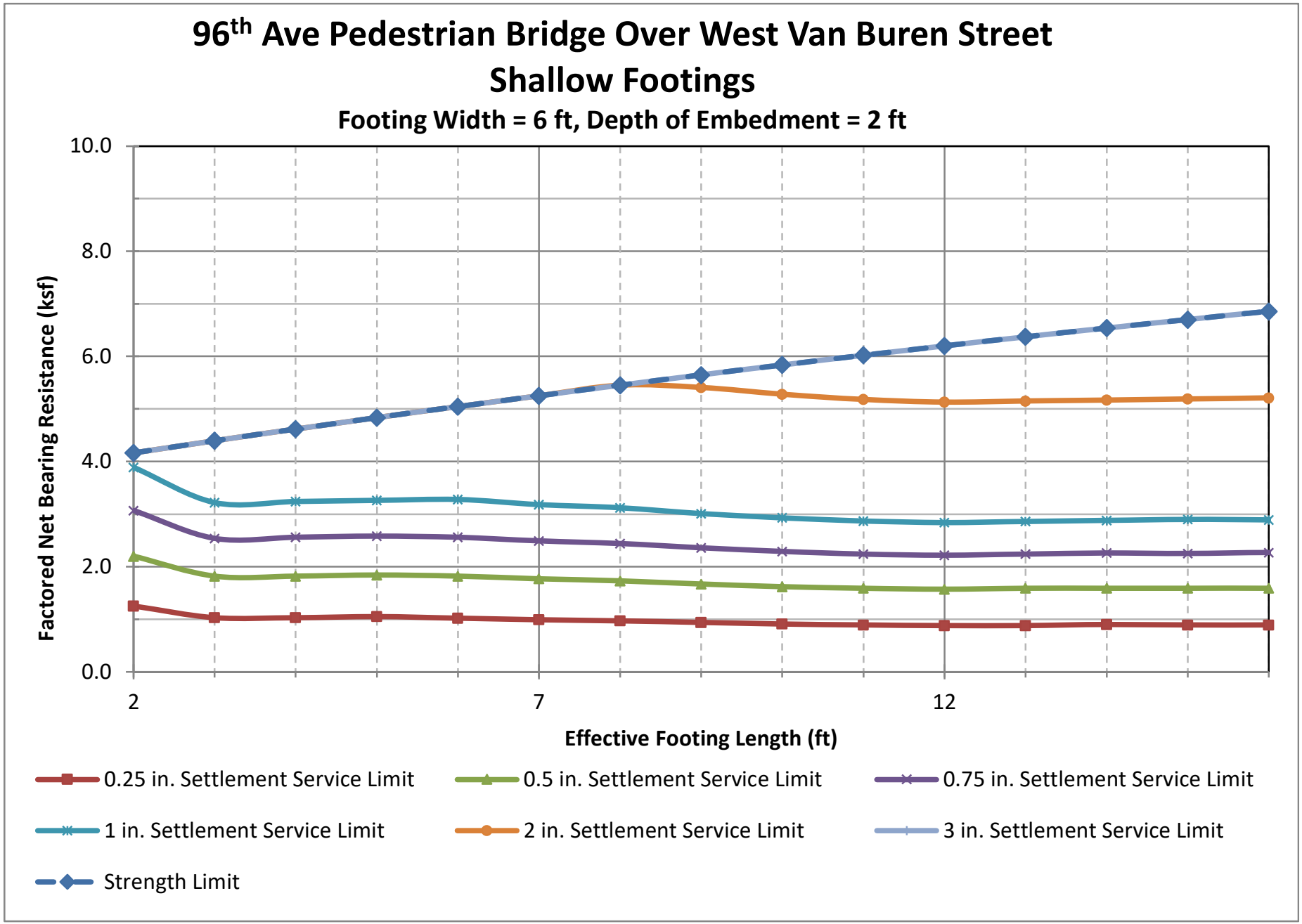

96th Ave Pedestrian Bridge Over West Van Buren Street
Shallow Footings
Footing Width = 6 ft, Depth of Embedment = 2 ft
Factored Net Bearing Resistance (ksf)
10.0
8.0
6.0
4.0
2.0
0.0
2
7
12
Effective Footing Length (ft)
0.25 in. Settlement Service Limit
0.5 in. Settlement Service Limit
0.75 in. Settlement Service Limit
1 in. Settlement Service Limit
2 in. Settlement Service Limit
3 in. Settlement Service Limit
Strength Limit

FIGURE D1

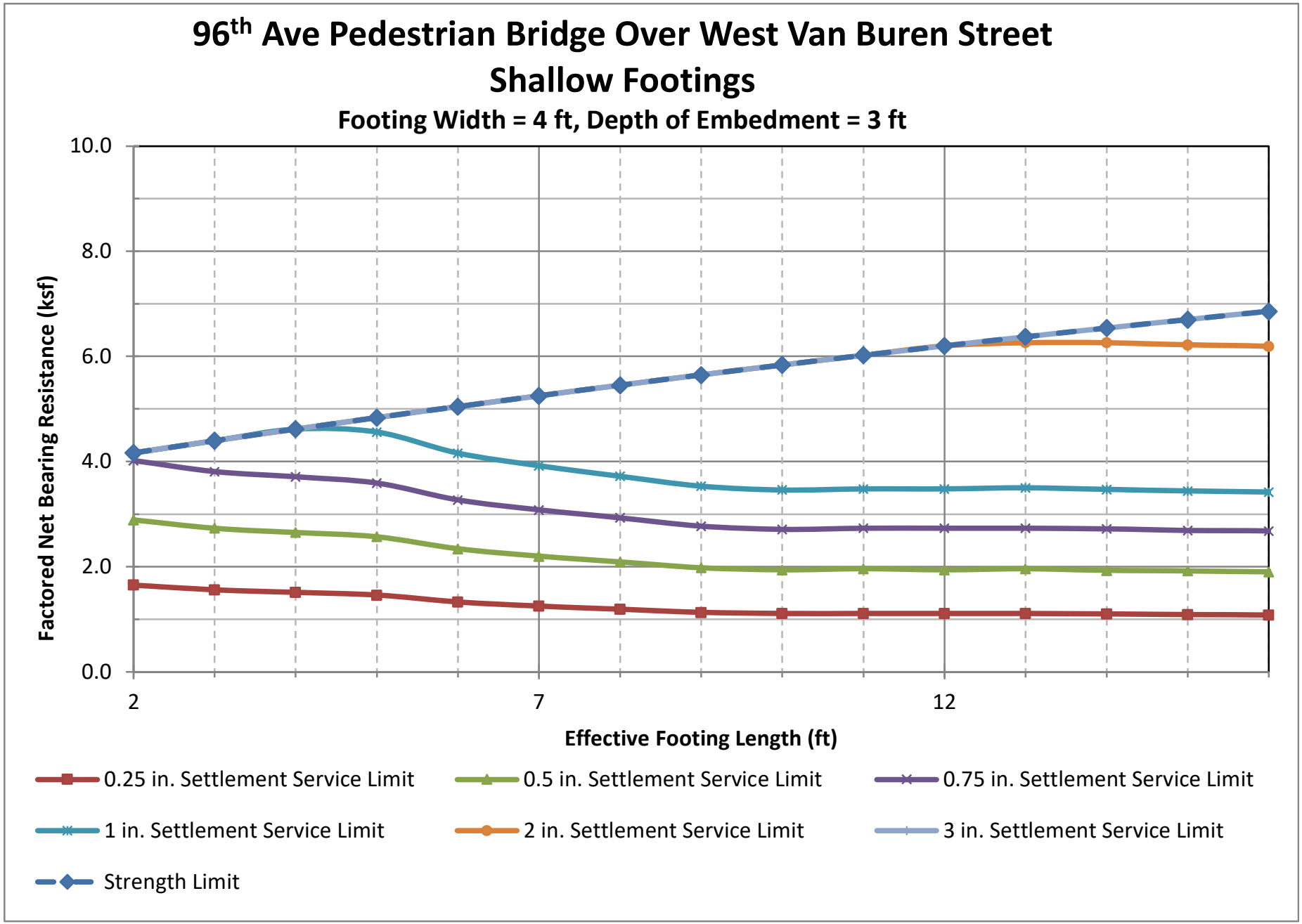

96th Ave Pedestrian Bridge Over West Van Buren Street
Shallow Footings
Footing Width = 4 ft, Depth of Embedment = 3 ft
Factored Net Bearing Resistance (ksf)
10.0
8.0
6.0
4.0
2.0
0.0
2
7
12
Effective Footing Length (ft)
0.25 in. Settlement Service Limit
0.5 in. Settlement Service Limit
0.75 in. Settlement Service Limit
1 in. Settlement Service Limit
2 in. Settlement Service Limit
3 in. Settlement Service Limit
Strength Limit

FIGURE D2

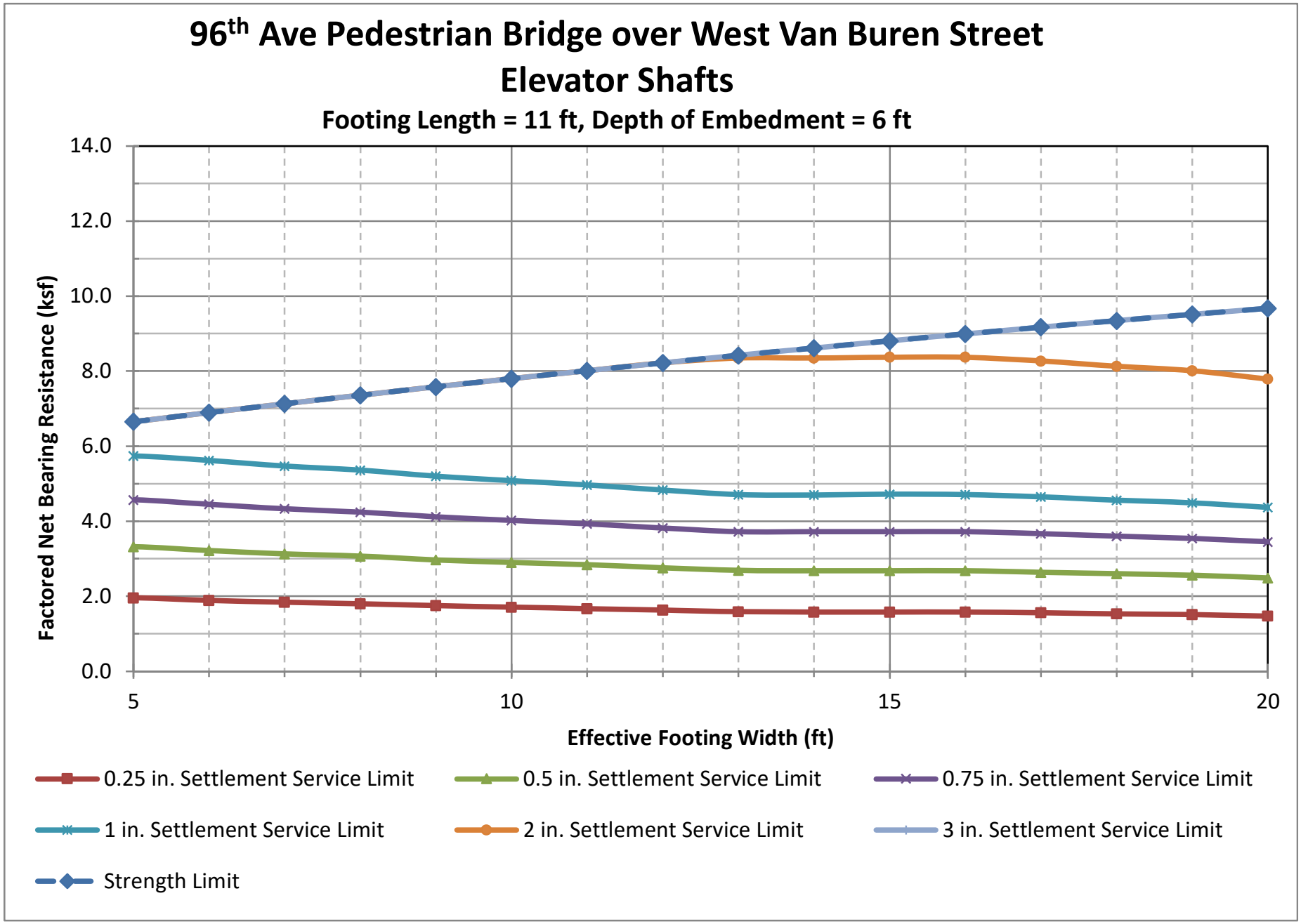

96th Ave Pedestrian Bridge over West Van Buren Street
Elevator Shafts
Footing Length = 11 ft, Depth of Embedment = 6 ft
Factored Net Bearing Resistance (ksf)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
5
10
15
20
Effective Footing Width (ft)
0.25 in. Settlement Service Limit
0.5 in. Settlement Service Limit
0.75 in. Settlement Service Limit
1 in. Settlement Service Limit
2 in. Settlement Service Limit
3 in. Settlement Service Limit
Strength Limit

FIGURE D3