

# **Division D**

## **Geotechnical Report**

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# **Geotechnical Engineering Study**

**Stinson Airport  
Hike and Bike Trail  
San Antonio, Texas**

**Arias Job No. 2016-18**



**Prepared For  
Gonzalez De La Garza**

**November 27, 2017**



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November 27, 2017  
Arias Job No. 2016-18

Ms. Ayda S. Gonzalez, P.E.  
Gonzalez De La Garza  
4800 Fredericksburg Road, Suite 200 SL  
San Antonio, Texas 78229

**RE: Geotechnical Engineering Study**  
Stinson Airport Hike and Bike Trail  
San Antonio, Texas

Dear Ms. Gonzalez:

This report presents the results of a Geotechnical Engineering Study for the proposed Hike and Bike Trail at Stinson Municipal Airport in San Antonio, Texas. Our scope of services was performed in general accordance with Arias Proposal 2016-18. This project was authorized by means of Standard Agreement for Professional Services Contract No. 4600014506 (TO-0000020); between Gonzalez De La Garza and Arias Geopprofessionals (Arias), effective May 26, 2016. Please consult with us as needed during any part of the design or construction process.

The long-term success of the project will be affected by the quality of materials used for construction and the adherence of the construction to the project plans and specifications. The quality of construction can be evaluated by implementing Quality Assurance (QA) testing. We recommend that the earthwork and foundation construction be tested and observed by Arias. A summary of our qualifications to provide QA testing is discussed in the "Quality Assurance Testing" section of this report. Furthermore, a message to the Owner with regard to QA testing is provided in the ASFE publication included in Appendix G.

We appreciate the opportunity to serve you during this phase of design. If we may be of further service, please call.

Sincerely,

**ARIAS GEOPROFESSIONALS**

TBPE Registration No: F-32

Golam Kibria, Ph.D., P.E.  
Geotechnical Engineer



11/27/2017

Christopher M. Szymczak, P.E.  
Senior Geotechnical Engineer

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## **INTRODUCTION**

The results of a Geotechnical Engineering Study for the proposed Stinson Hike and Bike Trail located in San Antonio, Texas are presented in this report. Our scope of services was performed in general accordance with Arias Proposal 2016-18. This project was authorized by means of Standard Agreement for Professional Services Contract No. 4600014506 (TO-0000020); between Gonzalez De La Garza and Arias Geoprosessionals (Arias), effective May 26, 2016.

## **SCOPE OF SERVICES**

The purpose of this geotechnical engineering study was to conduct a subsurface exploration and perform laboratory testing to establish engineering properties of the subsurface soil and groundwater conditions present on the site in order to:

- Develop foundation design parameters for the pedestrian bridge crossings;
- Provide trail section design thickness; and
- Provide box culvert design recommendations.

Environmental studies were not a part of our scope of services. Additionally, performing global stability analysis of creek embankments, slopes or retaining structures, parking lot pavement section design were also beyond our authorized service scope.

## **PROJECT DESCRIPTION AND SITE DESCRIPTION**

Gonzalez De La Garza & Associates (GDA) is assisting with the planning and design of a new hike and bike trail project in San Antonio, Texas. The proposed Hike and Bike Trail is located along the north side of Stinson Municipal Airport and the east side of the San Antonio River. A preliminary trail alignment drawing provided to us indicates that the planned trail and pathway will generally include a series of nature trails and pathways with a ring trail as the main pathway. The site is currently undeveloped and is heavily vegetated with mature trees and dense undergrowth. We understand that portions of the trail alignment have been cleared. The proposed trail surfaces may vary based on the locations, but will likely include concrete surfaces along the main ring trail.

The main entry point will be located along Mission Road near the existing Stinson Airport Terminal. The trail will connect to the existing San Antonio River Mission Trail. The entry point(s) will serve as trail heads and will likely include lighting, signage, maps, and a possible rest area. Artwork and potential entry monuments may also be included. A pedestrian bridge is planned over Acequia De Espada which will connect the proposed trail with the existing San Antonio River Walk trail. Additionally, a box culvert will be constructed in the north loop trail. A Vicinity Map is included as Figure 1 in Appendix A. The pedestrian bridge layout, trail layout and plan and profile of the proposed box culvert is presented in Appendix E.

## FIELD EXPLORATION

Five (5) soil borings were drilled to depths ranging from about 10 to 45 feet each on August 1 and 2, 2017, at the approximate locations shown on the Boring Location Plan (Figure 2) provided in Appendix A. Site photographs of existing site conditions taken at the time of drilling are also included in Appendix A.

The boring depths were measured from below the ground surface elevation that existed at the time of our drilling and sampling activities. The borings were sampled in accordance with ASTM D1586 for Split Spoon sampling and ASTM D1587 for Shelby tube sampling as described in Appendix C. A truck-mounted drill rig using continuous flight augers together with the sampling tool noted was used to secure the subsurface soil samples. After completion of drilling, the open boreholes were backfilled with soil cuttings.

Soil classifications and borehole logging were conducted during the exploration by one of our field-logging technicians, who is under the supervision of the project Geotechnical Engineer. Final soil classifications, as seen on the attached borings logs (Appendix B), were determined based on laboratory test results, in accordance with applicable ASTM procedures, and field observations. A summary of the boring number, corresponding project element, GPS coordinates, and approximate boring termination depth at each of the boring locations is provided in the Table below.

**Table 1: Boring Locations**

Boring No.	Description of Proposed Construction	Latitude	Longitude	Approximate Boring Termination Depth (ft)
B-1	Concrete Trail	N 29°20'22.7"	W 98°28'1.7"	10
B-2	Concrete Trail	N 29°20'19.5"	W 98°27'48.8"	10
B-3	Pedestrian Bridge	N 29°20'28.5"	W 98°27'38.6"	40
B-4	Pedestrian Bridge	N 29°20'28.2"	W 98°27'37.0"	45
B-5	Box Culvert	N 29°20'26.9"	W 98°28'5.0"	15

**Note:** Latitudes and longitudes of the bore locations are obtained using hand held GPS unit.

## LABORATORY TESTING

As a supplement to the field exploration, laboratory testing was conducted to determine soil moisture content, Atterberg Limits, unconfined compressive strength and percent passing the US Standard No. 200 sieve. The laboratory results are reported in the boring logs included in Appendix B. Grain size distribution curves are presented in Appendix D.

A key to the terms and symbols used on the logs is also included in Appendix B. The soil laboratory testing for this project was done in accordance with applicable ASTM procedures

with the specifications and definitions for these tests listed in Appendix C. Remaining soil samples recovered from this exploration will be routinely discarded following submittal of this report.

## SUBSURFACE CONDITIONS

Geology, generalized soil stratigraphy, and groundwater conditions at the project site are discussed in the following sections. The subsurface and groundwater conditions are based on conditions encountered at the boring locations to the depth explored.

### Geology

A Geologic Map is included as Figure 3 in Appendix A. The earth materials underlying the project site have been regionally mapped as Fluvatile Terrace Deposits (Qt). Fluvatile Terrace Deposits are alluvial soils consisting of clays and granular soils (i.e. sand or gravel) containing various amounts of clay, silt, sand, and/or gravel. The geotechnical engineering considerations for these types of deposits include the potential for significant variations over short distances and the potential of expansive clays being present.

### Site Stratigraphy and Engineering Properties

The general stratigraphic conditions and at the boring locations are provided subsequently in Tables 2 through 6 for Borings B-1 through B-5, respectively. *The presence and thickness of the various subsurface materials can be expected to vary away from and between the exploration locations.* The descriptions conform to the Unified Soils Classification System (ASTM D 2487).

**Table 2: Generalized Soil Conditions – Boring B-1**

Stratum	Depth, ft	Material Type	PI Range	No. 200 range	N Range
I	0 to 4.5	CLAYEY SAND (SC), medium dense, light brown to tan	*19	*48	25-30
II	4.5 to 10	SANDY LEAN CLAY (CL), very stiff to very hard, tan, with calcareous material	16-23	54-57	24-69

**Where:** Depth - Depth from existing ground surface during geotechnical study, feet  
 PI - Plasticity Index, %  
 No. 200 - Percent passing No. 200 sieve, %  
 N - Standard Penetration Test (SPT) value, blows per foot  
 \* - Only single test performed

**Table 3: Generalized Soil Conditions – Boring B-2**

Stratum	Depth, ft	Material Type	PI Range	No. 200 range	N Range
FILL	0 to 0.5	FAT CLAY with GRAVEL (CH), tan and gray	--	--	--
I	0.5 to 6.5	FAT CLAY (CH), stiff, dark brown and light brown	*35	--	9-15
II	6.5 to 8.0	POORLY GRADED GRAVEL with CLAY and SAND (GP-GC), medium dense, light brown to tan	*30	*10	*11
III	8 to 10	FAT CLAY (CH), stiff, brown and tan	*41	--	*9

**Where:** Depth - Depth from existing ground surface during geotechnical study, feet  
 PI - Plasticity Index, %  
 No. 200 - Percent passing No. 200 sieve, %  
 N - Standard Penetration Test (SPT) value, blows per foot  
 \* - Only single test performed

**Table 4: Generalized Soil Conditions – Boring B-3**

Stratum	Depth, ft	Material Type	PI Range	No. 200 range	PP range, tsf	N Range
I	0 to 8	SANDY FAT CLAY (CH), stiff to firm, light brown	25-31	*50	--	7-15
II	8 to 18	SANDY LEAN CLAY (CL), stiff to very stiff, light brown	*26	*50	*4.5+	*9
III	18 to 28	CLAYEY SAND with GRAVEL (SC), loose to medium dense, tan	--	14-15	--	9-23
IV	28 to 40	FAT CLAY (CH), very stiff, gray, with ferrous stain and gypsum crystals	*60	--	--	16-25

**Where:** Depth - Depth from existing ground surface during geotechnical study, feet  
 PI - Plasticity Index, %  
 No. 200 - Percent passing No. 200 sieve, %  
 N - Standard Penetration Test (SPT) value, blows per foot  
 PP - Pocket Penetrometer value, tsf  
 \* - Only single test performed

**Table 5: Generalized Soil Conditions – Boring B-4**

Stratum	Depth, ft	Material Type	PI Range	No. 200 range	PP range, tsf	N Range
I	0 to 4	CLAYEY GRAVEL with SAND (GC), dense, light gray and tan	*23	--	--	*39
II	4 to 10	CLAYEY SAND (SC), medium dense, tan and light brown, with gravel	*22	*37		24-27
III	10 to 18	SANDY LEAN CLAY (CL), stiff, dark brown, gray and tan, with ferrous stain, presence of wood piece	27-33	52-74	*1.25	*14
IV	18 to 23	CLAYEY SAND (SC), medium dense, light gray and brown	*25	*35	*4.0	--
V	23 to 28	CLAYEY GRAVEL with SAND (GC), medium dense, tan	--	--	--	*17
VI	28 to 45	FAT CLAY (CH), very stiff, gray and tan, with ferrous stain and gypsum crystal	*52	*96	3.0	20-30

**Where:** Depth - Depth from existing ground surface during geotechnical study, feet  
 PI - Plasticity Index, %  
 No. 200 - Percent passing No. 200 sieve, %  
 N - Standard Penetration Test (SPT) value, blows per foot  
 PP - Pocket Penetrometer value, tsf  
 \* - Only single test performed

**Table 6: Generalized Soil Conditions – Boring B-5**

Stratum	Depth, ft	Material Type	PI Range	No. 200 range	N Range
I	0 to 4.5	CLAYEY GRAVEL with SAND (GC), dense, light gray and tan	*29	*30	18-20
II	4.5 to 10	SANDY LEAN CLAY (CL), hard to stiff, light tan to tan	11-17	*58	11-33
III	10 to 15	CLAYEY GRAVEL with SAND (GC), medium dense, tan	--	*17	23-29

**Where:** Depth - Depth from existing ground surface during geotechnical study, feet  
 PI - Plasticity Index, %  
 No. 200 - Percent passing No. 200 sieve, %  
 N - Standard Penetration Test (SPT) value, blows per foot  
 \* - Only single test performed

## Groundwater

A dry soil sampling method was used to obtain the soil samples at the project site. Groundwater was encountered during the drilling of Borings B-3 and B-5 on August 1, 2017 and Boring B-4 on August 2, 2017. A summary of groundwater information is presented in the following table.

**Table 7: Groundwater Measurements in Borings**

Boring No.	Groundwater Depths (feet)		Caving Depths (feet)
	During Drilling	After Completion	
B-3	22.0	20.4	30.6
B-4	23.0	21.8	25.3
B-5	13.0	11.3	12.0

**Note:** Depth is measured from existing ground surface at the time of drilling on August 1 and August 2, 2017.

Groundwater levels will often change significantly over time and should be verified immediately prior to construction. Water levels in open boreholes may require several hours to several days to stabilize depending on the permeability of the soils. Groundwater levels at this site may differ during construction because fluctuations in groundwater levels can result from seasonal conditions, rainfall, drought, or temperature effects. Pockets or seams of gravels, sands, silts or open fractures and joints can store and transmit “perched” groundwater flow or seepage. Should dewatering become necessary during construction, it is considered “means and methods” and is solely the responsibility of the Contractor. Care should be utilized when excavating as these materials can slough and cave, especially if groundwater is present.

## Variations

Subsurface conditions may vary at other locations from the boring sites. Transition boundaries, contacts, and/or groundwater levels noted on the boring logs to separate material types are approximate. Actual contacts may be gradual and vary at different locations. If conditions encountered during construction indicate more variation than established as a result of this study, we should be contacted to evaluate the significance of the changed conditions relative to our recommendations.

## SOIL SHRINK/SWELL POTENTIAL DUE TO EXPANSIVE SOILS

The site soils have moderate to high expansion (shrink/swell) characteristics. Expansive clays shrink when they lose water and swell or grow in volume when they gain water. The potential of expansive clays to shrink and swell is typically related to the Plasticity Index (PI). Clays with a higher PI generally have a greater potential for soil volume changes due to moisture content variations. Change in soil moisture is a highly important factor affecting the shrinking and swelling of clays. More pronounced movements are commonly observed when soils are

exposed to extreme moisture fluctuations that occur between drought conditions and wet seasons.

We estimated potential vertical movement for this site using the Tex-124-E method outlined by the Texas Department of Transportation (TxDOT). The Tex-124-E method provides an estimate of potential vertical rise (PVR) using the liquid limits, plasticity indices, and existing water contents for soils. The PVR is estimated in the seasonally active zone, which was estimated at a 15-foot depth for this site.

Estimated PVRs are based upon assumed changes in soil moisture content from a dry to a wet condition; however, soil movements in the field depend on the actual changes in moisture content. Thus, actual soil movements could be less than that calculated if little soil moisture variations occur or the actual movement could exceed the estimated values if actual soil moisture content changes exceed the wet limits outlined by the PVR method. Such moisture conditions that exceed the limits of the PVR method may be the result of extended droughts, flooding, perched groundwater infiltration, poor surface drainage, and/or leaking irrigation lines.

The measured PIs of the near-surface soil samples obtained at this site suggest that the soils have a moderately high to high potential for shrinking and swelling due to fluctuations in soil moisture content.

To summarize, there is the potential for variable moisture conditions across the site that could lead to swelling conditions in some areas and shrinkage conditions in others. Based on the TxDOT method, we estimate that the PVR at the site will range from approximately **1.5 to 4.0 inches** at this site for the existing conditions.

## **PEDESTRIAN BRIDGE FOUNDATION RECOMMENDATIONS**

The type of foundation system selected for a given structure depends on several factors: (1) the function of the structure and the loads it may carry, (2) the subsurface conditions, (3) the cost of the foundation in comparison with the cost of the superstructure, and (4) the performance criteria of the foundation system.

Based on the 95% plan provided to us by Gonzalez De La Garza, we understand that the plan is to use two (2), 36-inch diameter straight-shaft drilled piers at each abutment to support the proposed pedestrian bridge. The design load of each drilled shaft is 65 ton. The top elevation of each pier is set at El. 537.2 feet. The piers, when properly founded can help reduce foundation movement of the superstructure. Geotechnical recommendations for straight-shaft drilled piers are presented in the following section.

### **Straight-Shaft Drilled Piers**

Items influencing the type of foundation selected for the proposed pedestrian bridges include the design axial and lateral foundation loads, the presence of expansive clays, and the

potential presence of groundwater. More specifically, the final pier dimensions, particularly to include the required length of pier, will be determined based on the foundation design loads, the depth of the active zone, the depth of potential erosion, the potential uplift force imposed by expansive soils within the active zone and the available side friction capacity and end-bearing capacity allotted to the subsurface stratigraphy (calculated allowable values are provided in the tables below). The active zone is the depth of the stratigraphy which is influenced by seasonal moisture variations. This depth is estimated at approximately 15 feet at this project site. The difference in elevation between the existing ground surface at the boring locations and the final top-of-pier elevation at the bridge abutments and bents will also influence the final pier dimensions.

Recommendations for evaluation of axial capacity and lateral capacity are presented in the following tables. Pier capacities for axial loading were evaluated based on design methodologies included in FHWA-NHI-10-016, May 2010 entitled Drilled Shafts: Construction Procedures and LRFD Design Methods. Both end bearing and side friction resistance may be used in evaluating the allowable bearing capacity of the pier shafts.

We encountered gypsum crystals at appreciable depths (about 38 feet in boring B-3 and about 33 feet in boring B-4) with minimal recovered samples. Our previous experience indicates that the presence of gypsum can sometimes signify moderate to high sulfate content. Prior to construction, we recommend determining the sulfate content of the subsoils for use in the design of the concrete (i.e., the concrete at this site should be designed in accordance with ACI 318 Table 19.3.2.1).

**Table 8: Drilled Pier Axial Design Parameters for Pedestrian Bridges, Boring Locations B-3 and B-4**

Elevation (feet)	Material	Recommended Design Parameters		
		Allowable Skin Friction, psf	Allowable End Bearing, psf	Uplift Force, kips
<b>Boring B-3 (Boring Elevation 537.54 feet) for Abutment 1</b>				
537.2 to 527.2	Stiff to Firm Sandy Fat Clay (CH), Stiff Sandy Lean Clay (CL)	Neglect Contribution		
527.2 to 519.5	Stiff to Very Stiff Sandy Lean Clay (CL)	350	--	50D
519.5 to 514.5	Loose Clayey Sand with Gravel (SC)	350	--	
514.5 to 509.5	Medium Dense Clayey Sand with Gravel (SC)	800	--	
509.5 to 497.5	Very Stiff Fat Clay (CH)	550	6,000	
<b>Boring B-4 (Boring Elevation 538.37 feet) for Abutment 2</b>				
537.2 to 527.2	Medium Dense and Dense Clayey Gravel with Sand(GC), Clayey Sand (SC) and Stiff Sandy Lean Clay (CL)	Neglect Contribution		
527.2 to 520.4	Stiff Sandy Lean Clay (CL)	450	--	40D
520.4 to 515.4	Medium Dense Clayey Sand (SC)	750	--	
515.4 to 510.4	Medium Dense Clayey Gravel with Sand (GC)	750	--	
510.4 to 493.4	Very Stiff Fat Clay (CH)	550	6,000	
<b>Constraints to be Imposed During Pier Design</b>				
<b>Proposed Pedestrian Bridge Location</b>	<b>East of Boring B-3</b>		<b>West of Boring B-4</b>	
<b>Minimum embedment depth</b>	Minimum 30 feet below existing ground surface, i.e. El. 507.5 feet (August 2017) or at least 5 feet into Very Stiff Gray Fat Clay (whichever depth is greater)		Minimum 30 feet below existing ground surface, i.e. El. 508.4 feet (August 2017) or at least 5 feet into Very Stiff Tan and Gray Fat Clay (whichever depth is greater)	
<b>Minimum void space beneath pier caps</b>	8 inches			
<b>Minimum shaft diameter</b>	36 inches			

**Notes:**

1. Potential scour depths were not available during preparation of this report. *The provided axial capacity should be adjusted to account scour depths.*
2. *For straight shaft piers, the contribution of the soils for the top 10 feet of soil embedment and for a length equal to at least 1 pier diameter from the bottom of the shaft should be neglected in determination of friction capacity. The recommended design parameters include a factor of safety of 2 for skin friction and of 3 for end bearing.*
3. The uplift force resulting from expansion of soils in the active zone may be computed using the above formula where D is the shaft diameter in feet. For drilled straight-sided piers, the contribution from soils to resist uplift is the allowable skin friction resistance of the soils below the 15-foot deep estimated active zone. Sustained dead loads will also aid in resisting uplift forces. Pier depths greater than the depths outlined above may be required to resist expansive soil uplift forces.
4. The minimum embedment depth was selected to locate the pier base below the depth of seasonal moisture change and within a specified desired bearing stratum. Pier vertical reinforcing steel should be designed to resist the uplift forces from swelling soils. A minimum of 1% of the gross cross-sectional area should be considered and the final reinforcing requirements should be determined by the project structural engineer. Tensile rebar steel should be designed in accordance with ACI Code Requirements.
5. Total and differential settlement of piers is expected to be less than 1 inch and 0.5 inch, respectively. Estimated settlements are based on performance of properly installed piers in the San Antonio metropolitan area. A detailed settlement estimate is outside of the scope of this service.
6. We anticipate that the piers will be subjected to water action and scour may occur. The pier length should be referenced from the level of the maximum scour depth. Likewise, the Lpile analysis should neglect the contribution of soils down to the maximum scour depth.
7. The near-surface soils have a potential to swell on the order of 2 to 3 inches. If pier caps are used for the planned bridges, we recommend that a positive void space be provided beneath the pier cap to allow for the underlying soils to swell without transmitting additional uplift swelling pressure to the planned structure.

Lateral pile analyses including capacity, maximum shear, and maximum bending moment will be evaluated by the project structural engineer using LPILE or similar software. In Table 9, Arias presents geotechnical input parameters for the encountered soils.

**Table 9: Drilled Pier Geotechnical Input Parameters for LPILE Analyses for Boring Locations B-3 and B-4**

Elevation (feet)	Material	$\gamma_e$	$C_u$	$\phi$	$K_{static}$	$e_{50}$
<b>Boring B-3 (Boring Elevation 537.54 feet) for Abutment 1</b>						
537.2 to 527.2	Stiff to Firm Sandy Fat Clay (CH), Stiff Sandy Lean Clay (CL)	Neglect Contribution				
527.2 to 519.5	Stiff to Very Stiff Sandy Lean Clay (CL)	120	1,000	0	500	0.007
519.5 to 517.1	Loose Clayey Sand with Gravel (SC)	120	0	30	25	--
517.1 to 514.5	Loose Clayey Sand with Gravel (SC)	58	0	30	20	--
514.5 to 509.5	Medium Dense Clayey Sand with Gravel (SC)	58	0	32	60	--
509.5 to 497.5	Very Stiff Fat Clay (CH)	58	2,000	0	1,000	0.005
<b>Boring B-4 (Boring Elevation 538.37 feet) for Abutment 2</b>						
537.2 to 527.2	Medium Dense and Dense Clayey Gravel (GC), Clayey Sand (SC) and Stiff Sandy Lean Clay (CL)	Neglect Contribution				
527.2 to 520.4	Stiff Sandy Lean Clay (CL)	120	1,000	0	500	0.007
520.4 to 516.1	Medium Dense Clayey Sand (SC)	120	0	32	90	--
516.1 to 515.4	Medium Dense Clayey Sand (SC)	58	0	32	60	
515.4 to 510.4	Medium Dense Clayey Gravel with Sand (GC)	58	0	32	60	
510.4 to 493.4	Very Stiff Fat Clay (CH)	58	2,000	0	1,000	0.005

**Where:**

- $\gamma_e$  = effective soil unit weight, pcf
- $C_u$  = undrained soil shear strength, psf
- $\phi$  = undrained angle of internal friction, degrees
- $K_{static}$  = modulus of subgrade reaction, pci
- $e_{50}$  = 50% strain value

**IBC Site Classification and Seismic Design Coefficients**

Site classification according to the International Building Code (2015) is based on the soil profile encountered to a 100-foot depth. The stratigraphy at the site location was explored to an approximate depth of about 45 feet. The soils encountered near the bottom of the deeper borings were extrapolated to also be present from between the 45 and 100-foot depths based on our experience with the local geologic conditions.

On the basis of the site class definitions included in the 2015 Code and the encountered generalized stratigraphy including SPT data, we characterize the site as Site Class D, a stiff soil profile.

Seismic design coefficients were determined using the on-line software, U.S. Seismic Design Maps Web Application, which was accessed through the following website: (<http://earthquake.usgs.gov/designmaps/us/application.php>). Analyses were performed considering the 2015 International Building Code (IBC). Input included GPS coordinates and Site Class D. Based on this information, the seismic design parameters for the site are summarized in the following table.

**Table 10: Seismic Design Parameters**

Site Classification	F <sub>a</sub>	F <sub>v</sub>	S <sub>s</sub>	S <sub>1</sub>
D	1.6	2.4	0.086 g	0.029 g

**Where:** F<sub>a</sub> = Site coefficient  
 F<sub>v</sub> = Site coefficient  
 S<sub>s</sub> = Mapped spectral response acceleration for short periods  
 S<sub>1</sub> = Mapped spectral response acceleration for a 1-second period

### **BOX CULVERT STRUCTURES**

We understand that the plan is to construct a new box culvert in north loop trail. Details of the box culvert is presented in Appendix E. Boring B-5 was drilled near the proposed box culvert and is being subsequently used for the geotechnical recommendations.

Excavations for the box culverts should preferably be neat-excavated. The excavations may need to be over-excavated to allow for the placement of bedding material that may be required by the project civil engineer. The bearing depth of the planned culverts was estimated as El. 540.8 feet from the provided plan and profile. Based on the results of our boring B-5, the table below outlines the allowable bearing pressures for the planned box culvert structure.

**Table 11: Box Culvert Allowable Bearing Pressure Information**

Stratum	Depth, ft (below existing ground surface)	Anticipated Bearing Soils	Allowable Bearing Pressure, psf
I	0 to 4.5	CLAYEY GRAVEL with SAND (GC), dense, light gray and tan	2,000
II	4.5 to 10	SANDY LEAN CLAY (CL), hard to stiff, light tan to tan	2,000
III	10 to 15	CLAYEY GRAVEL with SAND (GC), medium dense, tan	3,000

Using these allowable bearing pressures, total and differential settlement of the box culvert structures should be less than 1 inch and ½ inch, respectively, provided that the subgrade is prepared as discussed subsequently.

Depending on seasonal weather conditions, excavations may encounter free groundwater. Groundwater was observed at about 13 feet during the sampling activities and about 11.3 feet after completion of our drilling (below the existing ground surface on August 1, 2017). Thus, the contractor should anticipate the presence of ground water during proposed culvert construction. Depending on the volume of encountered ground water, conventional sump and pump methods may be utilized to temporarily dewater the base of the excavation to remain sufficiently dry to allow for concrete placement. The means and methods for dewatering the site are solely the responsibility of the contractor.

Excavation equipment may disturb the bearing soils and loose pockets can occur at the culvert’s bearing elevation. Accordingly, we recommend that the upper 6 inches of the base of the excavations be compacted to achieve a density of at least 95 percent of the maximum dry density as determined by TEX 114-E.

A common bedding material for culverts consists of 1-inch clean TXDOT concrete gravel Grade #5 (ASTM C-33 #67). Soil backfill above bedding materials and on top of the culverts (below the roadway section) should consist of select fill material meeting the following criteria: (1) free and clean of organic or other deleterious material, (2) have a plasticity index (PI) between 7 and 20, and (3) not contain particles exceeding 3 inches in maximum dimension. A filter fabric should be provided between any free-draining gravel and soil backfill to aid in preventing finer-grained soils from infiltrating into the free-draining gravel, which could lead to ground loss and distress to the overlying roadway. Onsite soils granular soils (Clayey Sand and/or Clayey Gravel), bedding materials, and select fill should be placed in lifts not to exceed 8 inches in loose measure and should be moisture conditioned to between -1 and +3 percentage points of optimum moisture content, and compacted to at least 95 percent of the maximum dry density

determined by TEX 114-E. A representative of Arias should observe the backfill and compaction processes.

A lean concrete “mud-mat” may also be used as an alternative to provide a layer of drainage gravel. The excavation may need to be over-excavated to: (a) remove soft “mucky” soils from the bearing surface, or (2) allow for the placement of the mud-mat and bedding material that may be required by the project civil engineer. Soft “mucky” soils, if encountered beneath the box culvert structure, should be replaced with select fill meeting the requirements in this report or flowable fill.

### Lateral Earth Pressures for Headwalls and Box Culverts

Lateral earth pressures that may act on the box culvert structures, stem walls, and headwalls can be evaluated by using the following equivalent fluid unit weights provided in the following table for the corresponding type of backfill. The equivalent fluid unit weights are based on “at-rest” earth pressure conditions.

**Table 12: Lateral Earth Pressures**

Backfill Type	Estimated Total Soil Unit Weight, (pcf)	Effective Soil Unit Weight, (pcf)	At-Rest Earth Pressure Coefficient, ( $k_o$ )	Equivalent Fluid Unit Weight	
				Dry Condition, (pcf)	Submerged Condition, (pcf)
Select Fill ( $7 \leq PI \leq 20$ )	125	63	0.50	63	94
Clean Gravel (Bedding Material)	125	63	0.44	55	90
On-site Clayey Sand (SC)	120	58	0.53	64	93
On-site Sandy Lean Clay (CL)	120	58	0.74	89	105
On-site Clayey Gravel (GC)	120	58	0.50	60	91

**Notes:**

1. The above equivalent fluid unit weights do not consider surcharge loads.
2. Soil and hydrostatic water pressures behind walls will impose a triangular stress distribution on the walls; surcharge loads will impose a rectangular stress distribution on the walls.

The equivalent fluid unit weight “submerged condition” values in the above table should be used if there is a chance for hydrostatic forces to develop. If hydrostatic water pressures are not expected to develop, the equivalent fluid unit weight “dry condition” values can be used.

Surcharge loads including equipment loads, traffic, and soil stockpiles should also be considered in the analysis of the box culvert structure and headwalls.

Measures should be taken to design against buoyancy forces. These methods may include reducing the potential for water to migrate beneath and around the sides of the box culvert structure and headwalls and/or by designing for the use of adequate overburden pressure. The weight of the box culvert structure and headwalls and soil backfill will aid in resisting potential buoyancy forces. The following design measures can be considered to reduce the risk of water entering backfill or becoming trapped under the box culverts:

- If granular soils are encountered in box culvert excavation bottoms, they can be undercut down to clay and replaced with lean clay select fill or flowable fill.
- Select fill consisting of the excavated clayey sand and lean clay can be used to backfill behind and above the box culverts and behind stem walls and wing walls.
- Concrete riprap aprons can be placed upstream and downstream of the box culverts, and “turn-down” reinforced concrete beams can be constructed to depths of at least 2 feet at the toes of the concrete aprons. The concrete beams and aprons are expected to be cast monolithically, so that there are not joints that water could possibly migrate through.
- Stone riprap, or other designed energy dissipator(s), can be placed upstream of the concrete apron to reduce flow velocities.

### **Erosion Control**

The performance of the outfall system will be directly related to the control of erosion; thus, protection against scour should be provided. Some potential erosion control methods are presented below:

- Rock Riprap
- Gabions and Slope Mattresses
- Concrete Lining
- Erosion Control Mats

Care should be taken to provide adequate anchorage for the erosion control materials. Actual measures for erosion and scour control should be determined by the project civil engineer.

Based on the results of the soil boring, we recommend that turn-down grade beams used for the planned box culvert and headwall structures extend a minimum of 36 inches below the lowest adjacent grade. Additional depth may be required by the designers based on the results of the site-specific scour analysis.

### **Site Drainage**

The favorable performance of any structure is dependent on positive site drainage. Careful consideration should be provided by the designers and contractor to ensure positive drainage of all storm waters away from the planned improvements both during and after construction.

## **TRAIL DESIGN CONSIDERATIONS**

We understand that the proposed trails and pathways will be limited to pedestrian and bicycle traffic. We anticipate that maintenance and light-duty service vehicles may be required to use new trails. The current plan is to use rigid concrete for the trail design. However, we have provided both rigid and flexible options for the trail design so that the design team can use the flexible pavement section if deemed necessary.

### **Moisture Fluctuations Beneath Pavements**

Dependent upon the pavement/pathway width, it is common for subsurface moisture content values to remain more constant beneath the middle of the structure. The moisture levels in the subgrade soils located near the edge of structure are more susceptible to changes in moisture that occur due to natural seasonal moisture fluctuations. The edges will dry and shrink during drought conditions, relative to the center of the structure. During wet climate periods, the edges will swell relative to the center of the structure. The shrinking and swelling of subgrade soils near the edge of pavements will result in longitudinal, surface cracking that occurs parallel to the structure. To help reduce the chances for moisture content variations of the subgrade soils, curbs should be extended a minimum of 6 inches to penetrate native soils to reduce lateral seepage behind the curbs into the base materials.

Landscaping along the pavement/pathway will increase the potential for moisture fluctuations along the pavement edges. Careful consideration should be provided by the designers to provide positive drainage away from adjoining landscapes. Ponding of water should not be allowed either on or near the edges of the planned pavements. Backfill behind curbs should consist of compacted, low-permeability clay. The use of landscape mulch or topsoil could provide an easy avenue for surface water to infiltrate behind and beneath curbs. This infiltration could adversely impact curb and pavement performance.

### **Pavement Recommendations**

Based on the results of the field exploration and the laboratory testing, the subgrade within the alignment of the proposed pathway/pavement may consist of clay (CL/CH) with varying amounts of sand and gravel, clayey gravel (GC) or clayey sand (SC). The design of the trail sections should adequately handle the anticipated pedestrian and light-duty traffic use. The pavement recommendations were prepared in accordance with the 1993 AASHTO Guide for the Design of Pavement Structures for asphalt and the ACI Design Guide for Design and Construction of Concrete Parking Lots for concrete. The following design parameters and assumptions were used in our analysis:

**Table 13: Pavement Design Assumptions**

<b>Pavement Type</b>	(1) Concrete-Paved Hike and Bike Trail (preferred) (2) Asphalt-Paved Hike and Bike Trail (optional)
<b>Traffic Load for Light-Duty Pavement</b>	15,000 equivalent single axle loads (ESALs)
<b>Concrete Compressive Strength</b>	3,500 psi
<b>Raw Subgrade California Bearing Ratio (CBR)</b>	3 for compacted soil subgrade
<b>Raw Subgrade Modulus of Subgrade Reaction, k in pci</b>	100 for compacted soil subgrade

Based on our experience with similar projects constructed on similar soils and our design assumptions, we recommend the following minimum thickness values be used to construct the planned trails.

**Table 14: Recommended Pavement Sections for Trails**

<b>Layer</b>	<b>Material</b>	<b>Flexible Asphaltic Concrete</b>	<b>Rigid Concrete</b>
Surface	HMAC/PCC	2"	5"
Base	Flexible Base	6"	--
Subgrade	Moisture Conditioned	6"	6"

**Rigid Concrete Pavement Joints**

Placement of expansion joints in concrete paving on potentially expansive subgrade or on granular subgrade subject to piping often results in horizontal and vertical movement at the joint. Many times, concrete spalls adjacent to the joint and eventually a failed concrete area results. This problem is primarily related to water infiltration through the joint.

One method to mitigate the problem of water infiltration through the joints is to eliminate all expansion joints that are not absolutely necessary. It is our opinion that expansion or isolation joints are needed only adjacent to where the pavement abuts intersecting drive lanes and other structures. Elimination of all expansion joints within the main body of the pavement area would significantly reduce access of moisture into the subgrade. Regardless of the type of expansion joint sealant used, eventually openings in the sealant occur resulting in water infiltration into the subgrade.

The use of sawed and sealed joints should be designed in accordance with current Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. Research has

proven that joint design and layout can have a significant effect on the overall performance of concrete pavement.

Recommendations presented herein are based on the use of reinforced concrete pavement. Local experience has shown that the use of distributed steel placed at a distance of 1/3 slab thickness from the top is of benefit in crack control for concrete pavements. Improved crack control also reduces the potential for water infiltration.

The use of a perimeter turned down beam should also be strongly considered for the rigid concrete section.

### **Performance Considerations**

Some shrink/swell movements due to moisture variations in the underlying soils should be anticipated over the life of the trails. The shrinking and swelling of the soils will provide movements in subgrade soils beneath the new trails. The relatively narrow width of the paths will make an asphalt section highly susceptible to shrink/swell movements. Longitudinal cracking along the pavement edges of asphalt trails will likely occur within a few years of construction due to seasonal moisture fluctuations. If the owner selects to use an asphalt surfaced trail, the owner should understand that the site pavements will likely crack due to the shrinking and swelling of the subgrade soils.

Although the initial construction cost of concrete paths will be somewhat higher than an asphalt-surfaced trail, the long-term performance of concrete will require less maintenance and up-keep when compared to an asphalt surface. Perimeter turn-down beams should also be considered to protect against erosion and scour at locations that will be prone to flooding.

The owner should recognize that over a period of time, pavements may crack and undergo some deterioration and loss of serviceability. As a result, some maintenance should be planned over the life of the pavement.

### **Pavement Subgrade and Section Materials**

Recommendations for subgrade preparation in the planned pavement areas are summarized in Table 15.

**Table 15: Pavement Subgrade and Section Materials**

<b>Stripping Depth</b>	6 inches or as needed to remove roots and organics
<b>Reuse Excavated Soils</b>	Provided they are free of roots and debris and meet the general fill material requirements.
<b>Undercut Extent</b>	2 feet beyond the paving limits
<b>Exposed Subgrade Treatment</b>	Proof roll with rubber tired vehicle weighting at least 15 tons such as a loaded dump truck with Geotechnical Engineer's representative present during proof rolling
<b>Pumping/Rutting Areas Discovered During Proofrolling</b>	Remove to firmer materials and replace with compacted general or select fill under direction of geotechnical engineer representative
<b>General Fill Type</b>	On-site material free of roots, debris and other deleterious material with a maximum particle size of 4 inches
<b>Maximum General Fill Loose Lift Thickness</b>	9 inches
<b>Flexible Base Material Type</b>	TxDOT Item 247, Type A, Grade 1 or 2
<b>Maximum Flexible Base Loose Lift Thickness</b>	9 inches
<b>Hot Mix Asphaltic Concrete (HMAC) Type</b>	TxDOT Standard Specifications Item 340, Type D
<b>Portland cement concrete (PCC)</b>	28-day compressive strength of 3,500 psi; 4 to 6-inch slump

To prevent degradation of the prepared subgrade and base material, paving preferably should be placed within 14 days. If pavement placement is delayed, protection of the base surface with an emulsion-based sealer should be considered. Alternately, the paving section could be slightly overbuilt so blading performed to remove distressed sections does not reduce the base thickness.

We understand that a planned parking lot at the Mission Road trailhead will be provided as part of a separate project. Therefore, it was not part of our scope of services to provide pavement recommendations for said planned parking lot.

### **CONSTRUCTION CRITERIA FOR PROPOSED SITE DEVELOPMENT**

This section provides Drilled Pier Construction considerations and Construction Criteria for General Sitework Considerations.

#### **Drilled Piers Construction Considerations**

The contractor should verify groundwater conditions before production pier installation begins. Comments pertaining to high-torque drilling equipment, groundwater, slurry, and temporary

casing are based on generalized conditions encountered at the explored locations. Conditions at individual pier locations may differ from those presented and may require that these issues be implemented to successfully install piers. Construction considerations for drilled pier foundations are outlined in the following table.

**Table 16: Drilled Pier Installation Considerations**

<b>Recommended installation procedure</b>	USACE refers to FHWA (FHWA-NHI-10-016, May 2010)
<b>High-torque drilling equipment anticipated</b>	May Be Required
<b>Groundwater anticipated</b>	Yes
<b>Temporary casing anticipated</b>	Yes; extent depends upon subsurface soil and groundwater conditions encountered during construction. Casing anticipated to be extended through water-bearing gravel, sand and clay soils; and sufficiently into the underlying relatively impervious fat clay layer to achieve an adequate seal. If temporary casing is used, and an impermeable layer is not encountered to seal the casing, Contractor may need to use slurry method for drilled shaft installation.
<b>Slurry installation anticipated</b>	Possible
<b>Concrete placement</b>	Same day as drilling.
<b>Maximum water accumulation in excavation</b>	2 inches
<b>Concrete installation method needed if water accumulates</b>	Tremie or pump to displace water
<b>Quality assurance monitoring</b>	Geotechnical engineer's representative should be present during drilling of all piers, should observe drilling and document the installed depth, should confirm and document bearing material type at the base of excavation and cleanliness of base, should observe placement of reinforcing steel

The following installation techniques will aid in successful construction of the shafts:

- The clear spacing between rebar or behind the rebar cage should be at least 3 times the maximum size of coarse aggregate.
- Centralizers on the rebar cage should be installed to keep the cage properly positioned.
- Cross-bracing of a reinforcing cage may be used when fabricating, transporting, and/or lifting. However, experience has shown that cross-bracing can contribute to the development of voids in a concrete shaft. Therefore, we recommend the removal of the cross-bracing prior to lowering the cage in the open shaft.
- The use of a tremie should be employed so that concrete is directed in a controlled manner down the center of the shaft to the shaft bottom. The concrete should not be allowed to ricochet off the pier reinforcing steel nor off the pier side walls.

- The pier concrete should be designed to achieve the desired design strength when placed at a 7-inch slump, plus or minus 1-inch tolerance. Adding water to a mix designed for a lower slump does not meet these recommendations.

Arias should be given the opportunity to review the proposed specifications prior to construction.

### General Site Earthwork Recommendations

If general fill is needed to raise site grade, the general fill may be obtained from on-site excavations provided it is processed to meet the criteria presented herein. Requirements for compacted general fill and for import select fill are outlined in the following table.

**Table 17: Site Work (Non Structural/General Fill and Select Fill) Requirements**

<b>Stripping Depth</b>	6 inch minimum or as needed to remove existing asphalt, concrete, and vegetation (if any)
<b>After Undercut, Exposed Subgrade Compaction Requirement</b>	Min. 95% of the max. Standard Proctor (ASTM D-698) at 0 to +4% of optimum moisture content for clay (CH/CL). Standard Proctor (ASTM D-698) at -1 to +3% of optimum moisture content for clayey gravel and/or clayey sand (GC/SC).
<b>Non-Structural/General Fill Type</b>	On-site material free of roots, debris and other deleterious material with a maximum particle size of 4 inches
<b>Maximum Non-Structural/General Fill Loose Lift Thickness</b>	9 inches
<b>Non-Structural/General Fill Compaction Requirement</b>	Min. 95% of the max. Standard Proctor (ASTM D-698) at 0 to +4% of optimum moisture content for clay (CH/CL). Standard Proctor (ASTM D-698) at -1 to +3% of optimum moisture content for clayey gravel and/or clayey sand (GC/SC).
<b>Imported Select Fill Type</b>	Imported Select Fill should be free of roots, debris and other deleterious material with a maximum particle size of 3 inches. It should have a Plasticity Index between 8 and 20 %.
<b>Maximum Select Fill Loose Lift Thickness</b>	9 inches
<b>Select Fill Compaction Requirement</b>	Min. 98% of the max. Standard Proctor (ASTM D-698) at -2 to +3% of optimum moisture content

Onsite soil generated from cut areas following clearing and grubbing that is free of excess organic material (three percent or less by weight) or debris may be suitable for use as general fill at the site. If used, imported Select Fill should be non-expansive, having a Plasticity Index between 8 and 20, and enough fines so the soil can bind together. Imported soil should be free of organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. The Geotechnical Engineer should approve imported Select Fill prior to delivery onsite.

The planned site fills may be located within the floodplain. The earthwork will be subject to erosion during extreme flood events. We recommend that the project Civil Engineer include a review of the planned earthwork to consider the effects of the flooding on the proposed site improvements. Erosion control measures and/or armoring of the embankments may be required as part of the planned improvements to reduce the potential for scour and undermining.

**Excavations**

The contractor should be aware that slope height, slope inclination, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, or federal safety regulations, e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, dated October 31, 1989. Such regulations are strictly enforced and, if not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties. The soils encountered at this site were classified as to type in accordance with this publication and are shown in the table below:

**Table 18: OSHA Soil Classifications**

Description	OSHA Classification
FAT CLAY (CH)/LEAN CLAY (CL)	C
CLAYEY GRAVEL (GC)/CLAYEY SAND (SC)	C

**\*\*It must be noted that layered slopes cannot be steeper at the top than the underlying slope and that all materials below the water table must be classified as Type “C” soils. The OSHA publication should be referenced for layered soil conditions, benching, etc.**

For excavations less than 20 feet deep, the maximum allowable slope for Type “C” soils is 1.5H:1V (34°). It should be noted that the table and allowable slopes above are for temporary slopes. Permanent slopes at this site should be sloped no steeper than 4H:1V and flatter slopes may be required. Flatter slopes may also be desired for mowing purposes.

It should be noted that heavy duty excavating equipment may be required. The contractor should be prepared for using heavy duty excavating equipment, if deemed necessary.

Appropriate trench excavation methods will depend on the various soil and groundwater conditions encountered. We emphasize that undisclosed soil conditions may be present at locations and depths other than those encountered in our borings. Consequently, flatter slopes and dewatering techniques may be required in these areas.

The soils to be penetrated by excavations may vary significantly across the site. Our preliminary soil classification is based solely on the materials encountered in the five borings. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different

subsurface conditions are encountered at the time of construction, we recommend that Arias be contacted immediately to evaluate the conditions encountered.

Trenches less than 5 feet deep are generally not required to be sloped back or braced following federal OSHA requirements for excavations. Sides of temporarily vertical excavations less than 5 feet deep may stay open for short periods of time; however, the granular soils that are expected to be encountered in trench excavations are subject to random caving and sloughing. If side slopes begin to slough, the sides should be either braced or be sloped back as needed.

If any excavation is extended to a depth of more than twenty (20) feet, it will be necessary to have the side slopes designed by a professional engineer registered in Texas. As a safety measure, it is recommended that all vehicles and soil piles be kept a minimum lateral distance from the crest of the slope equal to no less than the slope height.

Specific surcharge loads such as traffic, heavy cranes, earth stockpiles, pipe stacks, etc., should be considered by the Trench Safety Engineer. It is also important to consider any vibratory loads such as heavy truck traffic.

It is required by OSHA that the excavations be carefully monitored by a competent person making daily construction inspections. These inspections are required to verify that the excavations are constructed in accordance with the intent of OSHA regulations and the Trench Safety Design. If deeper excavations are necessary or if actual soil conditions vary from the borings, the trench safety design may have to be revised. It is especially important for the inspector to observe the effects of changed weather conditions, surcharge loadings, and cuts into adjacent backfills of existing utilities. The flow of water into the base and sides of the excavation and the presence of any surface slope cracks should also be carefully monitored by the Trench Safety Engineer.

The bottoms of trench excavations should expose strong competent soils, and should be dry and free of loose, soft, or disturbed soil. If fill soils are encountered at the base of trench excavations, their competency should be verified through proof-rolling, probing and density testing. Soft, wet, weak, or deleterious materials should be over-excavated to expose strong competent soils. If soft or weak soils are unexpectedly encountered to great depth, over-excavation to stronger soils may not be feasible and/or economical. In the event of encountering these areas of deep soft or weak soils, we recommend that the bottom of the trench be evaluated by the contractor's Trench Safety Engineer and the project Geotechnical Engineer.

### **Groundwater Control**

The contractor should be prepared with appropriate dewatering measures to dewater the site as necessary to allow for the proposed construction. Open sump and pump methods may be a possible method for use. We should note that the means, methods and designs of dewatering systems are solely the responsibility of the Contractor.

### **Drainage**

Good positive drainage during and after construction is very important to help reduce expansive soil volume changes, which can detrimentally affect the performance of the planned development. Proper attention to drainage details during the design and construction phases of development can prevent many potential soil shrink-swell and other moisture-related and erosion problems during and following the completion of the project.

### **Earthwork**

Exposure to the environment may weaken the soils at the bearing level if the excavation remains open for long periods of time. Therefore, it is recommended that all excavations be extended to final grade and constructed as soon as possible in order to reduce the risk of significant damage to bearing materials. If bearing materials are exposed to severe drying or wetting, the unsuitable material must be re-conditioned or removed as appropriate. The bearing level should be free of loose soil, ponded water or debris, and should be observed by the representative of the Geotechnical Engineer.

Subgrade preparation and fill placement operations should be monitored by the Geotechnical Engineer or his representative. Compaction tests should be performed as required. Any areas not meeting the required compaction should be recompacted and retested until compliance is met.

## **GENERAL COMMENTS**

This report was prepared as an instrument of service for this project exclusively for the use of Gonzalez De La Garza and the project design team. If the development plans change relative to layout, anticipated loads, or if different subsurface conditions are encountered during construction, we should be informed and retained to ascertain the impact of these changes on our recommendations. We cannot be responsible for the potential impact of these changes if we are not informed. Important information about this geotechnical report is provided in the ASFE publication included in Appendix F.

### **Geotechnical Design Review**

Arias should be given the opportunity to review the design and construction documents. The purpose of this review is to check to see if our geotechnical recommendations are properly interpreted into the project plans and specifications. Please note that design review was not included in the authorized scope and additional fees may apply.

### **Subsurface Variations**

Soil and groundwater conditions may vary between the sample boring locations. Transition boundaries or contacts, noted on the boring logs to separate soil types, are approximate. Actual contacts may be gradual and vary at different locations. The Contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions or highly variable subsurface conditions are encountered during construction, we

should be contacted to evaluate the significance of the changed conditions relative to our recommendations.

### **Quality Assurance Testing**

The long-term success of the project will be affected by the quality of materials used for construction and the adherence of the construction to the project plans and specifications. As Geotechnical Engineer of Record (GER), we should be engaged by the Owner to provide Quality Assurance (QA) testing. Our services will be to evaluate the degree to which constructors are achieving the specified conditions they're contractually obligated to achieve, and observe that the encountered materials during earthwork for foundation installation are consistent with those encountered during this study. In the event that Arias is not retained to provide QA testing, we should be immediately contacted if differing subsurface conditions are encountered during construction. Differing materials may require modification to the recommendations that we provided herein. A message to the Owner with regard to the project QA is provided in the ASFE publication included in Appendix G.

Arias has an established in-house laboratory that meets the standards of the American Standard Testing Materials (ASTM) specifications of ASTM E-329 defining requirements for Inspection and Testing Agencies for soil, concrete, steel and bituminous materials as used in construction. We maintain soils, concrete, asphalt, and aggregate testing equipment to provide the testing needs required by the project specifications. All of our equipment is calibrated by an independent testing agency in accordance with the National Bureau of Standards. In addition, Arias is accredited by the American Association of State Highway & Transportation Officials (AASHTO), the United States Army Corps of Engineers (USACE) and the Texas Department of Transportation (TxDOT), and also maintains AASHTO Materials Reference Laboratory (AMRL) and Cement and Concrete Reference Laboratory (CCRL) proficiency sampling, assessments and inspections.

Furthermore, Arias employs a technical staff certified through the following agencies: the National Institute for Certification in Engineering Technologies (NICET), the American Concrete Institute (ACI), the American Welding Society (AWS), the Precast/Prestressed Concrete Institute (PCI), the Mine & Safety Health Administration (MSHA), the Texas Asphalt Pavement Association (TXAPA) and the Texas Board of Professional Engineers (TBPE). Our services are conducted under the guidance and direction of a Professional Engineer (P.E.) licensed to work in the State of Texas, as required by law.

In addition to QA testing, Arias can also provide Storm Water Pollution Prevention Plan (SWPPP) services during construction.

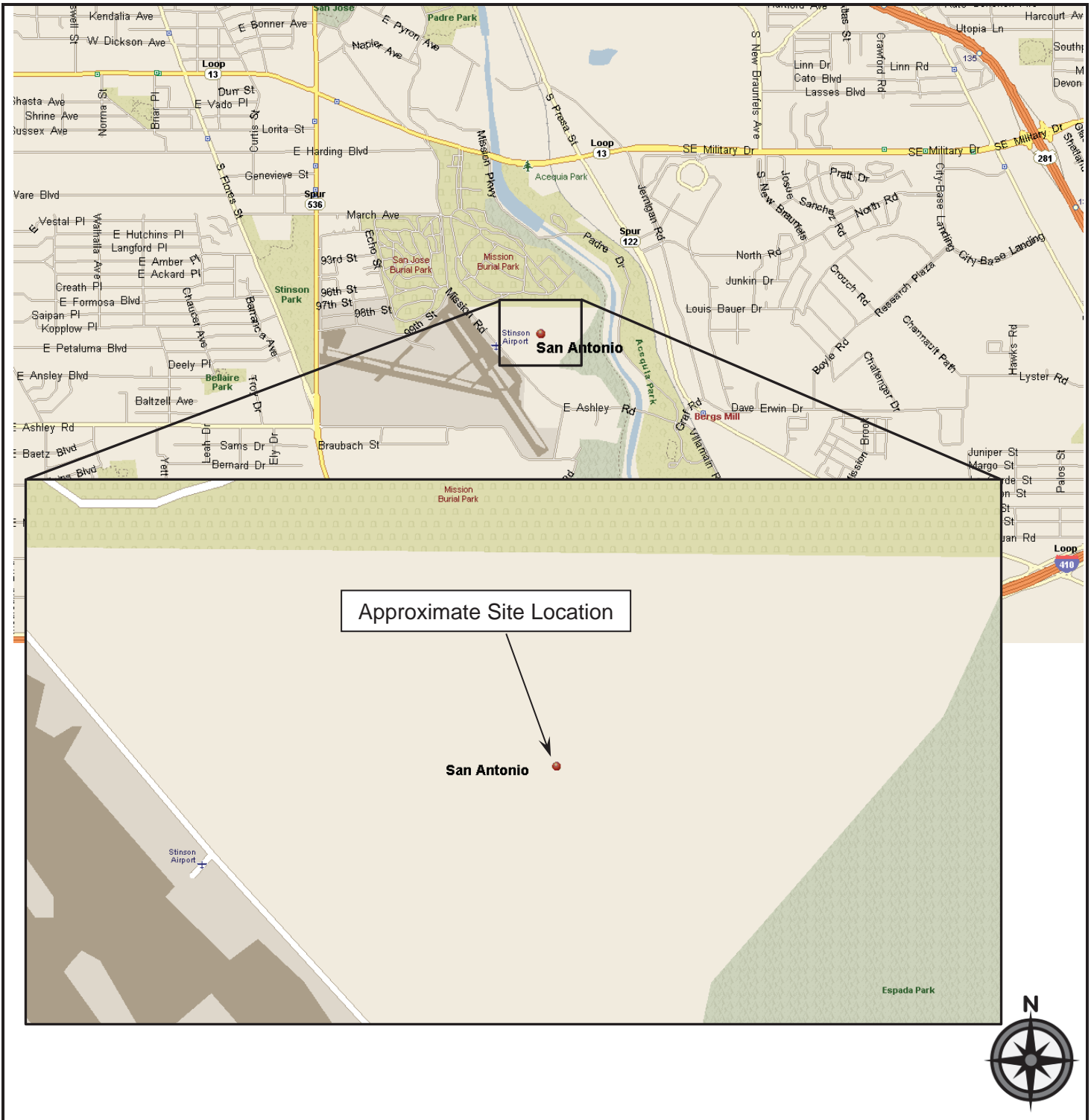
Subgrade preparation and fill placement operations should be observed and tested by the Geotechnical Engineer or his/her representative. As a guideline, at least one in-place density test should be performed according to the table below, with a minimum of 3 tests per lift. Any

areas not meeting the required compaction should be recompact and retested until compliance is met.

**Standard of Care**

Subject to the limitations inherent in the agreed scope of services as to the degree of care and amount of time and expenses to be incurred, and subject to any other limitations contained in the agreement for this work, Arias has performed its services consistent with that level of care and skill ordinarily exercised by other professional engineers practicing in the same locale and under similar circumstances at the time the services were performed.

## **APPENDIX A: FIGURES AND SITE PHOTOGRAPHS**



## VICINITY MAP

Stinson Hike and Bike Trails  
San Antonio, Texas

142 Chula Vista, San Antonio, Texas 78232  
Phone: (210) 308-5884 • Fax: (210) 308-5886

Date: September 7, 2017	Job No.: 2016-18
Drawn By: RWL	Checked By: GK
Approved By: CMS	Scale: N.T.S.

**Figure 1**

DISCLAIMER: This drawing is for illustration only and should not be used for design or construction purposes. All locations are approximate.



142 Chula Vista, San Antonio, Texas 78232  
 Phone: (210) 308-5884 • Fax: (210) 308-5886

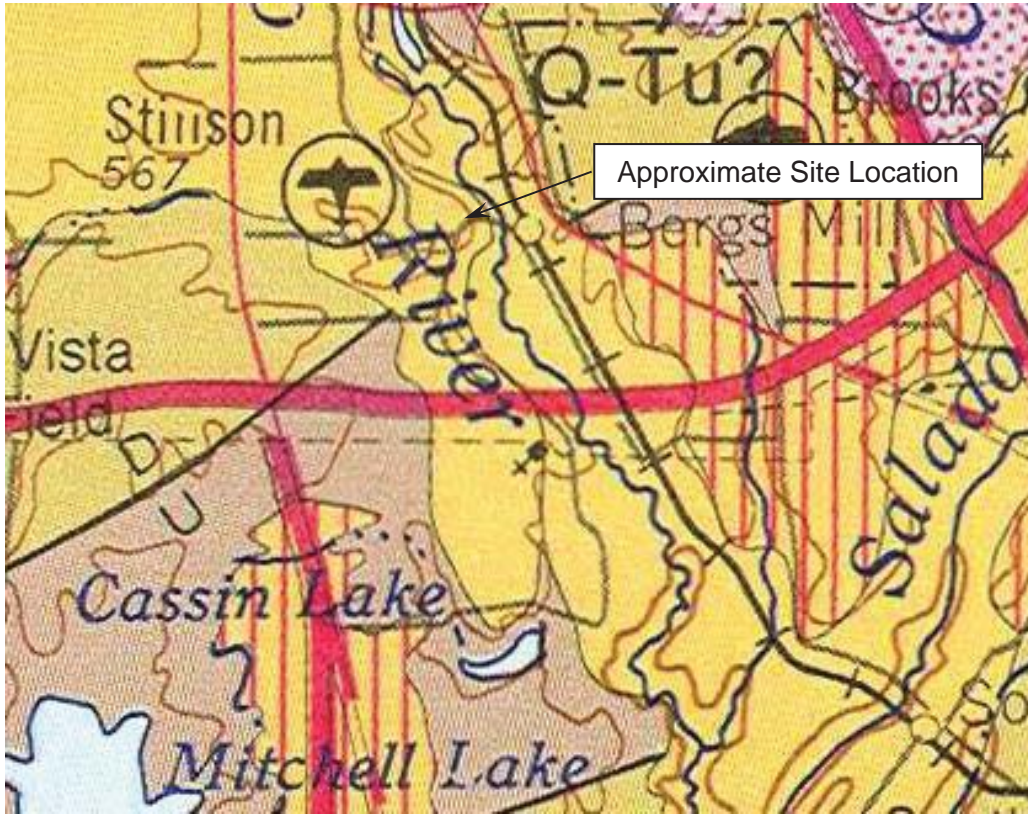
## BORING LOCATION PLAN

Stinson Hike and Bike Trails  
 San Antonio, Texas

Date: August 25, 2017	Job No.: 2016-18
Drawn By: GK	Checked By: GK
Approved By: CMS	Scale: N.T.S.

REVISIONS:		
No.:	Date:	Description:

**Figure 2**




PORTION OF GEOLOGIC ATLAS OF TEXAS  
(San Antonio Sheet)

**LEGEND**

<u>Symbol</u>	<u>Name</u>	<u>Age</u>
Qt	Fluviatile Terrace Deposits	Quaternary Period / Holocene
Q-tu	Uvalde Gravel	Quaternary Period / Pleistocene
Qle	Leona Formation	Quaternary Period / Pleistocene
Emi	Midway Group	Eocene
Ewi	Wilcox Group	Eocene



 Fault Segment with Indication of Relative Movement



142 Chula Vista, San Antonio, Texas 78232  
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**GEOLOGIC MAP**

Stinson Hike and Bike Trails  
San Antonio, Texas

Date: September 7, 2017	Job No.: 2016-18
Drawn By: RWL	Checked By: GK
Approved By: CMS	Scale: N.T.S.

**Figure 3**



Photo 1 – View looking at Boring 1 drilling operations.



Photo 2 – View looking at Boring 2 drilling operations.

DISCLAIMER: This drawing is for illustration only and should not be used for design or construction purposes. All locations are approximate.



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## SITE PHOTOS

Stinson Hike and Bike Trails  
San Antonio, Texas

### Appendix A

Date: September 7, 2017	Job No.: 2016-18
Drawn By: RWL	Checked By: GK
Approved By: CMS	Scale: N.T.S.



Photo 3 – View looking at Boring 3 drilling operations.



Photo 4 – View looking Boring 4 drilling operations.

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## SITE PHOTOS

Stinson Hike and Bike Trails  
 San Antonio, Texas

## Appendix A

Date: September 7, 2017	Job No.: 2016-18
Drawn By: RWL	Checked By: GK
Approved By: CMS	Scale: N.T.S.

## **APPENDIX B: BORING LOGS AND KEY TO TERMS**

# Boring Log No. B-1



Project: **Stinson Hike and Bike Trails**  
**San Antonio, Texas**

Sampling Date: 8/1/17

Elevation: 562.49 ft (By survey)

Coordinates: N29°20'22.7" W98°28'1.7"

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
CLAYEY SAND (SC), medium dense, light brown to tan	0 - 2	SS	6	16	35	19	25	48
	2 - 4	SS	6				30	
SANDY LEAN CLAY (CL), very hard, tan, with calcareous material  - hard from 6 to 8 feet  - very stiff below 8 feet	4 - 6	SS	6	17	33	16	69	57
	6 - 8	SS	11	15	38	23	33	54
	8 - 10	SS	8				24	

Borehole terminated at 10 feet

**Groundwater Data:**

During drilling: Not encountered

**Field Drilling Data:**

Coordinates: Hand-held GPS Unit  
 Logged By: R. Arizola  
 Driller: Eagle Drilling, Inc.  
 Equipment: Truck-mounted drill rig

Single flight auger: 0 - 10 ft

**Nomenclature Used on Boring Log**

Split Spoon (SS)

WC = Water Content (%)

PL = Plastic Limit

LL = Liquid Limit

PI = Plasticity Index

N = SPT Blow Count

-200 = % Passing #200 Sieve

2016-18.GPJ.11/27/17 (BORING LOG SA13-02,ARIASSA12-01.GDT,LIBRARY2013-01.GLB)

# Boring Log No. B-2



Project: **Stinson Hike and Bike Trails**  
**San Antonio, Texas**

Sampling Date: 8/1/17

Elevation: 536.73 ft (By survey)

Coordinates: N29°20'19.5" W98°27'48.8"

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
FILL: FAT CLAY with GRAVEL (CH), tan and gray								
FAT CLAY (CH), stiff, dark brown and light brown		SS	15				9	
	2							
		SS	12	17	52	35	11	
	4							
		SS	11				15	
	6							
POORLY GRADED GRAVEL with CLAY and SAND (GP-GC), medium dense, light brown to tan		SS	22	16	46	30	11	10
	8							
FAT CLAY (CH), stiff, brown and tan		SS	18	18	59	41	9	
	10							

Borehole terminated at 10 feet

**Groundwater Data:**


During drilling: Not encountered

**Field Drilling Data:**

Coordinates: Hand-held GPS Unit  
 Logged By: R. Arizola  
 Driller: Eagle Drilling, Inc.  
 Equipment: Truck-mounted drill rig

Single flight auger: 0 - 10 ft

**Nomenclature Used on Boring Log**

 Split Spoon (SS)

WC = Water Content (%)

-200 = % Passing #200 Sieve

PL = Plastic Limit

LL = Liquid Limit

PI = Plasticity Index

N = SPT Blow Count

2016-18.GPJ.11/27/17 (BORING LOG SA13-02,ARIASSA12-01.GDT,LIBRARY2013-01.GLB)

# Boring Log No. B-3



Project: **Stinson Hike and Bike Trails**  
**San Antonio, Texas**

Sampling Date: 8/1/17

Elevation: 537.54 ft (By survey)

Coordinates: N29°20'28.5" W98°27'38.6"

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200	DD	Uc
SANDY FAT CLAY (CH), stiff, light brown  - firm from 4.5 to 6.5 feet	0 - 5	SS	9	22	53	31	15			
	5 - 7	SS	7				12			
	7 - 9	SS	9				7			
	9 - 10	SS	9				15			
SANDY LEAN CLAY (CL), stiff to very stiff, light brown	10 - 14	SS	8	18	43	25	11	50		
	14 - 15	SS	14	17	43	26	9	50		
	15 - 17	T	17						103	2.69
CLAYEY SAND with GRAVEL (SC), loose, tan  - medium dense from 23 to 28 feet	20 - 25	SS	9				9	14		
	25 - 30	SS	18				23	15		
FAT CLAY (CH), very stiff, gray, with ferrous stain  (continued)	30 - 31	SS	26	25	85	60	16			

**Groundwater Data:**

First encountered during drilling: 22-ft depth  
 After : 20.4-ft depth

**Field Drilling Data:**

Coordinates: Hand-held GPS Unit  
 Logged By: R. Arizola  
 Driller: Eagle Drilling, Inc.  
 Equipment: Truck-mounted drill rig

Single flight auger: 0 - 40 ft

**Nomenclature Used on Boring Log**

Split Spoon (SS)

Thin-walled tube (T)

Water encountered during drilling

Delayed water reading

WC = Water Content (%)

PL = Plastic Limit

LL = Liquid Limit

PI = Plasticity Index

N = SPT Blow Count

-200 = % Passing #200 Sieve

DD = Dry Density (pcf)

Uc = Compressive Strength (tsf)

2016-18.GPJ.11/27/17 (BORING LOG SA13-02.ARIASSA12-01.GDT.LIBRARY2013-01.GLB)

# Boring Log No. B-3 (continued)



**Project: Stinson Hike and Bike Trails  
San Antonio, Texas**

**Sampling Date: 8/1/17**

**Elevation: 537.54 ft (By survey)**

**Coordinates: N29°20'28.5" W98°27'38.6"**

**Location: See Boring Location Plan**

**Backfill: Cuttings**

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200	DD	Uc
FAT CLAY (CH), very stiff, gray, with ferrous stain <i>(continued)</i>  - with gypsum crystals										
	35	SS	24				18			
	40	SS	26				25			

Borehole terminated at 40 feet

**Groundwater Data:**

First encountered during drilling: 22-ft depth  
After : 20.4-ft depth

**Field Drilling Data:**

Coordinates: Hand-held GPS Unit  
Logged By: R. Arizola  
Driller: Eagle Drilling, Inc.  
Equipment: Truck-mounted drill rig

Single flight auger: 0 - 40 ft

**Nomenclature Used on Boring Log**

Split Spoon (SS)

Thin-walled tube (T)

Water encountered during drilling

Delayed water reading

WC = Water Content (%)

PL = Plastic Limit

LL = Liquid Limit

PI = Plasticity Index

N = SPT Blow Count

-200 = % Passing #200 Sieve

DD = Dry Density (pcf)

Uc = Compressive Strength (tsf)

2016-18.GPJ.11/27/17 (BORING LOG SA13-02,ARIASSA12-01,GDT,LIBRARY2013-01.GLB)

# Boring Log No. B-4



Project: **Stinson Hike and Bike Trails**  
**San Antonio, Texas**

Sampling Date: 8/2/17

Elevation: 538.37 ft (By survey)

Coordinates: N29°20'28.2" W98°27'37"

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
CLAYEY GRAVEL with SAND (GC), dense, light gray and tan		SS	5	17	40	23		39	
		SS	3					42	
CLAYEY SAND (SC), medium dense, tan and light brown, with gravel	5	SS	9	16	38	22		24	37
		SS	10					27	
	10	SS	11					24	
SANDY LEAN CLAY (CL), stiff, dark brown, gray and tan, with ferrous stain  - presence of wood piece at 13 feet		SS	16	19	46	27		14	52
	15	T	16	18	51	33	1.25		74
CLAYEY SAND with Gravel (SC), medium dense, light gray and brown	20	T	15	21	46	25	1.75		35
CLAYEY GRAVEL with SAND (GC), medium dense, tan	25	SS	20					17	
FAT CLAY (CH), very stiff, gray and tan, with ferrous stain  (continued)	30	SS	29					20	

2016-18.GPJ.11/27/17.(BORING LOG SA13-02.ARIASSA12-01.GDT.LIBRARY2013-01.GLB)

**Groundwater Data:**

First encountered during drilling: 23-ft depth  
 After : 22.3-ft depth (23-ft open borehole depth)

**Field Drilling Data:**

Coordinates: Hand-held GPS Unit  
 Logged By: R. Arizola  
 Driller: Eagle Drilling, Inc.  
 Equipment: Truck-mounted drill rig  
 Single flight auger: 0 - 45 ft

**Nomenclature Used on Boring Log**

Split Spoon (SS)

Thin-walled tube (T)

Water encountered during drilling

Delayed water reading

WC = Water Content (%)

N = SPT Blow Count

PL = Plastic Limit

-200 = % Passing #200 Sieve

LL = Liquid Limit

PI = Plasticity Index

PP = Pocket Penetrometer (tsf)

# Boring Log No. B-4 (continued)



Project: **Stinson Hike and Bike Trails**  
**San Antonio, Texas**

Sampling Date: 8/2/17

Elevation: 538.37 ft (By survey)

Coordinates: N29°20'28.2" W98°27'37"

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FAT CLAY (CH), very stiff, gray and tan, with ferrous stain <i>(continued)</i>  - with gypsum crystal between 33 and 35 feet	35	SS	24					24	
	40	T	24	26	78	52	3		96
	45	SS	25					30	

Borehole terminated at 45 feet

**Groundwater Data:**

First encountered during drilling: 23-ft depth  
 After : 22.3-ft depth (23-ft open borehole depth)

**Field Drilling Data:**

Coordinates: Hand-held GPS Unit  
 Logged By: R. Arizola  
 Driller: Eagle Drilling, Inc.  
 Equipment: Truck-mounted drill rig  
 Single flight auger: 0 - 45 ft

**Nomenclature Used on Boring Log**

Split Spoon (SS)

Thin-walled tube (T)

Water encountered during drilling

Delayed water reading

WC = Water Content (%)

N = SPT Blow Count

PL = Plastic Limit

-200 = % Passing #200 Sieve

LL = Liquid Limit

PI = Plasticity Index

PP = Pocket Penetrometer (tsf)

2016-18.GPJ.11/27/17 (BORING LOG SA13-02,ARIASSA12-01.GDT,LIBRARY2013-01.GLB)

# Boring Log No. B-5



Project: **Stinson Hike and Bike Trails**  
**San Antonio, Texas**

Sampling Date: 8/1/17

Elevation: 551.65 ft (By survey)

Coordinates: N29°20'26.9" W98°28'5"

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
CLAYEY SAND with GRAVEL (SC), medium dense, brown	0 - 2	SS	6	20	49	29	18	30
	2 - 4	SS	7				20	
SANDY LEAN CLAY (CL), hard, light tan to tan  - stiff from 8.5 to 10 feet	4 - 6	SS	12	19	36	17	33	
	6 - 8	SS	11	16	27	11	24	58
	8 - 10	SS	11				11	
CLAYEY GRAVEL with SAND (GC), medium dense, tan	10 - 12	SS	11				23	
	12 - 14	SS	15				29	17

Borehole terminated at 15 feet

**Groundwater Data:**



First encountered during drilling: 13-ft depth  
 After : 11.3-ft depth (12-ft open borehole depth)

**Field Drilling Data:**

Coordinates: Hand-held GPS Unit  
 Logged By: R. Arizola  
 Driller: Eagle Drilling, Inc.  
 Equipment: Truck-mounted drill rig  
 Single flight auger: 0 - 15 ft

**Nomenclature Used on Boring Log**

 Split Spoon (SS)

 Water encountered during drilling  
 Delayed water reading

WC = Water Content (%)

-200 = % Passing #200 Sieve

PL = Plastic Limit

LL = Liquid Limit

PI = Plasticity Index

N = SPT Blow Count

2016-18.GPJ.11/27/17 (BORING LOG SA13-02.ARIASSA12-01.GDT.LIBRARY2013-01.GLB)

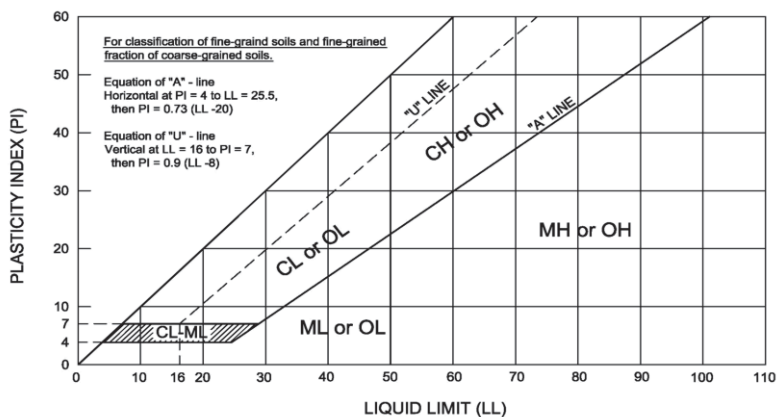
## KEY TO TERMS AND SYMBOLS USED ON BORING LOGS

MAJOR DIVISIONS			GROUP SYMBOLS	DESCRIPTIONS				
<b>COARSE-GRAINED SOILS</b>	More than half of material LARGER than No. 200 Sieve size	<b>GRAVELS</b>	Clean Gravels (little or no Fines)	<b>GW</b>	Well-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines			
			Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines	<b>GP</b>	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines			
			Silty Gravels, Gravel-Sand-Silt Mixtures	<b>GM</b>	Silty Gravels, Gravel-Sand-Silt Mixtures			
			Clayey Gravels, Gravel-Sand-Clay Mixtures	<b>GC</b>	Clayey Gravels, Gravel-Sand-Clay Mixtures			
		<b>SANDS</b>	More than half of Coarse fraction is SMALLER than No. 4 Sieve size	Clean Sands (little or no Fines)	<b>SW</b>	Well-Graded Sands, Gravelly Sands, Little or no Fines		
				Poorly-Graded Sands, Gravelly Sands, Little or no Fines	<b>SP</b>	Poorly-Graded Sands, Gravelly Sands, Little or no Fines		
				Silty Sands, Sand-Silt Mixtures	<b>SM</b>	Silty Sands, Sand-Silt Mixtures		
				Clayey Sands, Sand-Clay Mixtures	<b>SC</b>	Clayey Sands, Sand-Clay Mixtures		
				<b>SILTS &amp; CLAYS</b>	Liquid Limit less than 50	Inorganic Silts & Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity	<b>ML</b>	Inorganic Silts & Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity
						Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays	<b>CL</b>	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
Inorganic Silts, Micaceous or Diatomaceous Fine Sand or Silty Soils, Elastic Silts	<b>MH</b>	Inorganic Silts, Micaceous or Diatomaceous Fine Sand or Silty Soils, Elastic Silts						
Inorganic Clays of High Plasticity, Fat Clays	<b>CH</b>	Inorganic Clays of High Plasticity, Fat Clays						
<b>FORMATIONAL MATERIALS</b>	<b>SANDSTONE</b>		Massive Sandstones, Sandstones with Gravel Clasts	Massive Sandstones, Sandstones with Gravel Clasts				
	<b>MARLSTONE</b>		Indurated Argillaceous Limestones	Indurated Argillaceous Limestones				
	<b>LIMESTONE</b>		Massive or Weakly Bedded Limestones	Massive or Weakly Bedded Limestones				
	<b>CLAYSTONE</b>		Mudstone or Massive Claystones	Mudstone or Massive Claystones				
	<b>CHALK</b>		Massive or Poorly Bedded Chalk Deposits	Massive or Poorly Bedded Chalk Deposits				
	<b>MARINE CLAYS</b>		Cretaceous Clay Deposits	Cretaceous Clay Deposits				
<b>GROUNDWATER</b>			Indicates Final Observed Groundwater Level Indicates Initial Observed Groundwater Location	Indicates Final Observed Groundwater Level Indicates Initial Observed Groundwater Location				

Density of Granular Soils	
Number of Blows per ft., N	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
Over 50	Very Dense

Consistency and Strength of Cohesive Soils		
Number of Blows per ft., N	Consistency	Unconfined Compressive Strength, $q_u$ (tsf)
Below 2	Very Soft	Less than 0.25
2 - 4	Soft	0.25 - 0.5
4 - 8	Medium (Firm)	0.5 - 1.0
8 - 15	Stiff	1.0 - 2.0
15 - 30	Very Stiff	2.0 - 4.0
Over 30	Hard	Over 4.0

**PLASTICITY CHART (ASTM D 2487-11)**



# KEY TO TERMS AND SYMBOLS USED ON BORING LOGS

## TABLE 1 Soil Classification Chart (ASTM D 2487-11)

Criteria of Assigning Group Symbols and Group Names Using Laboratory Tests <sup>A</sup>				Soil Classification			
				Group Symbol	Group Name <sup>B</sup>		
<b>COARSE-GRAINED SOILS</b>	Gravels (More than 50% of coarse fraction retained on No. 4 sieve)	Clean Gravels (Less than 5% fines <sup>C</sup> )	$Cu \geq 4$ and $1 \leq Cc \leq 3^D$	GW	Well-Graded Gravel <sup>E</sup>		
			$Cu < 4$ and/or [ $Cc < 1$ or $Cc > 3$ ] <sup>D</sup>	GP	Poorly-Graded Gravel <sup>E</sup>		
	More than 50% retained on No. 200 sieve	Gravels with Fines (More than 12% fines <sup>C</sup> )	Fines classify as ML or MH		GM	Silty Gravel <sup>E,F,G</sup>	
			Fines classify as CL or CH		GC	Clayey Gravel <sup>E,F,G</sup>	
		Sands (50% or more of coarse fraction passes No. 4 sieve)	Clean Sands (Less than 5% fines <sup>H</sup> )	$Cu \geq 6$ and $1 \leq Cc \leq 3^D$ $Cu < 6$ and/or [ $Cc < 1$ or $Cc > 3$ ] <sup>D</sup>		SW	Well-Graded Sand <sup>I</sup>
			Sands with Fines (More than 12% fines <sup>H</sup> )	Fines classify as ML or MH Fines classify as CL or CH		SP	Poorly-Graded Sand <sup>I</sup>
<b>FINE-GRAINED SOILS</b>	Silt and Clays	inorganic	$PI > 7$ and plots on or above "A" line <sup>J</sup>	CL	Lean Clay <sup>K,L,M</sup>		
		Liquid limit less than 50	$PI < 4$ or plots below "A" line <sup>J</sup>	ML	Silt <sup>K,L,M</sup>		
	50% or more passes the No. 200 sieve	Silt and Clays	inorganic	Liquid limit - oven dried < 0.75 Liquid limit - not dried	OL	Organic Clay <sup>K,L,M,N</sup>	
			Liquid limit 50 or more	$PI$ plots on or above "A" line	CH	Organic Silt <sup>K,L,M,O</sup>	
		Silt and Clays	inorganic	$PI$ plots on or below "A" line		MH	Fat Clay <sup>K,L,M</sup>
			Liquid limit 50 or more	Liquid limit - oven dried < 0.75 Liquid limit - not dried		OH	Elastic Silt <sup>K,L,M</sup>
<b>HIGHLY ORGANIC SOILS</b>		Primarily organic matter, dark in color, and organic odor		PT	Peat		

<sup>A</sup> Based on the material passing the 3-inch (75mm) sieve

<sup>B</sup> If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name

<sup>C</sup> Gravels with 5% to 12% fines require dual symbols:

- GW-GM well-graded gravel with silt
- GW-GC well-graded gravel with clay
- GP-GM poorly-graded gravel with silt
- GP-GC poorly-graded gravel with clay

<sup>D</sup>  $Cu = D_{60}/D_{10}$        $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

<sup>E</sup> If soil contains  $\geq 15\%$  sand, add "with sand" to group name

<sup>F</sup> If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM

<sup>G</sup> If fines are organic, add "with organic fines" to group name

<sup>H</sup> Sand with 5% to 12% fines require dual symbols:

- SW-SM well-graded sand with silt
- SW-SC well-graded sand with clay
- SP-SM poorly-graded sand with silt
- SP-SC poorly-graded sand with clay

<sup>I</sup> If soil contains  $\geq 15\%$  gravel, add "with gravel" to group name

<sup>J</sup> If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay

<sup>K</sup> If soil contains 15% to < 30% plus No. 200, add "with sand" or "with gravel," whichever is predominant

<sup>L</sup> If soil contains  $\geq 30\%$  plus No. 200, predominantly sand, add "sandy" to group name

<sup>M</sup> If soil contains  $\geq 30\%$  plus No. 200, predominantly gravel, add "gravelly" to group name

<sup>N</sup>  $PI \geq 4$  and plots on or above "A" line

<sup>O</sup>  $PI < 4$  or plots below "A" line

<sup>P</sup>  $PI$  plots on or above "A" line

<sup>Q</sup>  $PI$  plots below "A" line

### TERMINOLOGY

<b>Boulders</b>	Over 12-inches (300mm)	<b>Parting</b>	Inclusion < 1/8-inch thick extending through samples
<b>Cobbles</b>	12-inches to 3-inches (300mm to 75mm)	<b>Seam</b>	Inclusion 1/8-inch to 3-inches thick extending through sample
<b>Gravel</b>	3-inches to No. 4 sieve (75mm to 4.75mm)	<b>Layer</b>	Inclusion > 3-inches thick extending through sample
<b>Sand</b>	No. 4 sieve to No. 200 sieve (4.75mm to 0.075mm)		
<b>Silt or Clay</b>	Passing No. 200 sieve (0.075mm)		
<b>Calcareous</b>	Containing appreciable quantities of calcium carbonate, generally nodular		
<b>Stratified</b>	Alternating layers of varying material or color with layers at least 6mm thick		
<b>Laminated</b>	Alternating layers of varying material or color with the layers less than 6mm thick		
<b>Fissured</b>	Breaks along definite planes of fracture with little resistance to fracturing		
<b>Slickensided</b>	Fracture planes appear polished or glossy sometimes striated		
<b>Blocky</b>	Cohesive soil that can be broken down into small angular lumps which resist further breakdown		
<b>Lensed</b>	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay		
<b>Homogeneous</b>	Same color and appearance throughout		

## **APPENDIX C: FIELD AND LABORATORY EXPLORATION**

## FIELD AND LABORATORY EXPLORATION

The field exploration program included drilling at selected locations within the site and intermittently sampling the encountered materials. The boreholes were drilled using single flight auger (ASTM D 1452). Samples of encountered materials were obtained using a split-barrel sampler while performing the Standard Penetration Test (ASTM D 1586) or using a thin-walled tube sampler (ASTM D 1587). The sample depth interval and type of sampler used is included on the soil boring log. Arias' field representative visually logged each recovered sample and placed a portion of the recovered sampled into a plastic bag for transport to our laboratory.

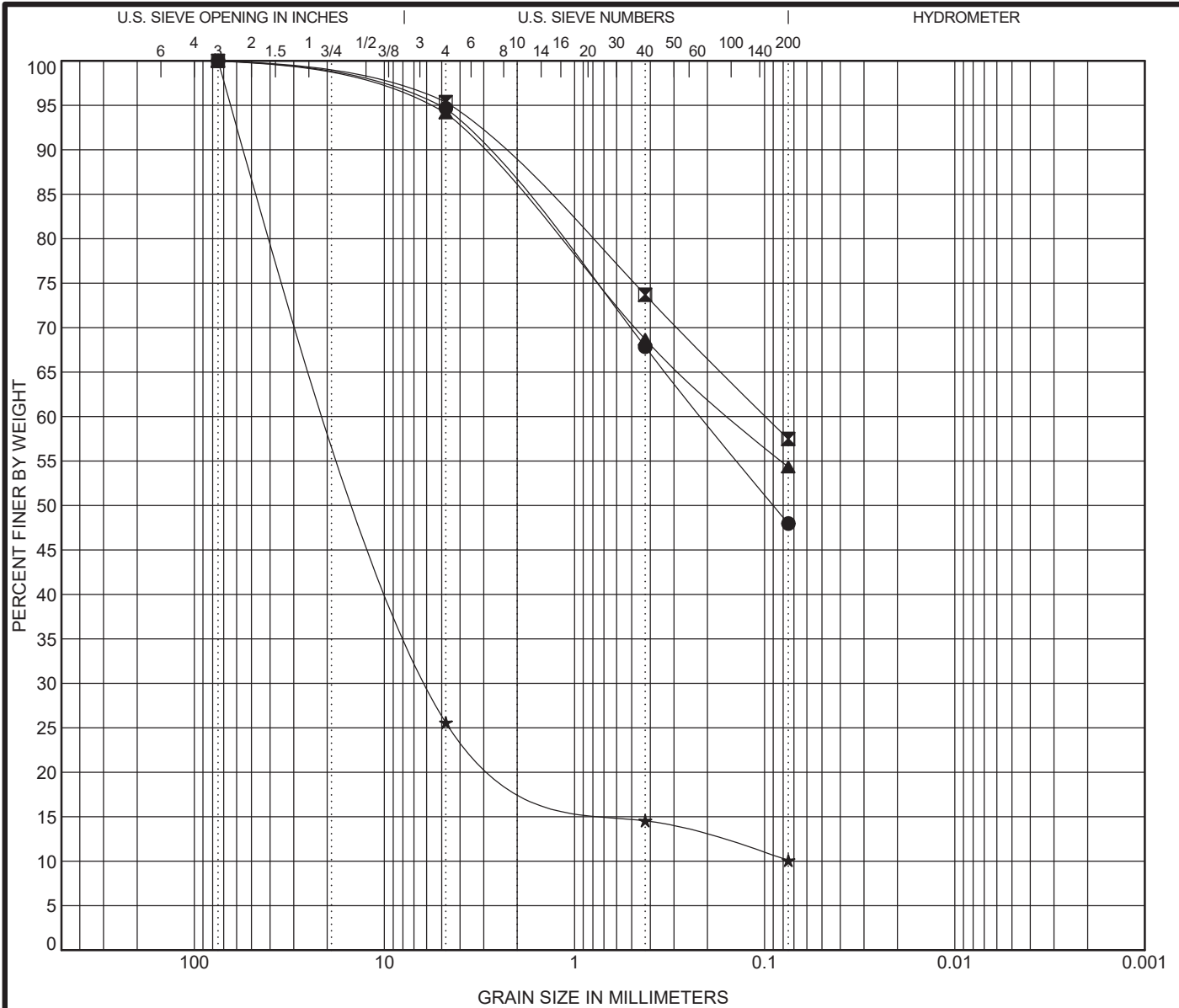
SPT N values and blow counts for those intervals where the sampler could not be advanced for the required 18-inch penetration are shown on the soil boring log. If the test was terminated during the 6-inch seating interval or after 10 hammer blows were applied used and no advancement of the sampler was noted, the log denotes this condition as blow count during seating penetration. Penetrometer readings recorded for thin-walled tube samples that remained intact also are shown on the soil boring log.

Arias performed soil mechanics laboratory tests on selected samples to aid in soil classification and to determine engineering properties. Tests commonly used in geotechnical exploration, the method used to perform the test, and the column designation on the boring log where data are reported are summarized as follows:

Test Name	Test Method	Log Designation
Water (moisture) content of soil and rock by mass	ASTM D 2216	WC
Liquid limit, plastic limit, and plasticity index of soils	ASTM D 4318	PL, LL, PI
Amount of material in soils finer than the No. 200 sieve	ASTM D 1140	-200
Unconfined Compressive Strength of Soil	ASTM D 2166	Uc

The laboratory results are reported on the soil boring log.

## **APPENDIX D: GRAIN SIZE DISTRIBUTION CURVES**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	1	0.5	CLAYEY SAND (SC)	35	16	19		
■	1	4.5	SANDY LEAN CLAY (CL)	33	17	16		
▲	1	6.5	SANDY LEAN CLAY (CL)	38	15	23		
★	2	6.5	POORLY GRADED GRAVEL with CLAY and SAND (GP-GC)	46	16	30	25.32	234.17

Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	1	75	0.214			5.3	46.7	48.0	
■	1	75	0.098			4.6	37.9	57.5	
▲	1	75	0.149			5.8	39.8	54.3	
★	2	75	17.02	5.596		74.4	15.5	10.1	

Silt and clay fractions were determined using 0.002 mm as the maximum particle size for clay.

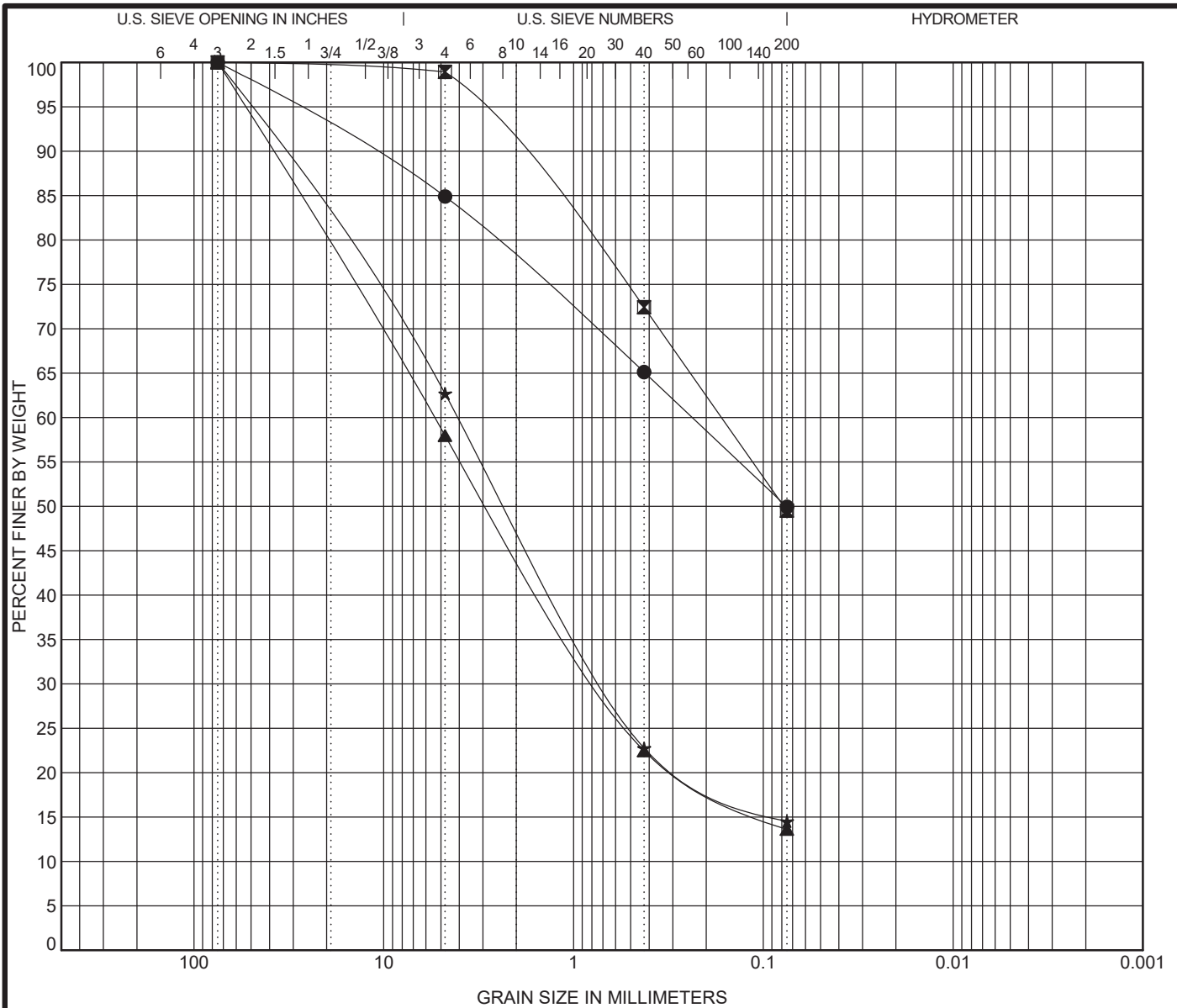


142 Chula Vista  
 San Antonio, Texas 78232  
 Phone: (210) 308-5884  
 Fax: (210) 308-5886

### GRAIN SIZE DISTRIBUTION

Project: Stinson Hike and Bike Trails  
 Location: See Boring Location Plan  
 Job No.: 2016-18

2016-18.GPJ.9/7/17 (GRAIN SIZE ARIAS, US LAB.GDT.LIBRARY/2013-01.GLB)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	3	8.5	CLAYEY SAND with GRAVEL (SC)	43	18	25		
☒	3	10.5	SANDY LEAN CLAY (CL)	43	17	26		
▲	3	18.5	CLAYEY SAND with GRAVEL (SC)					
★	3	23.5	CLAYEY SAND with GRAVEL (SC)					

Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	3	75	0.237			15.1	35.0	49.9	
☒	3	75	0.166			1.1	49.4	49.5	
▲	3	75	5.426	0.71		42.0	44.4	13.6	
★	3	75	4.037	0.658		37.3	48.2	14.5	

Silt and clay fractions were determined using 0.002 mm as the maximum particle size for clay.

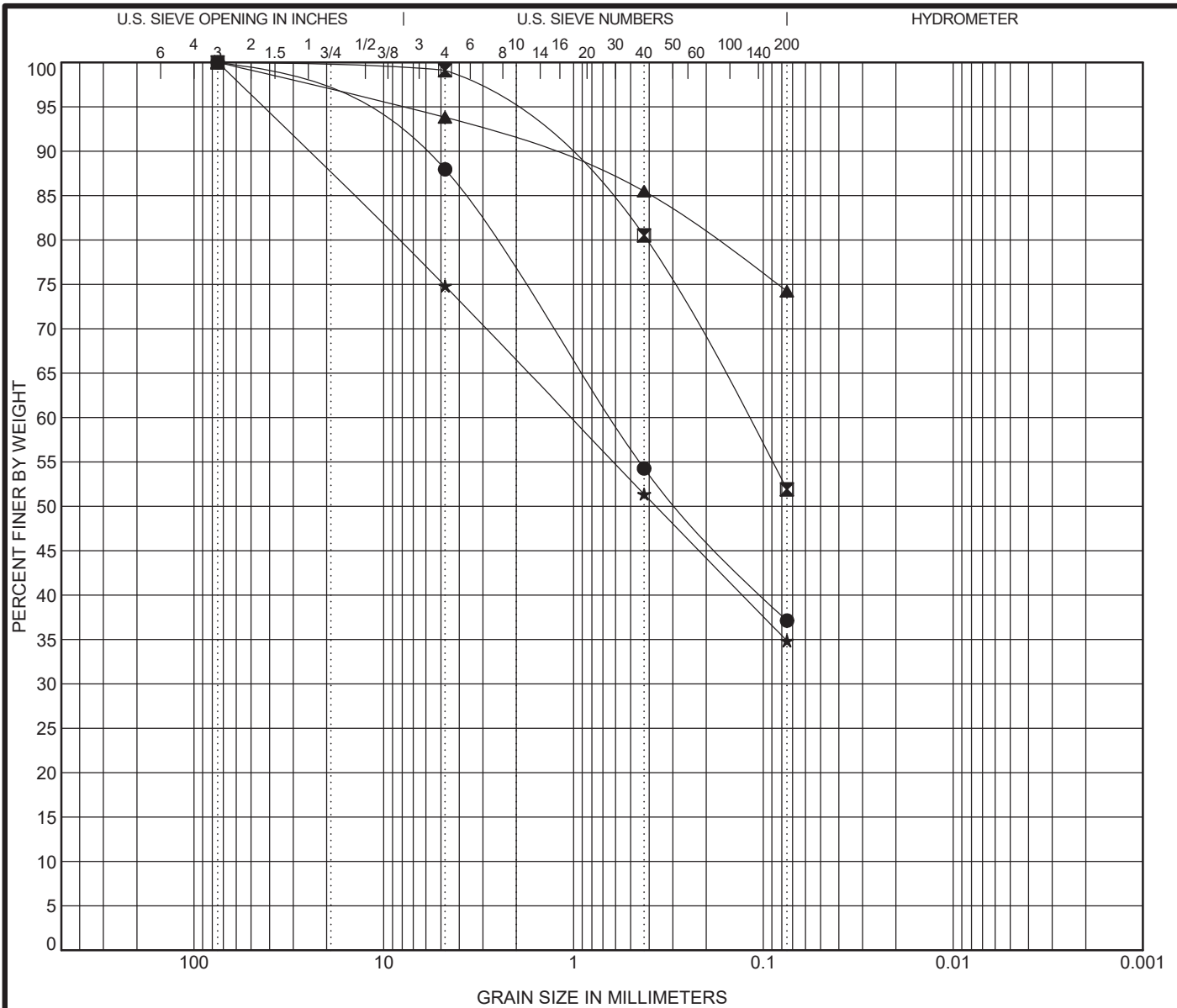


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 San Antonio, Texas 78232  
 Phone: (210) 308-5884  
 Fax: (210) 308-5886

**GRAIN SIZE DISTRIBUTION**

Project: Stinson Hike and Bike Trails  
 Location: See Boring Location Plan  
 Job No.: 2016-18

2016-18.GPJ.9/7/17 (GRAIN SIZE ARIAS,US\_LAB.GDT.LIBRARY/2013-01.GLB)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	4	4.5	CLAYEY SAND (SC)	38	16	22		
⊠	4	10.5	SANDY LEAN CLAY (CL)	46	19	27		
▲	4	13.0	FAT CLAY with SAND (CH)	51	18	33		
★	4	18.0	CLAYEY SAND with GRAVEL (SC)	46	21	25		

Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	4	4.5	75	0.641		12.0	50.8	37.1	
⊠	4	10.5	75	0.122		0.9	47.2	51.9	
▲	4	13.0	75			6.2	19.6	74.2	
★	4	18.0	75	1.033		25.2	40.0	34.9	

Silt and clay fractions were determined using 0.002 mm as the maximum particle size for clay.

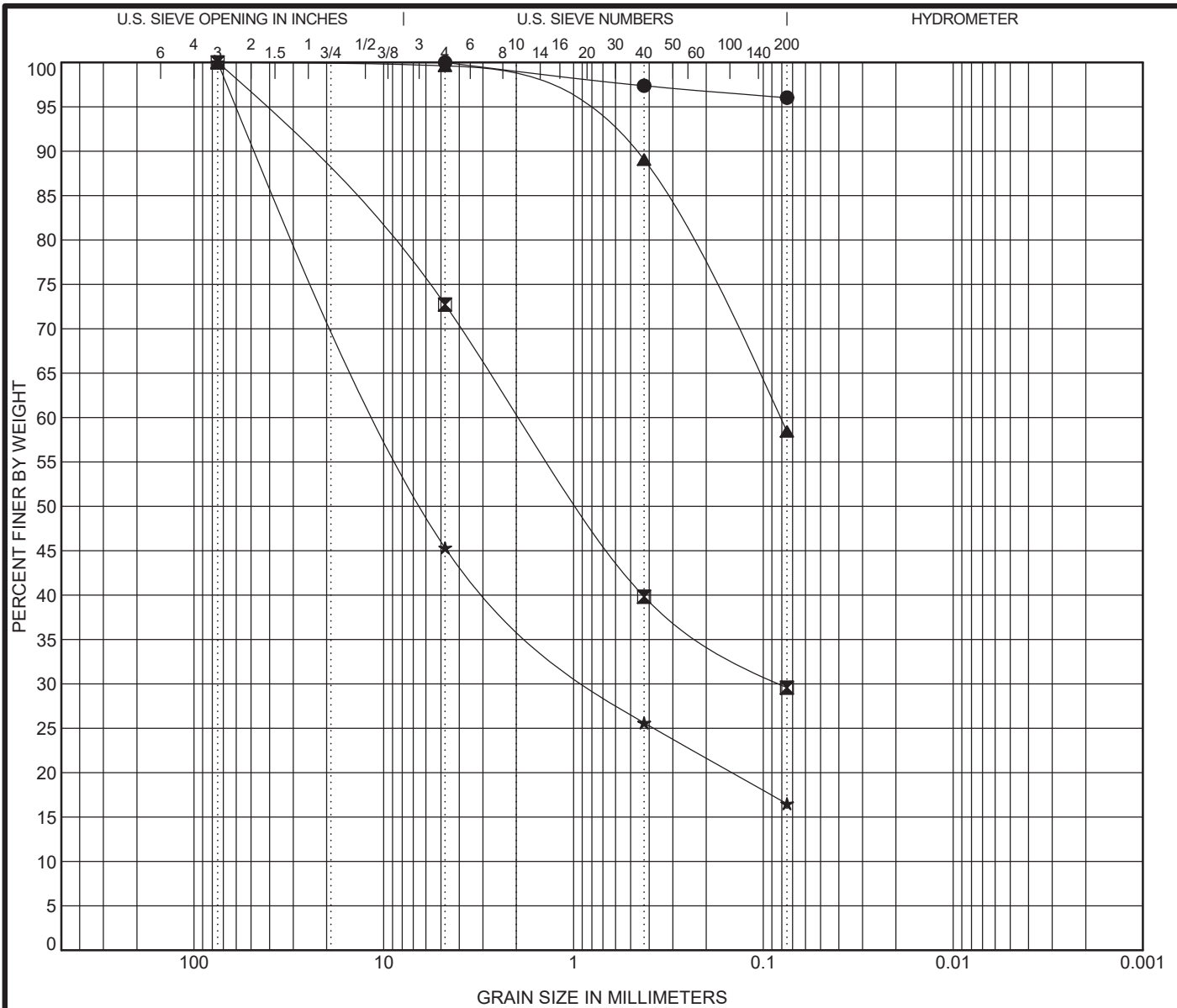


142 Chula Vista  
 San Antonio, Texas 78232  
 Phone: (210) 308-5884  
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**GRAIN SIZE DISTRIBUTION**

Project: Stinson Hike and Bike Trails  
 Location: See Boring Location Plan  
 Job No.: 2016-18

2016-18.GPJ.9/7/17 (GRAIN SIZE ARIAS, US LAB.GDT.LIBRARY/2013-01.GLB)




COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	4	38.0	<b>FAT CLAY (CH)</b>	78	26	52		
☒	5	0.5	<b>CLAYEY SAND with GRAVEL (SC)</b>	49	20	29		
▲	5	6.5	<b>SANDY LEAN CLAY (CL)</b>	27	16	11		
★	5	13.5	<b>CLAYEY GRAVEL with SAND (GC)</b>					

Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	4	4.75				0.0	4.0	96.0	
☒	5	75	1.867	0.081		27.3	43.2	29.6	
▲	5	75	0.082			0.4	41.2	58.4	
★	5	75	9.964	0.728		54.7	28.8	16.5	

Silt and clay fractions were determined using 0.002 mm as the maximum particle size for clay.

 <p>142 Chula Vista San Antonio, Texas 78232 Phone: (210) 308-5884 Fax: (210) 308-5886</p>	<b>GRAIN SIZE DISTRIBUTION</b>	
	Project: Stinson Hike and Bike Trails	
	Location: See Boring Location Plan	
	Job No.: 2016-18	

2016-18.GPJ.9/7/17 (GRAIN SIZE ARIAS, US LAB.GDT.LIBRARY/2013-01.GLB)

**APPENDIX E: TRAIL AND BRIDGE LAY OUT, AND BOX CULVERT  
PLAN AND PROFILE**







115 E. FRANKLIN STREET SUITE 800  
SAN ANTONIO, TX 78205  
PHONE: (214) 343-1111  
FAX: (214) 343-1111  
WWW.GDLG.COM

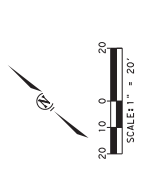
**STINSON  
MUNICIPAL AIRPORT  
SAN ANTONIO, TEXAS**



**STINSON  
HIKE AND BIKE  
TRAIL**

**PRELIMINARY DESIGN**

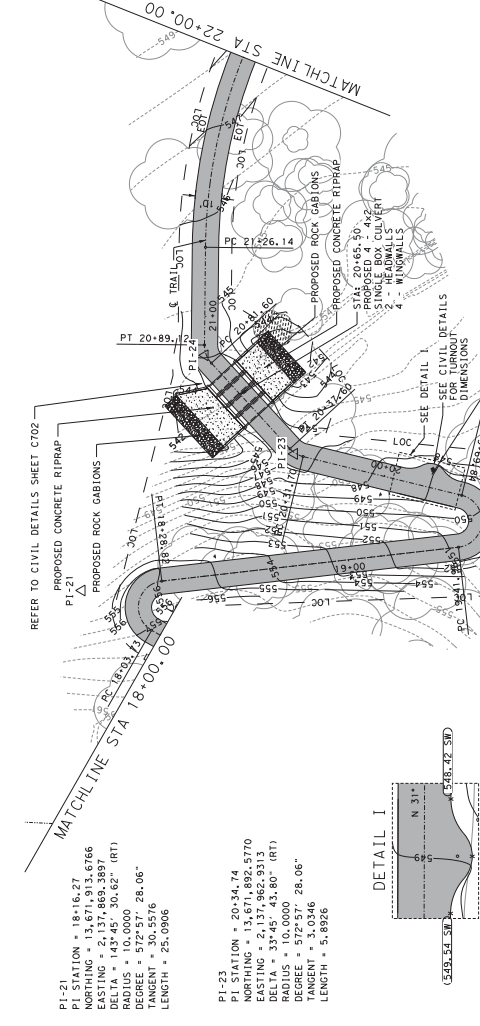
PROPOSED DOCUMENT  
INTERIM PLAN ONLY  
NO FIELD VERIFICATION  
CONSTRUCTION  
START DATE: 01/01/2017  
DATE REVISION: 01/01/2017



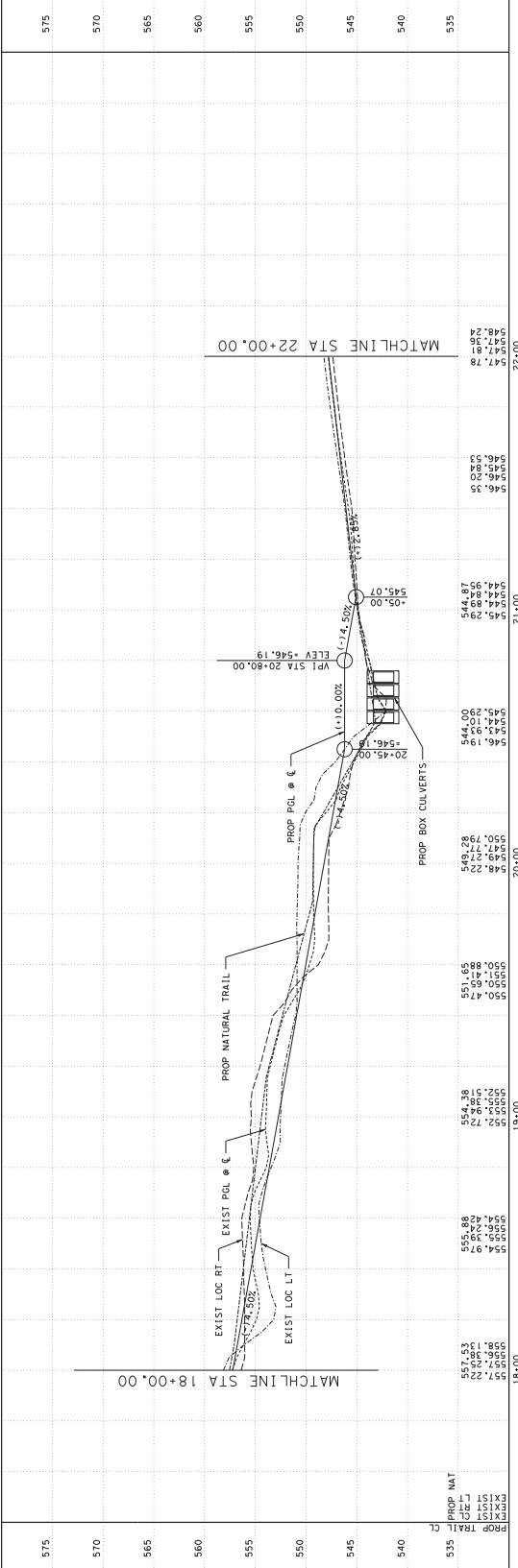
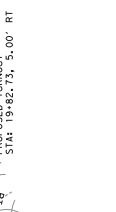
- LEGEND**
- LOC --- LIMITS OF CONSTRUCTION
  - 564 --- EXISTING CONTOUR
  - 562 --- PROPOSED CONTOUR
  - EOT --- EDGE OF TRAIL
  - PROPOSED CONCRETE TRAIL
  - EXISTING TREE
  - SIDEWALK ELEVATION

PI-21 STATION = 20+95.55  
NORTHING = 13,671,943.5139  
EASTING = 2,137,965.2417  
DELTA = 43° 05' 22.39" (RT)  
RADIUS = 174,557.00  
DEGREE = 572° 57' 28.06"  
TANGENT = 3,9481  
LENGTH = 7,5205

PI-22 STATION = 15+50.05  
NORTHING = 13,671,943.5139  
EASTING = 2,138,025.6822  
DELTA = 159° 44' 55.79" (LT)  
RADIUS = 10,000  
DEGREE = 572° 57' 28.06"  
TANGENT = 21,8814  
LENGTH = 21,8814



- NOTES:**
- NORTH LOOP BASE BID SHALL BE FOR CONCRETE TRAIL AS SHOWN ON PLANS. ALTERNATE BID SHALL BE FOR NATURAL TRAIL FOLLOWING THE PROPOSED HORIZONTAL AND VERTICAL ALIGNMENTS. CLEARING, BGP PLANS, MULCHING, REMOVAL OF MATERIAL, AND MINIMAL GRADING. TREE MULCH SHALL BE SPREAD ON TOP OF NATURAL TRAIL.
  - NATURAL TRAIL PROFILE IS DEPICTED BY THE PROPOSED PGL # 6 EXCEPT BETWEEN STA 18+25.00 TO STA 21+05.00.



STATION	ELEVATION
18+00	535.00
18+05	535.00
18+10	535.00
18+15	535.00
18+20	535.00
18+25	535.00
18+30	535.00
18+35	535.00
18+40	535.00
18+45	535.00
18+50	535.00
18+55	535.00
18+60	535.00
18+65	535.00
18+70	535.00
18+75	535.00
18+80	535.00
18+85	535.00
18+90	535.00
18+95	535.00
19+00	535.00
19+05	535.00
19+10	535.00
19+15	535.00
19+20	535.00
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19+75	535.00
19+80	535.00
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20+80	535.00
20+85	535.00
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20+95	535.00
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21+10	535.00
21+15	535.00
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21+50	535.00
21+55	535.00
21+60	535.00
21+65	535.00
21+70	535.00
21+75	535.00
21+80	535.00
21+85	535.00
21+90	535.00
21+95	535.00
22+00	535.00

STATION	ELEVATION
18+00	545.00
18+05	545.00
18+10	545.00
18+15	545.00
18+20	545.00
18+25	545.00
18+30	545.00
18+35	545.00
18+40	545.00
18+45	545.00
18+50	545.00
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21+60	545.00
21+65	545.00
21+70	545.00
21+75	545.00
21+80	545.00
21+85	545.00
21+90	545.00
21+95	545.00
22+00	545.00

DATE ISSUED: FEBRUARY 22, 2017  
REVIEWED BY: AG  
DRAWN BY: EV  
DESIGNED BY: AR

SHEET TITLE  
**NORTH LOOP  
PLAN AND PROFILE  
STA 18+00.00 TO  
STA 22+00.00**

SHEET NUMBER  
**C312**

**APPENDIX F: ASFE INFORMATION – GEOTECHNICAL REPORT**

# Important Information about Your Geotechnical Engineering Report

*Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.*

*While you cannot eliminate all such risks, you can manage them. The following information is provided to help.*

## **Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects**

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply the report for any purpose or project except the one originally contemplated.*

## **Read the Full Report**

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

## **A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors**

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,

- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

## **Subsurface Conditions Can Change**

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

## **Most Geotechnical Findings Are Professional Opinions**

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

## **A Report's Recommendations Are *Not* Final**

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

### **A Geotechnical Engineering Report Is Subject to Misinterpretation**

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

### **Do Not Redraw the Engineer's Logs**

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

### **Give Contractors a Complete Report and Guidance**

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time to perform additional study.* Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

### **Read Responsibility Provisions Closely**

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that

have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

### **Geoenvironmental Concerns Are Not Covered**

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

### **Obtain Professional Assistance To Deal with Mold**

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

### **Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance**

Membership in ASFE/THE BEST PEOPLE ON EARTH exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.



8811 Colesville Road/Suite G106, Silver Spring, MD 20910

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e-mail: info@asfe.org www.asfe.org

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IIGER06085.0MRP

## **APPENDIX G: QUALITY ASSURANCE TESTING**

# A Message to Owners

Construction materials engineering and testing (CoMET) consultants perform quality-assurance (QA) services to evaluate the degree to which constructors are achieving the specified conditions they're contractually obligated to achieve. Done right, QA can save you time and money; prevent unanticipated-conditions claims, change orders, and disputes; and reduce short-term and long-term risks, especially by detecting molehills before they grow into mountains.

**Done right, QA can save you time and money; prevent claims and disputes; and reduce risks. Many owners don't do QA right because they follow bad advice.**

Many owners don't do QA right because they follow bad advice; e.g., "CoMET consultants are all the same. They all have accredited facilities and certified personnel. Go with the low bidder." But there's no such thing as a standard QA scope of service, meaning that – to bid low – each interested firms *must* propose the cheapest QA service it can live with, jeopardizing service quality and aggravating risk for the entire project team. Besides, the advice is based on misinformation.

Fact: ***Most CoMET firms are not accredited,*** and the quality of those that are varies significantly. Accreditation – which is important – nonetheless means that a facility met an accrediting body's minimum criteria. Some firms practice at a much higher level; others just barely scrape by. And what an accrediting body typically evaluates – management, staff, facilities, and equipment – can change substantially before the next review, two, three, or more years from now.

**Most CoMET firms are not accredited. It's dangerous to assume CoMET personnel are certified.**

Fact: ***It's dangerous to assume CoMET personnel are certified.*** Many have no credentials at all; some are certified by organizations of questionable merit, while others have a valid certification, but *not* for the services they're assigned.

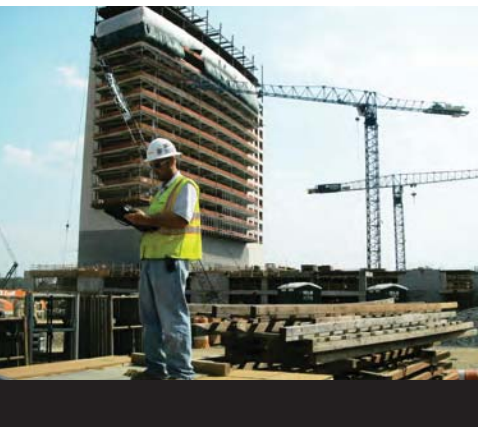
Some CoMET firms – the "low-cost providers" – *want* you to believe that price is the only difference between QA providers. It's not, of course. Firms that sell low price typically lack the facilities, equipment, personnel, and insurance quality-oriented firms invest in to achieve the reliability concerned owners need to achieve quality in quality assurance.

**ASFE** THE GEOPROFESSIONAL  
BUSINESS ASSOCIATION

8811 Colesville Road  
Suite G106  
Silver Spring, Maryland 20910  
Voice: 301.565.2733  
Fax: 301.589.2017  
E-mail: [info@asfe.org](mailto:info@asfe.org)  
Internet: [www.asfe.org](http://www.asfe.org)



Firms that sell **low price typically lack the facilities, equipment, personnel,** and insurance quality-oriented firms invest in to achieve the reliability concerned owners need to achieve quality in quality assurance.



To derive maximum value from your investment in QA, require the CoMET firm's project manager to serve actively on the project team from beginning to end, a level of service that's relatively inexpensive and can pay huge dividends. During the project's planning and design stages, experienced CoMET professionals can help the design team develop uniform technical specifications and establish appropriate observation, testing, and instrumentation procedures and protocols. They can also analyze plans and specs much as constructors do, looking for the little errors, omissions, conflicts, and ambiguities that often become the basis for big extras and big claims. They can provide guidance about operations that need closer review than others, because of their criticality or potential for error or abuse. They can also relate their experience with the various constructors that have expressed interest in your project.

To derive maximum value, **require the project manager to serve actively on the project team from beginning to end.**

CoMET consultants' construction-phase QA services focus on two distinct issues: those that relate to geotechnical engineering and those that relate to the other elements of construction.

The geotechnical issues are critically important because they are essential to the "observational method" geotechnical engineers use to significantly reduce the amount of sampling they'd otherwise require. They apply the observational method by developing a sampling plan for a project, and then assigning field representatives to ensure

samples are properly obtained, packaged, and transported. The engineers review the samples and, typically, have them tested in their own laboratories. They use the information they derive to characterize the site's subsurface and develop *preliminary* recommendations for the structure's foundations and for the specifications of various "geo" elements, like excavations, site grading, foundation-bearing grades, and roadway and parking-lot preparation and surfacing.

**Geotechnical engineers cannot finalize their recommendations until they or their field representatives are on site to observe what's excavated to verify that the subsurface conditions the engineers predicted are those that actually exist.**

When unanticipated conditions are observed, recommendations and/or specifications should be modified.

Responding to client requests, many geotechnical-engineering firms have expanded their field-services mix, so they're able to perform overall construction QA, encompassing – in addition to geotechnical issues – reinforced concrete, structural steel, welds, fireproofing, and so on. Unfortunately, that's caused some confusion. Believing that all CoMET consultants are alike, some owners take bids for the overall CoMET package, including the geotechnical field observation. *Entrusting geotechnical field observation to someone other than the geotechnical engineer of record (GER) creates a significant risk.*

Geotechnical engineers cannot finalize their recommendations until they are on site to verify that the subsurface conditions they predicted are those that actually exist. **Entrusting geotechnical field observation to someone other than the geotechnical engineer of record (GER) creates a significant risk.**

GERs have developed a variety of protocols to optimize the quality of their field-observation procedures. Quality-focused GERs meet with their field representatives before they leave for a project site, to brief them on what to look for and where, when, and how to look. (*No one can duplicate this briefing*, because no one else knows as much about a project’s geotechnical issues.) And once they arrive at a project site, the field representatives know to maintain timely, effective communication with the GER, because that’s what the GER has trained them to do. By contrast, it’s extremely rare for a different firm’s field personnel to contact the GER, even when they’re concerned or confused about what they observe, because they regard the GER’s firm as “the competition.”

Divorcing the GER from geotechnical field operations is almost always penny-wise and pound-foolish. Still, because owners are given bad advice, it’s commonly done, helping to explain why *“geo” issues are the number-one source of construction-industry claims and disputes.*

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To derive the biggest bang for the QA buck, identify three or even four quality-focused CoMET consultants. (If you don’t know any,

use the “Find a Geoprofessional” service available free at [www.asfe.org](http://www.asfe.org).) Ask about the firms’ ongoing and recent projects and the clients and client representatives involved; *insist upon receiving verification of all claimed accreditations, certifications, licenses, and insurance coverages.*

**Insist upon receiving verification of all claimed accreditations, certifications, licenses, and insurance coverages.**

Once you identify the two or three most qualified firms, meet with their representatives, preferably at their own facility, so you can inspect their laboratory, speak with management and technical staff, and form an opinion about the firm’s capabilities and attitude.

Insist that each firm’s designated project manager participate in the meeting. You will benefit when that individual is a seasoned QA professional familiar with construction’s rough-and-tumble. Ask about others the firm will assign, too. There’s no substitute for experienced personnel who are familiar with the codes and standards involved and know how to:

- read and interpret plans and specifications;
- perform the necessary observation, inspection, and testing;
- document their observations and findings;
- interact with constructors’ personnel; and
- respond to the unexpected.

*Important:* Many of the services CoMET QA field representatives perform – like observing operations and outcomes – require the good judgment afforded by extensive training and experience, especially in situations where standard operating procedures do not apply. You need to know who will be exercising that judgment: a 15-year “veteran” or a rookie?

Many of the services **CoMET QA field representatives perform** require good judgment.

Also consider the tools CoMET personnel use. Some firms are passionate about proper calibration; others, less so. Passion is a good thing! Ask to see the firm's calibration records. If the firm doesn't have any, or if they are not current, be cautious. *You cannot trust test results derived using equipment that may be out of calibration.* Also ask a firm's representatives about their reporting practices, including report distribution, how they handle notifications of nonconformance, and how they resolve complaints.

**Scope flexibility is needed to deal promptly with the unanticipated.**

For financing purposes, some owners require the constructor to pay for CoMET services. **Consider an alternative approach** so you don't convert the constructor into the CoMET consultant's client. If it's essential for you to fund QA via the constructor, have the CoMET fee included as an allowance in the bid documents. This arrangement ensures that you remain the CoMET consultant's client, and it prevents the CoMET fee from becoming part of the constructor's bid-price competition. (Note that the International Building Code (IBC) *requires the owner to pay* for Special Inspection (SI) services commonly performed by the CoMET consultant as a service separate from QA, to help ensure the SI services' integrity. Because failure to comply could result in denial of an occupancy or use permit, having a contractual agreement that conforms to the IBC mandate is essential.)

If it's essential for you to fund QA via the constructor, **have the CoMET fee included as an allowance in the bid documents.** Note, too, that the International Building Code (IBC) **requires the owner to pay for Special Inspection (SI) services.**

CoMET consultants can usually quote their fees as unit fees, unit fees with estimated total (invoiced on a unit-fee basis), or lump-sum (invoiced on a percent-completion basis referenced to a schedule of values). No matter which method is used, estimated quantities need to be realistic. Some CoMET firms lower their total-fee estimates by using quantities they know are too low and then request change orders long before QA is complete.

Once you and the CoMET consultant settle on the scope of service and fee, enter into a written contract. Established CoMET firms have their own contracts; most owners sign them. Some owners prefer to use different contracts, but that can be a mistake when the contract was prepared for construction services. *Professional services are different.* Wholly avoidable problems occur when a contract includes provisions that don't apply to the services involved and fail to include those that do.

**Some owners create wholly avoidable problems by using a contract prepared for construction services.**





This final note: CoMET consultants perform QA for owners, not constructors. While constructors are commonly allowed to review QA reports as a *courtesy*, you need to make it clear that constructors do *not* have a legal right to rely on those reports; i.e., if constructors want to forgo their own observation and testing and rely on results derived from a scope created to meet *only* the needs of the owner, they

*must do so at their own risk.* In all too many cases where owners have not made that clear, some constructors have alleged that they did have a legal right to rely on QA reports and, as a result, the CoMET consultant – not they – are responsible for their failure to deliver what they contractually promised to provide. The outcome can be delays and disputes that entangle you and all other principal project participants. Avoid that. Rely on a CoMET firm that possesses the resources and attitude needed to manage this and other risks as an element of a quality-focused service. Involve the firm early. Keep it engaged. And listen to what the CoMET consultant says. A good CoMET consultant can provide great value.

For more information, speak with your ASFE-Member CoMET consultant or contact ASFE directly.



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