Geotechnical Engineering Study

Martin Luther King Park Improvements
San Antonio, Texas

Arias Job No. 2015-1018

ARIAS
GEOPROFESSIONALS

Prepared For
Halff Associates, Inc.

August 19, 2016

ARIAS GEOPROFESSIONALS

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Via Email: eHerolt@halff.com

Mr. Ed Herolt, P.E.
Halff Associates, Inc.
300 E. Sonterra Blvd., Suite 230
San Antonio, Texas 78258

RE: Geotechnical Engineering Study
Martin Luther King Park Improvements
San Antonio, Texas

Dear Mr. Herolt:

Arias Geoprofessionals, Inc. (Arias) is pleased to submit the results of a Geotechnical Engineering Study for the above referenced project. Our services were performed as outlined in our proposal, dated December 2, 2015, and formally authorized in the Standard Agreement for Professional Services, executed January 23, 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 F.

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

STATE OF TEXAS
TIMOTHY J. FOX
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INTRODUCTION

This report presents the results of a Geotechnical Engineering Study for the proposed Martin Luther King (MLK) Park Improvements in San Antonio, Texas. We understand that the proposed project will consist of various upgrades and improvements to provide access through the MLK Park from the south side of the Salado Creek, to the north through the Wheatley Heights Sports Complex lease area. Preliminary plans include the removal of the existing low water crossing and installation of a new bridge structure at Salado Creek, roadway alignment adjustments, design of a new monuments/entry sign at MLK Park off ML King Drive, and modifications to the existing sign off Houston Street. A Vicinity Map is included as Figure 1 in Appendix A.

This project was authorized by means of Standard Subcontract for Engineering Services between Halff Associates, Inc. (Halff) and Arias Geoprofessionals (Arias), dated January 13, 2016. Our scope of services was performed in general accordance with Arias Proposal No. 2015-1018, dated December 2, 2015, which was included as Attachment 2 to the subcontract agreement noted above.

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:

- perform a pavement design for the planned roadway reconstruction to include options for flexible and rigid pavements; and
- develop design parameters for drilled shafts (piers) to support the new bridge structure that include: allowable side friction and end bearing for axial loading and Lpile parameters for lateral loading.

Environmental studies were not a part of our scope of services. Additionally, slope stability and/or global stability analysis of slopes or retaining structures was also beyond our authorized service scope.

FIELD EXPLORATION

Seven (7) soil borings were drilled to depths ranging from about 10 to 50 feet between April 26 and 29, 2016, at the approximate locations shown on the Boring Location Plans, Figures 2a and 2b provided in Appendix A. In addition, Hand Auger Boring HA-5 was drilled to a depth of about 10 feet, and Scour Samples 1 and 2 were obtained to a depth of 2 feet, at the locations shown on Figures 2a and 2b. 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, ASTM D1587 for Shelby tube sampling, or ASTM D1452 for material taken from the auger as it was advanced 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.

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 procedure, and field observations.

Locations and depths of the borings were specified by Halff in consultation with Arias. The as-drilled boring locations were surveyed by others and are summarized in Table 1.

Table 1: Approximate Boring Locations and Depths

Boring No.	GPS Coordinates		Ground Surface Elevation (feet)	Depth Drilled (feet)	Proposed Structure
	Northing	Easting			
B-1	13698309.61	2153433.05	610.08	30	Monument Sign
B-2	13698843.80	2153658.34	606.53	10	Roadway
B-3	13699062.60	2153674.99	600.19	50	Bridge
B-4	13699256.83	2153570.56	595.92	50	Bridge
B-5	13699363.07	2153300.33	603.10	10	Roadway
B-6	13699754.18	2153066.79	607.09	10	Roadway
B-7	13702397.60	2153572.65	656.33	30	Monument Sign
HA-5	Refer to Boring Location Plan		--	10	Roadway
Scour 1	Refer to Boring Location Plan		--	2	Bridge
Scour 2	Refer to Boring Location Plan		--	2	Bridge

NOTE: Ground surface elevations and location coordinates were provided to Arias by Halff. Boring depths are approximate and were measured relative to the existing ground or pavement surface at the time of the borings.

After completion of drilling, the open boreholes were backfilled using a mixture of soil cuttings and bentonite hole-plug. For borings drilled in the existing pavement areas, the upper 3 feet were backfilled using quick-crete followed by cold patch where the asphalt section is present.

LABORATORY TESTING

As a supplement to the field exploration, laboratory testing was conducted to determine soil moisture content, Atterberg Limits, and percent passing the US Standard No. 200 sieve. In addition, sieve/hydrometer analyses were performed on the two scour samples obtained from Salado Creek. The laboratory results are reported in the boring logs included in Appendix B, and are plotted as Grain Size Distribution Curves in Appendix D. Furthermore, sulfate testing was performed on select samples of the subgrade soils, as detailed in the subsequent section.

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.

Sulfate Testing

Laboratory testing was conducted on five (5) samples from the soil borings located in the proposed pavement areas to evaluate for potential adverse reactions to calcium based treatment agents such as lime, cement, and fly ash. A high sulfate content subgrade will chemically react with calcium based treated pavement layers and will result in excessive heaving of the subgrade soils. It should be noted that the use of lime or cement treatment is not recommended where sulfate contents are greater than 3,000 parts per million (ppm). Accordingly, testing was performed in accordance with TxDOT test method Tex-145-E “Determining Sulfate Content in Soils” in order to evaluate whether it is appropriate to lime or cement treat the pavement subgrade clay soils. The results are presented below in Table 2:

Table 2: Sulfate Test Results

Boring No.	Approx. Sample Depth (feet)	Sulfate Result (ppm)
B-2	1 - 2.5	140
B-3	0 - 1.5	120
B-4	1 - 2.5	160
B-5	0 - 2	140
B-6	1 - 2.5	180

Note:

1. Approximate sample depth is referenced from the existing ground surface at the time of the geotechnical field exploration.

These values shown above are indicative of relatively low soil sulfate contents. Based on the results of the sulfate testing, lime or cement treatment of the subgrade could be considered as a suitable site improvement option. Additional samples of the exposed subgrade should be tested during construction prior to the addition of a calcium based treatment agent such as lime or fly ash. Similarly, any import fill material should also be tested prior to any such treatment.

Lime Series

A lime series was performed in general accordance with ASTM C 977 on a sample of the pavement subgrade soils obtained from Hand Auger Boring HA-5. The test results are plotted on Figure 3 (Page 1 of 2) in Appendix A. The liquid limit (LL) and plasticity index (PI) of the untreated soil was 40 and 27, respectively. Based on these test results, a minimum of 4 percent hydrated lime by weight would be required.

However, the majority of the subgrade soils have LL values of 52 to 59 and PI values of 34 to 36. Accordingly, samples of the dark brown fat clay were obtained in the upper 2 feet near Borings B-3, B-4, and B-5, and the LL ranged from 55 to 58 and PI from 36 to 38. The samples were then combined and a lime series performed. Based on the results presented on Figure 3 (Page 2 of 2), Arias recommends that the pavement subgrade soils be treated with 7 percent lime by weight to achieve a pH of 12.4 as presented in Table 11.

SUBSURFACE CONDITIONS

Existing pavement structure, geology, generalized stratigraphy, and groundwater conditions at the project site are discussed in the following sections based on conditions encountered at the boring locations to the depths explored.

Existing Pavement Structure

Four (4) of the seven (7) soil borings were drilled through the existing pavement. The observed pavement structure at each of those four (4) boring locations is summarized in the table below. The thicknesses of the existing pavement structure will likely vary away from the exploration locations.

Table 3: Existing Pavement Section

Boring No.	Pavement Section, inches		
	Asphalt	Base Material	Total
B-2	2	8	10
B-4	2½	6	8½
B-6	12	--	12
B-7	5	6	11

Geology

A Geologic Map is included as Figure 4 in Appendix A. The earth materials underlying the project site have been regionally mapped as the Fluviatile Terrace Deposits (Qt) overlying ancient marine deposits of the Navarro Formation (Kknm) of the upper Cretaceous Period of the Geologic Time Scale. The Navarro Formation consists mainly of clay, marly clay, marl and shale. The contact between the alluvial and ancient marine deposits represents a significant erosional time gap which could be irregular with depth within the project area.

Site Stratigraphy and Engineering Properties

The general stratigraphic conditions at the boring locations are provided in Table 4 below. *The presence and thickness of the various subsurface materials can be expected to vary away from and between the exploration locations.*

Table 4: Generalized Soil Conditions

Stratum	Depth (feet)	Material Type	PI Range	No. 200 Range	N-value Range	PP Range
Pavement	0 to (0.7 -1.0)	2” to 12” Asphalt over 0” to 8” of Base Material (See Table 3 for Details)	--	--	--	--
Fill	(0.9 -1.0) to (2.5 - 6.0)	Fill: Clayey SAND with Gravel (SC), loose, dark brown, brown and light tan; and/or FAT CLAY (CH), stiff to very stiff, dark brown **Note:** Only noted at B-6 and B-7	25 - 35	43 - 73	8 - 12	*4.0
I	(0 -6) to (2 - 12.5)	FAT CLAY (CH) with varying amounts of sand, stiff to very stiff, dark brown	34 - 41	70 - 89	4 - 18	1.75 - 4.25
II	(2 - 10) to (4 - 22)	LEAN CLAY with varying amounts of gravel (CL), stiff to very hard, dark brown, brown, tan brown, light tan and gray; or CLAYEY GRAVEL with sand, medium dense, brown	14 - 31	22 - 74	7 - 69	4.25 - 12.3
III	(4 - 22) to (10 - 28)	Well-Graded GRAVEL (GW), Poorly-Graded GRAVEL (GP-GM), SILTY GRAVEL (GM), CLAYEY GRAVEL (GC), and/or SILTY CLAYEY SAND (SC-SM), medium dense to very dense, tan and gray, light tan and gray, tan and light brown, and/or brown **Note:** Not encountered at B-5 and B-6	*31	12 - 33	3 - 52	--
IV	(11 - 18) to (18 - 33)	FAT CLAY (CH), firm to hard, tan and gray, tan and dark gray, and gray and tan **Note:** Not encountered at B-2, B-5, B-6, and B-7	*57	--	7 - 30	6.0 - 8.25

Stratum	Depth (feet)	Material Type	PI Range	No. 200 Range	N-value Range	PP Range
V	33 to 50+	SHALEY FAT CLAY (CH), hard to very hard, dark gray **Note:** Encountered only at B-3 and B-4	*53	*100	33 - 54	--

Where: Depth - Depth from existing ground surface during geotechnical study, feet
PI - Plasticity Index, %
No. 200 - Percent passing #200 sieve, %
N - Standard Penetration Test (SPT) value, blows per foot
Uc - Unconfined Compressive strength value, tons per square foot
** - Blow Counts During Seating Penetration
* - Only one test performed
-- - No Test

Groundwater

A dry soil sampling method was used to obtain the soil samples. Groundwater was observed within three (3) of the seven (7) borings during sampling activities. Groundwater observations are noted on the individual borings logs and summarized in Table 5, subsequently.

Table 5: Groundwater Measurements in Borings

Boring No.	Approximate Ground Surface Elevation (feet)	Depth Drilled (feet)	Groundwater Depth (Elevation), feet	
			During Drilling	At Completion and up to 20 Minutes After
B-1	610.1	30	15.0 (595.1)	--
B-3	598.8	50	15.0 (583.8)	14.8 (584.0)
B-4	597.1	50	15.0 (582.1)	4.0 (593.1)

Notes:

1. Depth is measured from existing ground surface at the time of the geotechnical field exploration.
2. Groundwater depth during drilling is where groundwater was first observed. Groundwater level was measured up to 20 minutes after the completion of the boring.

Groundwater levels at the time of construction may differ from the observations obtained during the field exploration because perched groundwater is subject to seasonal conditions, recent rainfall, flooding, drought or temperature affects. Importantly, South Texas, including the area of the project site, has generally experienced drought conditions in recent years, although significant rainfall events have occurred in the San Antonio area since late 2014. Pockets or seams of calcareous deposits, gravel, sand, silt or open fractures and joints can store and transmit "perched" groundwater flow or seepage. "Perched" groundwater flow or seepage may also occur in sand and gravel deposits at the interfaces with clay (fill or native). Should dewatering become necessary during construction, it is considered "means and methods" and is solely the responsibility of the Contractor.

PAVEMENT EVALUATION

Expansive Soil Considerations

We have not been provided with plan and profile sheets for the proposed roadway alignment. Thus, the amounts of planned cut/fill in the new pavement areas are not known to us. Therefore, the pavement considerations and design thicknesses are based on the subgrade materials encountered at grade within the borings performed. Once the plan and profile sheets are available, we should be contacted in order to evaluate the impact on our pavement recommendations and make appropriate adjustments, if required.

The pavement subgrade soils of the existing subgrade are anticipated to consist primarily of dark brown fat clay (fill or native) with high shrink-swell characteristics. Expansive clay materials shrink when they lose water and swell or grow in volume when they gain water content. The potential of expansive soils to shrink and swell is related to the Plasticity Index (PI). Clayey soils with a higher PI have a greater potential for soil volume changes due to moisture content variations. Change in soil moisture is the single most important factor affecting the shrinking and swelling of clayey soils. The most pronounced movements are commonly observed when soils are exposed to extreme moisture fluctuations that occur between drought conditions and wet seasons.

It has been our experience that with these soil types, deep-seated moisture content changes within the expansive clayey subgrade can lead to pavement cracking, undulating pavement and/or the development of potholes. The roadway may be properly designed and constructed with the proper section thickness and materials, but still not perform well due to these expansive soil movements.

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 is assumed to be 15 feet. Using the TxDOT method, we estimated that the PVR at the boring locations (based on existing moisture conditions) varies from approximately **2 to 4 inches**.

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 assumed dry and 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.

We've performed our pavement analyses for this project using the 1993 AASHTO Guide for Design of Pavement Structure. The AASHTO procedure includes provisions to account for roadbed swelling through a reduction in serviceability or ride quality over time as the roadbed swells. Based on the estimated site PVR, we estimate a loss if serviceability of about 0.5 over a 20 year service life due to expansive soil-related movements.

To account for this loss in serviceability, the pavement section can be increased as per the AASHTO procedure. However, it is Arias' opinion that this increase in pavement structure will have little benefit in terms of reducing expansive soil-related pavement distress due to an estimated active zone of about 15 feet. A more effective approach would be to reduce the potential for moisture fluctuations beneath the pavement by providing the following:

- positive site drainage,
- curb and gutter systems,
- moisture barriers as discussed in the subsequent report section, and
- subgrade treatment.

Moisture Fluctuations beneath Pavements

It is common for moisture content values to remain more constant in the middle of the roadway. The moisture levels in the subgrade soils located near the edge of roadways 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 roadway. During extremely wet climate periods, the edges will swell relative to the center of the roadway. The shrinking and swelling of subgrade soils near the edge of pavements will result in longitudinal, surface cracking that occurs parallel to the roadway. Undulating pavement and curbs could also result from these shrink/swell movements. Based on our experience, edge cracking typically occurs at a distance of 3 to 9 feet from the edge of the roadway. Edge cracking associated with soil shrinkage movements may occur at greater distances during extreme environmental conditions. The implementation of moisture barriers, concrete curbs, and positive site drainage can improve the long term performance of the pavement by reducing the impact of the expansive soils.

Based the results of this study, the Owner can consider the option of constructing vertical and/or lateral moisture barriers to help maintain more consistent moisture conditions beneath the pavement, thus reducing the severity of expansive soil-related distress. Even with the implementation of a moisture barrier, the Owner should be prepared to provide pavement maintenance and repair; please refer to the "Performance and Maintenance Considerations" section of this Report for additional information. The Owner may decide to forgo the implementation of a moisture barrier and accept an increased risk for expansive soil-related movement. Potential risks would include costs for maintenance such as patching of cracks and occasional overlays over the life of the pavements.

Some options for moisture barriers to aid in reducing moisture change in the pavement subgrade soils include:

- Vertical Moisture Barriers (VMB). VMBs may consist of polyethylene plastic sheeting placed in an excavated vertical trench that is backfilled with flowable fill. We recommend that a VMB be installed at least 4 feet deep and be located at the pavement edges beneath the curb or directly behind the curb. VMBs should be considered for installation along the length of the project on both sides of the street. Careful coordination will be required by the installation contractor during construction to prevent from damaging existing utilities. It is our opinion that VMBs would be effective in reducing the chances and severity of edge cracking.

- Lateral Moisture Barriers (LMB). LMBs can consist of contiguous sidewalks of sufficient width located directly adjacent to the planned pavements. The use of sidewalks along the length of the project will help provide protection from moisture fluctuations along the pavement edges. It has been our experience that sidewalks acting as an LMB will be most beneficial when located directly adjacent to the concrete curbs. As previously noted based on our experience, edge cracking typically occurs at a distance of 3 to 9 feet from the edge of the roadway. Thus, the wider the sidewalks the more protection will be provided.

Potential landscaping adjacent to the existing roadways 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 these areas. Ponding should not be allowed near the edges of the planned pavements.

Effects of Trees and Vegetation

Soil moisture can be affected by the roots of vegetation that extend beneath pavements. Trees remove large quantities of water from the soil through their root systems during the growing season and cause localized drier areas in the vicinity of the roots. The limits of affected areas are typically related to the lateral extent of a root system, which are a function of the tree height and the spread of its branches. It is generally accepted that a root system will influence the soil moisture levels to a distance roughly equivalent to the drip line (extent of branches). Pavements constructed over a tree root system may shrink due to changes in moisture content and result in cracking. These types of movements result in concentric crack patterns in street pavements located near trees.

If trees will be located next to the roadways, the designers may wish to consider installing localized root barriers as part of the pavement construction in these areas. The root barriers may reduce the potential for future pavement distress due to soil moisture variations from tree roots. Should root barriers be considered, we recommend the designers consult with a tree expert to discuss the effect of barriers on the health of the trees.

PAVEMENT RECOMMENDATIONS

We have been informed that the proposed roadway reconstruction will be classified as a City of San Antonio (CoSA) “Local Type A with bus traffic.” If a different street classification is to be utilized, then we should be contacted to provide additional recommendations.

Design Parameters and Traffic Conditions

Based on the results of our fieldwork and laboratory testing and once all of the existing asphalt and base materials are removed, it appears likely that the roadway subgrade will consist of a relatively fat clay (CH) material. We recommend a subgrade CBR value of 2.5 be utilized for the pavement design. For localized sections of the roadways which may require an increase in the existing grade, it is assumed that the subgrade will be general fill consisting of on-site soils. We recommend that general fill used to increase sections of the roadway grade have a CBR value greater than 2.5. The suitability of all fill materials should be approved by the Geotechnical Engineer.

As previously mentioned, we have not received plan and profile sheets for this project. Thus, the amounts of planned cut/fill in the new pavement areas are not known to us. It should be noted that the conditions and recommendations contained herein are based on the materials encountered at the time of field exploration. These conditions may differ once the road grading (cut/fill) operations are performed. Once the plan and profile sheets are available, we should be contacted in order to evaluate the impact on our pavement recommendations and make appropriate adjustments, if required. We recommend that a representative of Arias be retained to observe that our recommendations are followed and to assist in determining the actual subgrade material classification at a particular location.

Recommendations in this section were prepared in accordance with the 1993 AASHTO Guide for Design of Pavement Structure and the CIMS Design Guidance Manual (DGM), Appendix 10-A "City of San Antonio Pavement Design Standards". Structural material coefficients are provided in Table 6 below, and design parameters utilized in our pavement evaluation are presented subsequently in Table 7.

Table 6: Material Coefficients

Material	Structural Coefficient
Hot Mix Asphaltic Concrete – Type “D” or “C” Surface Course	0.44
Hot Mix Asphaltic Concrete – Type “B” Base Course	0.38
Flexible Base Course – TxDOT Item 247, Type A, Grades 1 or 2	0.14
Lime Treated Subgrade	0.08
Moisture Conditioned Subgrade	--

Table 7: Parameters for Park Road (Local Type A with bus traffic)

Design Parameters	Flexible Pavement	Rigid Pavement
Reliability Factor	70%	70%
Overall Standard Deviation	0.45	0.35
Initial Serviceability Index	4.2	4.5
Terminal Serviceability Index	2.0	2.0
18-kip Equivalent Axle Loads (ESALs)	1,000,000	1,500,000
Service Design Period	20 years	30 years
28-day Concrete Modulus of Rupture	N/a	600 psi @ 28 days
28-day Concrete Elastic Modulus	N/a	4,000,000 psi
Load Transfer Coefficient	N/a	3.2
Drainage Coefficient	N/a	1.03
Minimum Required Structural Number or Pavement Section	SN=2.58	6”
Maximum Required Structural Number or Pavement Section	SN=4.20	8”

We have reviewed the existing roadway conditions and applied the anticipated traffic levels in accordance with the methods outlined in the 1993 AASHTO Guide for the Design of Pavement Structures and the ACI Design Guide SCM-28 (95).

Due to the low anticipated design speeds, Arias considers it appropriate to consider both Jointed Concrete Pavements (JCP) and Continuously Reinforced Concrete Pavements (CRCP) for use in the site pavements.

Arias recommends that the dowel sizes and embedment depths for the transverse contraction joints and the longitudinal construction joints for JRCP be designed in accordance with the TxDOT concrete pavement standards presented on CPCD-94. We recommend the use of the TxDOT detail: CPCD-94, Concrete Pavement Details, Contraction Design (CPCD), except that the pavement should also include distributed reinforcing steel (No. 4 rebar @ 18-inch spacing each way, placed D/3 form the top of the slab) to account for the expansive clay soils. The distributed steel should not be continued through the pavement joints to allow the joints to function properly. The JRCP pavements are anticipated to require more maintenance related to the joints than CRCP pavements.

We recommend that the longitudinal and transverse steel for use in CRCP be sized by the designers to meet the minimum requirements presented on the TxDOT design standards presented on CRCP-11. We recommend the use of the TxDOT detail: CRCP (1)-03, Continuously Reinforced Concrete Pavement, One-Layer Steel Bar Placement.

We have assumed a modulus of rupture of 600 psi and elastic modulus of 4,000,000 psi at 28 days for concrete. We have used a load transfer coefficient of 3.2 based on the use of dowels for transfer across transverse joints, and that shoulders will not be provided.

Flexible Pavement Recommendations

Based on the parameters provided in the previous tables, a subgrade design CBR of 2.5 and the CoSA CIMS DGM, a structural number (SN) of 3.75 was attained for flexible pavement for the park road (Local Type A with bus traffic). This number is between the minimum and maximum SN values of 2.58 and 4.20, respectively, recommended by the CoSA CIMS DGM. Thus, our proposed design sections provided in this report were based on a design SN of 3.75.

The following pavement thickness options may be considered in order to meet the design requirements for a Local Type A with bus traffic. Many other choices or alternatives are possible

It should be noted that subsurface conditions should be expected to vary along the alignment and that planned cut/fill could have a significant impact on the type of material ultimately delineated as subgrade at a particular location. We recommend that site preparation procedures be observed and tested by a representative of Arias.

Table 8: Flexible Pavement Options for Park Road (Local Type A with Bus Traffic)

Subgrade Classification	**FAT CLAY (CH) Subgrade**		
Subgrade Design CBR	CBR = 2.5		
Required Structural No.	3.75		
Recommended Subgrade Treatment	Hydrated Lime		
Pavement Section Options			
	Option 1	**Option 2**	**Option 3**
Type “D” or “C” HMAC Surface Course	3”	2”	2”
Type “B” HMAC Base Course	--	6.5”	4”
Type “A” Flex Base Course (Crushed Limestone)	14”	--	7”
Lime Treated Subgrade	6”	6”	6”
Total Pavement Section	23”	14.5”	15”
Calculated Structural No.	**3.76**	**3.83**	**3.86**

If other subgrade conditions are encountered, then we should be contacted to provide additional recommendations. A representative of Arias should be retained during construction in order to observe and test the various pavement materials and layers.

Rigid Pavement Recommendations for Local Type "A" Streets with Bus Traffic

Based on the parameters provided in the previous tables a modulus of subgrade reaction (k) of 150 pci for a lime treated subgrade, a pavement thickness of 7 inches was attained for rigid pavement (Local Type "A" Streets with Bus Traffic). This number is greater than the minimum required structural pavement section thickness of 6 inches and lower than the maximum required structural pavement thickness of 8 inches as noted in the CIMS DGM. Therefore, the use of **7 inches** of concrete is recommended for the rigid pavement section as presented subsequently in Table 9.

Table 9: Rigid Pavement Option for Park Road (Local Type A with Bus Traffic)

Subgrade Classification	High Plasticity CLAY (CH) Subgrade
Recommended Subgrade Treatment	Hydrated Lime
Pavement Section Options	
Portland Cement Concrete	7"
Lime Treated Subgrade	6"

Notes:

1. Pavements founded on top of site soils will be subjected to PVR soil movements estimated and presented in this report (i.e., about 2 to 4 inches).
2. During the paving life, maintenance to seal surface cracks and to reseal joints within concrete pavement should be undertaken to achieve the desired paving life. Perimeter drainage should be controlled to reduce the influx of surface water from areas surrounding the paving. Water penetration into subgrade materials, sometimes due to irrigation or surface water infiltration leads to pre-mature paving degradation. Curbs should be used in conjunction with paving to reduce potential for infiltration of moisture into the subgrade.

Arias recommends the use of reinforcing steel in the concrete pavement. Both Jointed Concrete Pavements (JCP) and Continuously Reinforced Concrete Pavements (CRCP) can be considered. CRCP will have a higher level of performance and will require less maintenance than JCP.

For CRCP, we recommend that the longitudinal and transverse steel be sized by the designers to meet the minimum requirements presented on the TxDOT design standards presented on CRCP-13. We recommend the use of the TxDOT detail: CRCP (1)-13, Continuously Reinforced Concrete Pavement, One-Layer Steel Bar Placement.

For JCP, we recommend that the dowel sizes and embedment depths for the transverse contraction joints and the longitudinal contraction and construction joints be designed in accordance with the TxDOT concrete pavement standards presented on CPCD-14. We recommend the use of the TxDOT detail: CPCD-14, Concrete Pavement Details, Contraction Design (CPCD). We recommend the pavements also include distributed reinforcing steel (At least No. 4 rebar @ 18- inch spacing each way, placed D/3 form the top of the slab) to account for the expansive clay soils. The distributed steel should not be continued through the pavement joints to allow the joints to function properly.

Rigid Concrete Pavement Joints

Placement of expansion joints in concrete paving on potentially expansive subgrade 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 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.

Site Drainage

The favorable performance of any pavement structure is dependent on positive site drainage. This is particularly important at this site due to the expansive soils encountered in the borings. Careful consideration should be provided by the designers to assure positive drainage of all storm waters away from the planned pavements. Ponding should not be allowed either on or along the edges of the pavements.

Performance and Maintenance Considerations

Our pavement recommendations have been developed to provide an adequate structural thickness to support the anticipated traffic volumes. Shrink/swell movements due to moisture variations in the underlying soils should be anticipated over the life of the pavements. The owner should recognize that over a period of time, pavements may crack and undergo some deterioration and loss of serviceability. Deterioration can occur more rapidly as a result of climatic extremes such as drought conditions, or periods that are wetter than normal. We recommend the project budgets include an allowance for maintenance such as patching of cracks, repairing potholes and other distressed areas, or occasional overlays over the life of the pavement.

It has been our experience that pavement cracking will provide a path for surface runoff to infiltrate through the pavements and into the subgrade. Once moisture is allowed into the subgrade, the potential for pavement failures and potholes will increase. We recommend the owners implement a routine maintenance program with regular site inspections to monitor the performance of the site pavements. Cracking, which may occur on the asphalt surface due to shrink/swell movements, should be sealed immediately using a modified polymer hot-applied asphalt based sealant. Additional crack sealing will likely be required over the design life of the pavements. Crack sealing is a proven, routine, maintenance practice to help preserve pavements and help reduce pavement wear and deterioration. Failure to provide routine crack-sealing will increase the potential for pavement failures and potholes to develop.

PAVEMENT CONSTRUCTION CRITERIA

Pavement Subgrade and Section Materials

Recommendations for subgrade preparation in the planned pavement areas, as well as for the pavement section materials, are provided in Table 10 thru 13 provided subsequently.

Table 10: Subgrade Preparation and Fill Requirements

Subgrade Preparation Prior to Pavement Section Construction	
Minimum undercut depth	6 inches or as needed to remove organics and existing pavement, deleterious materials, rubble, soft and/or yielding areas, and dispose of offsite.
Reuse excavated soils	Provided they are free of roots and debris and meet the material requirements for their intended use
Horizontal extent for pavement removal	2 feet beyond the paving limits
Exposed subgrade (before lime-treatment)	Proof roll with rubber tired vehicle weighing at least 20 tons such as a loaded dump truck with Geotechnical Engineer's representative present during proof rolling. Moisture condition at least the top 6-inches to 0% to +4% of optimum moisture at ≥ 95% maximum dry density as obtained by Standard Proctor ASTM D698.
Pumping/rutting areas discovered during proof rolling	Remove to firmer materials and replace with compacted general or select fill under direction of Geotechnical Engineer's representative
Fill Requirements for Grade Increases	

General fill type	Material free of roots, debris and other deleterious material with a maximum rock size of 3 inches; on-site clays having CBR ≥ 2.5 may be used. Imported fill materials used under pavements should have a CBR value of at least 2.5.
Fill placement procedure	CoSA Standard Specifications for Construction, Item 107, “Embankment”
Minimum general fill thickness	As required to achieve grade
Maximum general fill loose lift thickness	8 inches
General fill compaction and moisture criteria	ASTM D 698 ≥ 95% compaction at 0 to +4 from optimum moisture

Table 11: Subgrade Treatment

Subgrade Treatment Option – Lime Treatment	
Treatment depth	6 inches
Treatment type	Hydrated lime
Application rate (estimated)	7% by dry weight, which corresponds to 33 lbs per square yard of subgrade for a 6-inch compacted thickness
Soil dry unit weight (estimated)	105 pcf but may be variable
Determination of application rate	The actual application rate should be determined by laboratory testing of soil samples taken after the pavement subgrade elevation has been achieved. The quantity of lime should be sufficient to result in a pH of at least 12.4 when tested in accordance with ASTM C 977, Appendix XI. Alternately, the optimum lime content may be determined through Atterberg limits testing on treated samples with varying percentages of lime. Additional sulfate testing of the exposed subgrade should be performed prior to the use of lime, cement or other calcium-based treatment agents.
Treatment procedure	CoSA Standard Specifications for Construction, Item 108, “Lime Treated Subgrade”
Treatment layer compaction and moisture criteria	TEX-114-E ≥ 95% compaction at 0 to +4 from optimum

Table 12: Pavement Section Materials

Flexible Pavement Section Materials	
Flexible Base Material Type	CoSA Standard Specifications for Construction, Item 200, "Flexible Base", Type A, Grade 1 or 2
Flexible Base compaction and moisture criteria	TEX-113-E ≥ 95% compaction at -3 to +3 from optimum
Maximum Flexible Base Loose Lift Thickness	9 inches
Hot Mix Asphaltic Concrete (HMAC) Type	CoSA Standard Specifications for Construction, Item 205, "Hot Mix Asphaltic Concrete Pavement" Base Course: Type B Surface Course: Type C or D
Portland Cement Concrete (PCC)	
Minimum compressive strength at 28 days	CoSA Item 209
Desired slump during placement	5 ± 1 inch
Expansion Joints	May be eliminated except at tie-ins with existing concrete and structures
Placement	In accordance with ACI 304R (guide for measuring, mixing, transporting, and placing), ACI 305R (hot weather concreting, and ACI 306R (cold weather concreting)
In-Place Density and Moisture Verification Testing	
Testing frequency **(Subgrade, Flexible Base, Asphaltic Base, Asphalt Course(s))**	Every 150 Linear Feet for each Lift

Curb and Gutter

It has been our experience that pavements typically perform at a higher level when designed with adequate drainage including the implementation of curb and gutter systems. Accordingly, we recommend that curb and gutters be considered for this project. Furthermore, to aid in reducing the chances for water to infiltrate into the pavement base course and pond on top of the pavement subgrade, we highly recommend that pavement curbs be designed to extend through the pavement base course penetrating at least 3 inches into the onsite clay subgrade. If water is allowed to infiltrate beneath the site pavements, frequent and premature pavement distress can occur.

Construction Site Drainage

We recommend that areas along the roadways be properly maintained to allow for positive drainage as construction proceeds and to keep water from ponding adjacent to the site pavements. This consideration should be included in the project specifications.

BRIDGE FOUNDATION DESIGN RECOMMENDATIONS

We understand that a new bridge is proposed to replace the existing low water crossing at Salado Creek. The proposed bridge is anticipated to have 10 spans of 25 feet for a total span length of 225 feet. The bridge is proposed to be 46 feet in width that will include a 2-lane roadway and a pedestrian walkway. The bridge abutments are proposed to be supported on five 24-inch diameter straight-shaft drilled piers each with a service load of 30 tons (60 kips). The bents are proposed to be supported on three 24-inch diameter straight-shaft drilled piers each with a service load of 95 tons (190 kips).

Straight-Shaft Drilled Piers

The piers, when properly founded, can help reduce foundation movement of the superstructure. The drilled piers will need to be designed for potential lateral loading from structural loads including wind loading and potential loading from rising floodwaters. The piers would also need to be based adequately below the zone of seasonal moisture change to resist expansive soil-related movement, as well as be below the depth of potential scour.

More specifically, the final pier dimensions, particularly to include the required length of piers, will be determined based on:

- foundation design loads,
- depth of the active zone,
- depth required to establish an effective casing seal in low permeability clay beneath water-bearing granular soils
- potential uplift force imposed by expansive soils within the active zone,
- depth of scour, which must be considered for axial and lateral capacity,
- potential lateral and uplift loading from floodwaters, and
- available side friction and end-bearing capacity of the subsurface soils.

The active zone is the depth influenced by seasonal moisture variations. As previously noted, this depth is estimated at approximately 15 feet at this project site. *It is important to note that hard to very hard shaley fat clay was encountered at Borings B-3 and B-4 drilled in the area of the planned bridge. Thus, high-torque drilling equipment will likely be required in hard to very hard materials. The Contractor should be prepared for such conditions.* We understand that the bottom of Abutment Nos. 1 and 10 are estimated at about Elevation (El.) 599 feet and El. 597 feet, respectively. Furthermore, the existing ground surface at Bent Nos. 2 through 9 ranges from about El. 598 to 588. If these elevations change, we should be contacted to determine whether our recommendations should be revised.

Pier capacities for axial loading were evaluated based on design methodologies included in FHWA-NHI-10-016 - 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 as presented subsequently in Tables 13 and 14 based on Borings B-3 and B-4, respectively, which were drilled in the vicinity of the proposed bridge.

Table 13: Drilled Pier Axial Design Parameters at Bridge (Boring B-3)

<table>
<tr><th rowspan="2">Elevation (ft)</th><th rowspan="2">Material</th><th colspan="3">Recommended Design Parameters</th></tr>
<tr><th>Allowable Skin Friction, psf ($\alpha c/FS$)</th><th>Allowable End Bearing, psf (cN_c/FS)</th><th>Uplift Force, kips</th></tr>
<tr><td>599 - 594</td><td>Stiff Dark Brown FAT CLAY over Very Stiff LEAN CLAY</td><td colspan="2">Neglect Contribution</td><td rowspan="6">50•D</td></tr>
<tr><td>594 - 592</td><td>Very Stiff Brown LEAN CLAY</td><td>275</td><td>--</td></tr>
<tr><td>592 - 582</td><td>Dense to Medium Dense Tan and Gray SILTY GRAVEL with Sand over CLAYEY GRAVEL</td><td>275</td><td>--</td></tr>
<tr><td>582 - 577</td><td>Stiff Tan and Gray FAT CLAY</td><td>400</td><td>--</td></tr>
<tr><td>577 - 567</td><td>Very Stiff to Hard Tan, Gray, and Dark Gray FAT CLAY</td><td>800</td><td>--</td></tr>
<tr><td>567 - 550</td><td>Hard to Very Hard Dark Gray SHALEY FAT CLAY</td><td>1,200</td><td>15,000</td></tr>
<tr><th colspan="5">Constraints to be Imposed During Pier Design</th></tr>
<tr><td colspan="2">Allowable Skin Friction - Compression</td><td colspan="3">• <u>Abutment Piers</u>: use the allowable skin friction along the pier shaft to 1 pier diameter above the bottom of the shaft.
• <u>Bent Piers</u>: use the allowable skin friction from the scour elevation to 1 pier diameter above the bottom of the shaft.</td></tr>
<tr><td colspan="2">Allowable Skin Friction - Uplift</td><td colspan="3">• <u>Abutment Piers</u>: use the allowable skin friction along the pier shaft below Elevation 592 feet.
• <u>Bent Piers:</u> use the allowable skin friction from 10 feet below the scour elevation to the bottom of the shaft.</td></tr>
<tr><td colspan="2">Minimum Embedment into Bearing Material</td><td colspan="3">• Elevation 563 feet and also 4 feet into the Stratum IV – Hard to Very Hard Dark Gray SHALEY FAT CLAY, whichever results in the deeper pier tip.</td></tr>
<tr><td colspan="2">Minimum Spacing Center to Center</td><td colspan="3">3 pier diameters</td></tr>
<tr><td colspan="2">Minimum shaft diameter</td><td colspan="3">24 inches</td></tr>
</table>

Notes:

1. The piers must bear sufficiently into the Stratum IV – Hard to Very Hard Dark Gray SHALEY FAT CLAY to meet the requirements given above. The recommended design parameters include a factor of safety of 2 for skin friction and of 3 for end bearing.
2. 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-shaft piers, the uplift capacity is computed using the allowable skin friction as indicated above. Sustained dead loads will also aid in resisting uplift forces.

3. Pier vertical reinforcing steel should be designed to resist the uplift forces from swelling soils, and uplift and lateral forces from wind and possible floodwater loading. 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.
4. Total and differential settlement of piers is expected to be less than 1 inch and ½ inch, respectively. Estimated settlements are based on performance of properly installed piers in the San Antonio area. A detailed settlement estimate is outside of the scope of this service.
5. If the piers are subject to water action, scour may occur. If this is the case, 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.

Table 14: Drilled Pier Axial Design Parameters at Bridge (Boring B-4)

<table>
<tr><th rowspan="2">Elevation (ft)</th><th rowspan="2">Material</th><th colspan="3">Recommended Design Parameters</th></tr>
<tr><th>Allowable Skin Friction, psf ($\alpha c/FS$)</th><th>Allowable End Bearing, psf (cN_c/FS)</th><th>Uplift Force, kips</th></tr>
<tr><td>597 - 592</td><td>Fill over Soft to Stiff Dark Brown, Gray Brown, and/or Gray FAT CLAY</td><td colspan="2">Neglect Contribution</td><td rowspan="6">50•D</td></tr>
<tr><td>592 - 583</td><td>Stiff Gray and Brown FAT CLAY</td><td>275</td><td>--</td></tr>
<tr><td>583 - 578</td><td>Very Loose Gray SILTY, CLAYEY SAND</td><td>200</td><td>--</td></tr>
<tr><td>578 - 568</td><td>Stiff Tan and Gray FAT CLAY</td><td>400</td><td>--</td></tr>
<tr><td>568 - 558</td><td>Very Stiff Gray and Tan FAT CLAY over Very Stiff Dark Gray SHALEY FAT CLAY</td><td>800</td><td>--</td></tr>
<tr><td>558 - 546</td><td>Hard Dark Gray SHALEY FAT CLAY</td><td>1,200</td><td>15,000</td></tr>
<tr><th colspan="5">Constraints to be Imposed During Pier Design</th></tr>
<tr><td colspan="2">Allowable Skin Friction - Compression</td><td colspan="3">• <u>Abutment Piers</u>: use the allowable skin friction along the pier shaft to 1 pier diameter above the bottom of the shaft.
• <u>Bent Piers</u>: use the allowable skin friction from the scour elevation to 1 pier diameter above the bottom of the shaft.</td></tr>
<tr><td colspan="2">Allowable Skin Friction - Uplift</td><td colspan="3">• <u>Abutment Piers</u>: use the allowable skin friction along the pier shaft below Elevation 592 feet.
• <u>Bent Piers:</u> use the allowable skin friction from 10 feet below the scour elevation to the bottom of the shaft.</td></tr>
</table>

Elevation (ft)	Material	Recommended Design Parameters		
		Allowable Skin Friction, psf ($\alpha c/FS$)	**Allowable End Bearing, psf (cN_c/FS)**	**Uplift Force, kips**
Minimum Embedment into Bearing Material		• Elevation 554 feet and also 4 feet into the Stratum IV – Hard Dark Gray SHALEY FAT CLAY, whichever results in the deeper pier tip.		
Minimum Spacing Center to Center		3 pier diameters		
Minimum shaft diameter		24 inches		
Allowable Skin Friction - Compression		• **Abutment Piers:** use the allowable skin friction along the pier shaft to 1 pier diameter above the bottom of the shaft. • **Bent Piers:** use the allowable skin friction from the scour elevation to 1 pier diameter above the bottom of the shaft.		

Notes:

1. The piers must bear sufficiently into the Stratum IV – Hard to Very Hard Dark Gray SHALEY FAT CLAY to meet the requirements given above. The recommended design parameters include a factor of safety of 2 for skin friction and of 3 for end bearing.
2. 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-shaft piers, the uplift capacity is computed using the allowable skin friction as indicated above. Sustained dead loads will also aid in resisting uplift forces.
3. Pier vertical reinforcing steel should be designed to resist the uplift forces from swelling soils, and uplift and lateral forces from wind and possible floodwater loading. 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.
4. Total and differential settlement of piers is expected to be less than 1 inch and ½ inch, respectively. Estimated settlements are based on performance of properly installed piers in the San Antonio area. A detailed settlement estimate is outside of the scope of this service.
5. If the piers are subject to water action, scour may occur. If this is the case, 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.

Lateral Pile Capacity

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 Tables 15 and 16 given subsequently, Arias presents geotechnical input parameters for the encountered soils in Bridge Borings B-3 and B-4, respectively. The elevations to the top and bottom of each layer were interpreted using data developed at the explored boring locations and layer boundaries as shown on the logs for the respective borings.

Furthermore, as noted previously for axial pier capacity, consideration should be given to possible loss of lateral support if scour occurs around the upper portion of the bridge piers. It was beyond the scope of this study to evaluate scour potential.

Table 15: Geotechnical Input Parameters for Lpile Analyses at Bridge (Boring B-3)

Elevation (ft)	Material	LPILE Soil Type	γ_e	C_u	ϕ	K Static	e_{50}
599 - 594	Stiff Dark Brown FAT CLAY over Very Stiff LEAN CLAY	3	0.069	6.94	0	300	0.009
594 - 592	Very Stiff Brown LEAN CLAY	3	0.072	10.41	0	500	0.007
592 - 582	Dense to Medium Dense Tan and Gray SILTY GRAVEL with Sand over CLAYEY GRAVEL	4	0.036	0	30	60	--
582 - 577	Stiff Tan and Gray FAT CLAY	3	0.036	11.11	0	500	0.007
577 - 567	Very Stiff to Hard Tan, Gray, and Dark Gray FAT CLAY	3	0.036	20.83	0	1,000	0.005
567 - 550	Hard to Very Hard Dark Gray SHALEY FAT CLAY	3	0.039	34.72	0	1,500	0.004

Where:

γ_e = effective soil unit weight, pci
c_u = undrained soil shear strength, psi
ϕ = undrained angle of internal friction, degrees
K = modulus of subgrade reaction, pci
e_{50} = 50% strain value

Table 16: Geotechnical Input Parameters for Lpile Analyses at Bridge (Boring B-4)

Elevation (ft)	Material	LPILE Soil Type	γ_e	C_u	ϕ	K Static	e_{50}
597 - 592	Fill over Soft to Stiff Dark Brown, Gray Brown, and/or Gray FAT CLAY	3	0.069	5.56	0	300	0.010
592 - 583	Stiff Gray and Brown FAT CLAY	3	0.033	8.33	0	400	0.008
583 - 578	Very Loose Gray SILTY, CLAYEY SAND	4	0.028	0	24	20	--
578 - 568	Stiff Tan and Gray FAT CLAY	3	0.036	11.11	0	500	0.007
568 - 558	Very Stiff Gray and Tan FAT CLAY over Very Stiff Dark Gray SHALEY FAT CLAY	3	0.036	20.83	0	1,000	0.005
558 - 546	Hard Dark Gray SHALEY FAT CLAY	3	0.039	34.72	0	1,500	0.004

Where:

γ_e = effective soil unit weight, pci
c_u = undrained soil shear strength, psi
ϕ = undrained angle of internal friction, degrees
K = modulus of subgrade reaction, pci
e_{50} = 50% strain value

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 locations of Borings B-3 and B-4. 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 17: Drilled Pier Installation Considerations

Recommended installation procedure	USACE refers to FHWA (FHWA-NHI-10-016, May 2010)
High-torque drilling equipment anticipated	Yes, where hard to very hard clay and shaley clay are encountered.
Groundwater anticipated	Yes
Temporary casing anticipated	Yes; extent depends upon subsurface soil and groundwater conditions encountered during construction. Casing anticipated to be extended through Stratum III water-bearing gravel and sand soils and sufficiently into the underlying relatively impervious stiff to very stiff fat clay to achieve an adequate seal.
Slurry installation anticipated	Yes, if casing seal cannot be achieved in the relatively impervious fat clay as noted above.
Concrete placement	Same day as drilling. If a pier excavation cannot be drilled and filled with concrete on the same day, temporary casing or slurry may be needed to maintain an open excavation.
Maximum water accumulation in excavation	2 inches
Concrete installation method needed if water accumulates	Tremie or pump to displace water
Quality assurance monitoring	Geotechnical engineer's representative should be present during drilling of all piers, should observe drilling and document the installed depth, should confirm and document the bearing material type at the base of excavation and cleanliness of base, and should observe placement of steel rebars.

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

- 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 pier 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.

DESIGN RECOMMENDATIONS AND CONSTRUCTION CRITERIA FOR MONUMENTS

We understand that the seating area and a portion of the adjacent sidewalk at the northeast corner of the entrance to MLK Park from Martin Luther King (MLK) Drive will be demolished. A new structure identified as Monument Plaza "A" will be constructed at this location. At the northwest corner of the entrance, the existing small sign will be removed, and will be replaced with Monument Wall "B."

Engineering Evaluation and Recommendations

Boring B-1 was drilled in the area of the proposed Monument Plaza "A." The soil conditions consisted of highly expansive fat clay to a depth of about 4 feet underlain by moderately expansive lean clay to about 6 feet. The lean clay was underlain by gravel deposits with a low shrink/swell potential to a depth of about 11 feet, and then highly expansive clay was encountered to the explored depth of about 30 feet.

We estimate the **PVR is about 3 inches** at the proposed locations of Monuments "A" and "B." Accordingly, we recommend that the dark brown expansive clay be removed down to the tan brown lean clay (i.e. removal of about 4 feet). The removal of the unsuitable fat clay should extend at least 3 feet beyond the perimeter of planned structures. The resulting subgrade should be proof rolled, unstable areas replaced, and then the resulting subgrade should be prepared, and select fill placed and compacted as outlined subsequently in this report. The improved building pad is estimated to have a **PVR of about 1 inch**.

Foundations for proposed structures should be based a minimum of 2 feet below adjacent finished grade, and can be designed for an allowable bearing pressure of 2,000 pounds per square foot. The ground surface should be sloped away from structures to provide suitable surface water drainage.

Site Preparation and Select Fill Requirements

In the area of development for proposed Monuments “A” and “B” at the MLK Drive entrance, topsoil stripping should be performed as needed to remove organic materials, “mucky” soils, vegetation, and roots in the area of the proposed construction. Furthermore, removal should include any debris, trash, landfill materials, etc. and be properly disposed of offsite.

A loaded dump truck weighing at least 20 tons should be utilized to proof-roll over the resulting subgrade areas for the proposed construction. A representative of the Geotechnical Engineer should be present to observe proof-rolling operations. As per the representative of the Geotechnical Engineer, areas of deflection should be removed, re-compacted and/or replaced with similar material.

The resulting subgrade following proof-rolling should then be scarified to a depth of at least 12 inches, moisture conditioned to between optimum and plus four (+4) percentage points of optimum moisture content, and compacted to at least 95 percent of the maximum density determined using ASTM D 698. We recommend that one of our representatives be scheduled to observe that the site preparation operations are performed in accordance with our recommendations.

Select Fill should consist of Crushed Limestone Base Material that meets the following requirements:

- TxDOT Item 247, Type A, Grade 1, 2, or 5 Base Material
- placed in maximum 9-inch loose lifts;
- moisture conditioned from -2 to +3 percent of optimum moisture; and
- compacted to at least 95% of the maximum dry density (ASTM D 698).

Conformance testing during construction to assure quality control will be necessary for this process. The suitability of all fill materials should be approved by the Geotechnical Engineer.

Earthwork and Foundation Acceptance

Exposure to the environment may weaken the soils within the foundation bearing levels if the excavations remain open for long periods of time. Therefore, it is recommended that all foundation excavations be extended to final grade and backfilled as soon as possible to reduce potential damage to the bearing soils. If the bearing soils are exposed to severe drying or wetting, the unsuitable soils must be re-conditioned or removed as appropriate, prior to completing foundations. The foundation bearing level should be free of loose soil, ponded water or debris and should be observed prior to placing fills by the Geotechnical Engineer or his representative.

Foundation leveling pads should not be placed on soils that have been disturbed by rainfall or seepage. If the bearing soils are softened by surface water intrusion during exposure or by desiccation, the unsuitable soils must be removed from the foundation excavation and replaced with compacted select fill or flowable fill prior to placement of concrete.

GENERAL COMMENTS

This report was prepared as an instrument of service for this project exclusively for the use of Halff and the project design team. If the development plans change relative to layout, anticipated traffic 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 E.

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 F.

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 recompacted 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

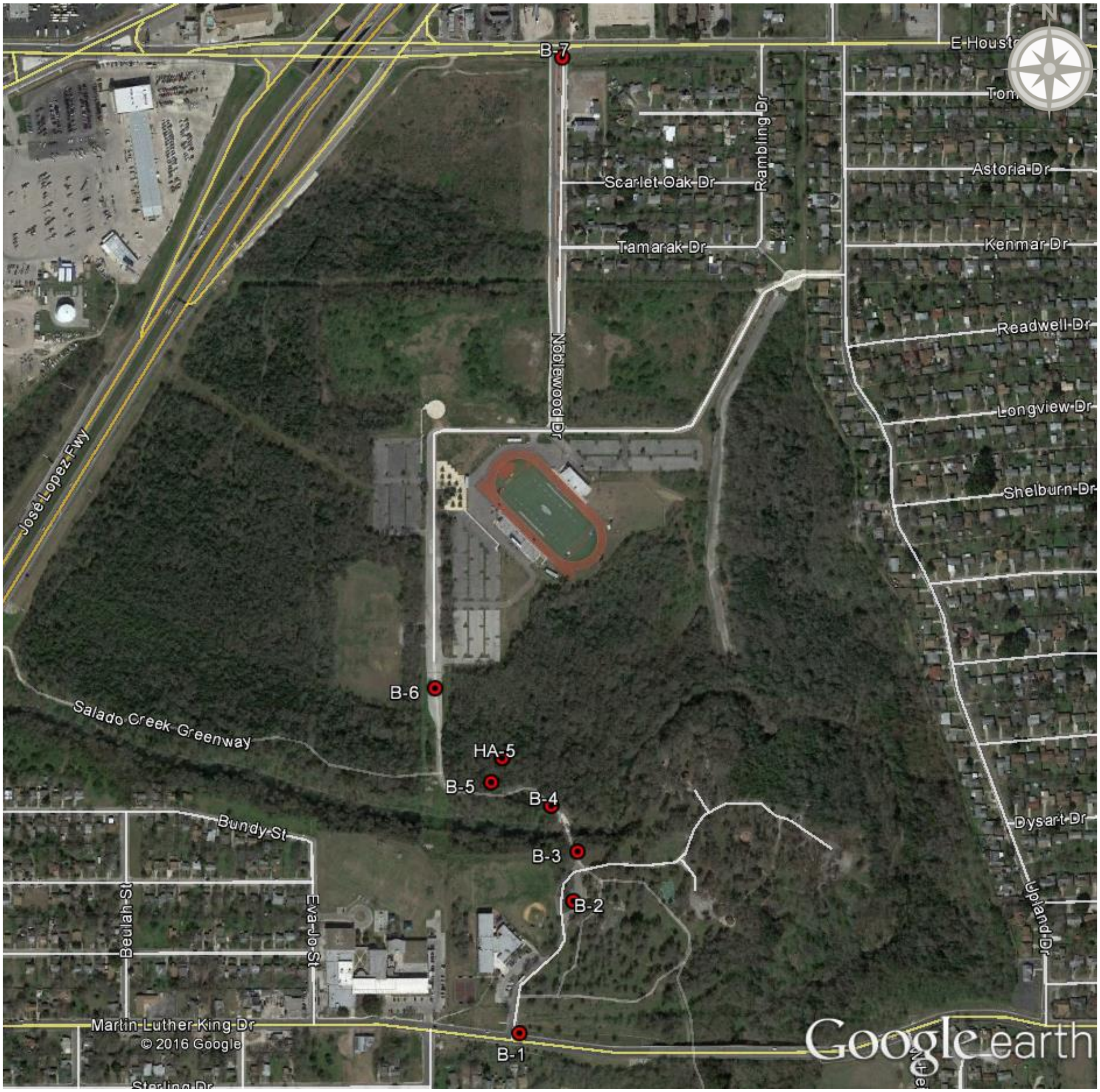
ARIAS GEOPROFESSIONALS
142 Chula Vista, San Antonio, Texas 78232
Phone: (210) 308-5884 • Fax: (210) 308-5886

VICINITY MAP

Martin Luther King Park Improvements
San Antonio, Texas

Date: May 19, 2016	Job No.: 2015-1018
Drawn By: TAS	Checked By: TJF
Approved By: SAH	Scale: N.T.S.

Figure 1



ARIAS GEOPROFESSIONALS

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

OVERALL LOCATION PLAN

Martin Luther King Park Improvements
San Antonio, Texas

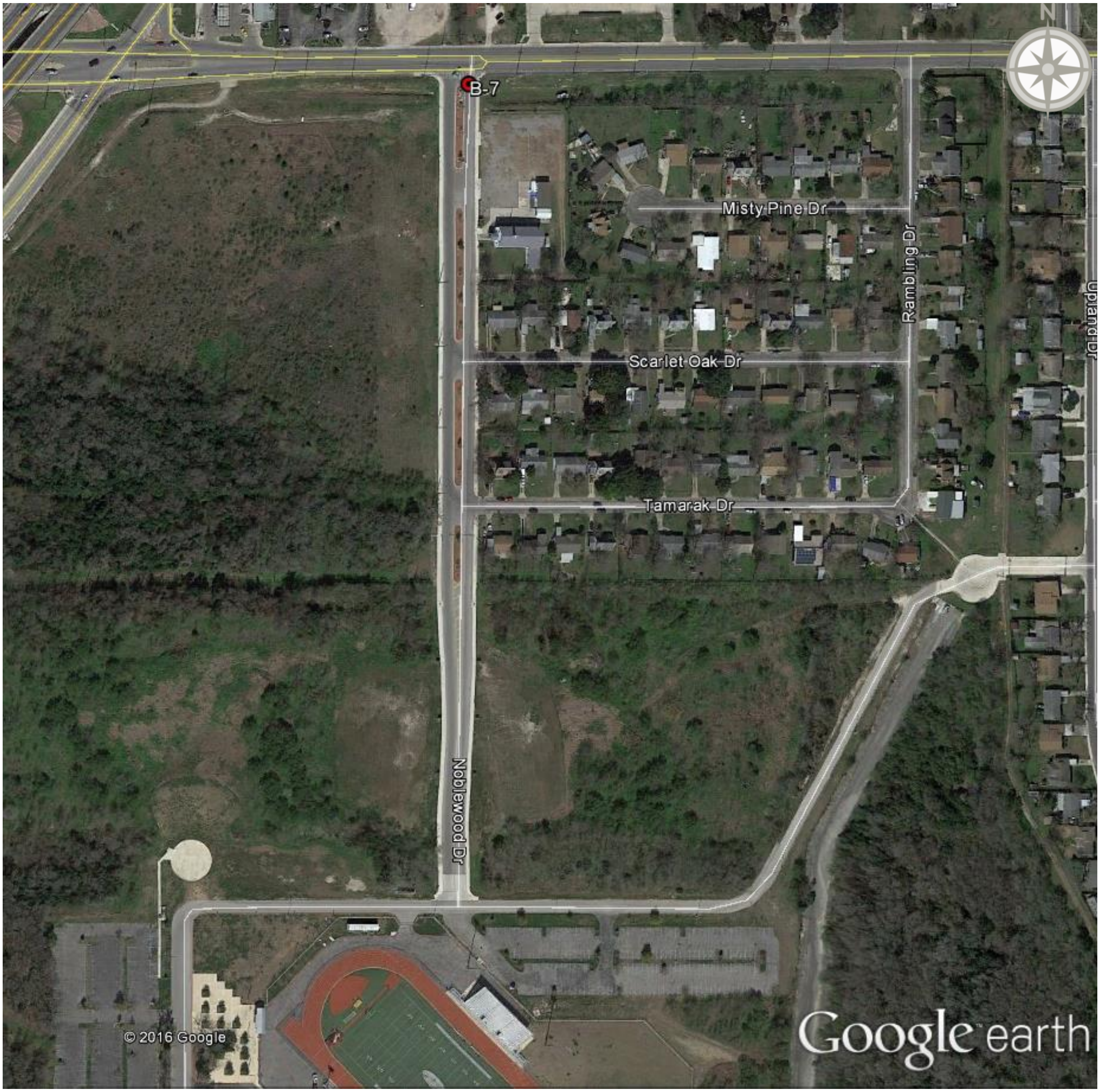
Date: June 24, 2016	Job No.: 2015-1018
Drawn By: TAS	Checked By: TJF
Approved By: SAH	Scale: N.T.S.

REVISIONS:

No.:	Date:	Description:

Figure 2a

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



ARIAS GEOPROFESSIONALS

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

BORING LOCATION PLAN

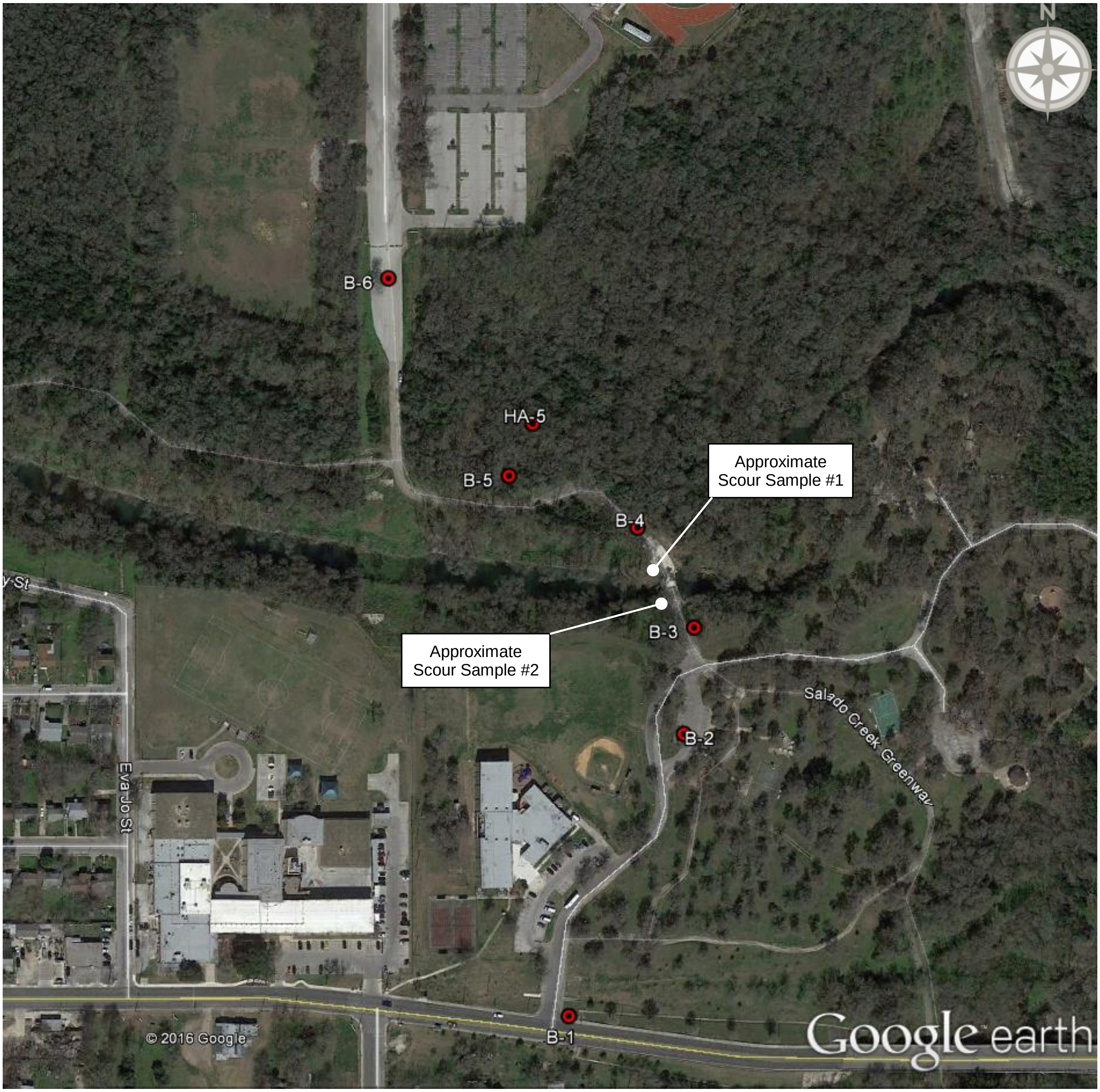
Martin Luther King Park Improvements
San Antonio, Texas

Date: July 6, 2016	Job No.: 2015-1018
Drawn By: TAS	Checked By: TJF
Approved By: SAH	Scale: N.T.S.

REVISIONS:

No.:	Date:	Description:

Figure 2b



ARIAS GEOPROFESSIONALS

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

BORING LOCATION PLAN

Martin Luther King Park Improvements
San Antonio, Texas

Date: July 6, 2016	Job No.: 2015-1018
Drawn By: TAS	Checked By: TJF
Approved By: SAH	Scale: N.T.S.

REVISIONS:

No.:	Date:	Description:

Figure 2b

Quicklime and Hydrated Lime for Soil Stabilization Report

Report Date: May 2, 2016

Soil Description: Clay, brown
Material Origin: Composite Sample: B-1 (0-2')
B-3 (0-2'), B-4 (0-2'), B-7 (0-2')
Date Sampled: May, 2, 2016
Test Method: ASTM D4318, ASTM D1140, ASTM D6276, and ASTM C977
Application: Treated Subgrade
Comments: *At 6 % of lime complies with City of San Antonio Specifications.

Trial No.	% Lime	Liquid Limit	PI
1	0	40	26
2	3	41	14
3	5	39	12
4	7	38	9
5	9	39	9
6	11	35	4

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

LIME SERIES

Martin Luther King Park Improvements
San Antonio, Texas

Date: June 23, 2016	Job No.: 2015-1018
Drawn By: TAS	Checked By: TJF
Approved By: SAH	Scale: N.T.S.

REVISIONS:

No.:	Date:	Description:

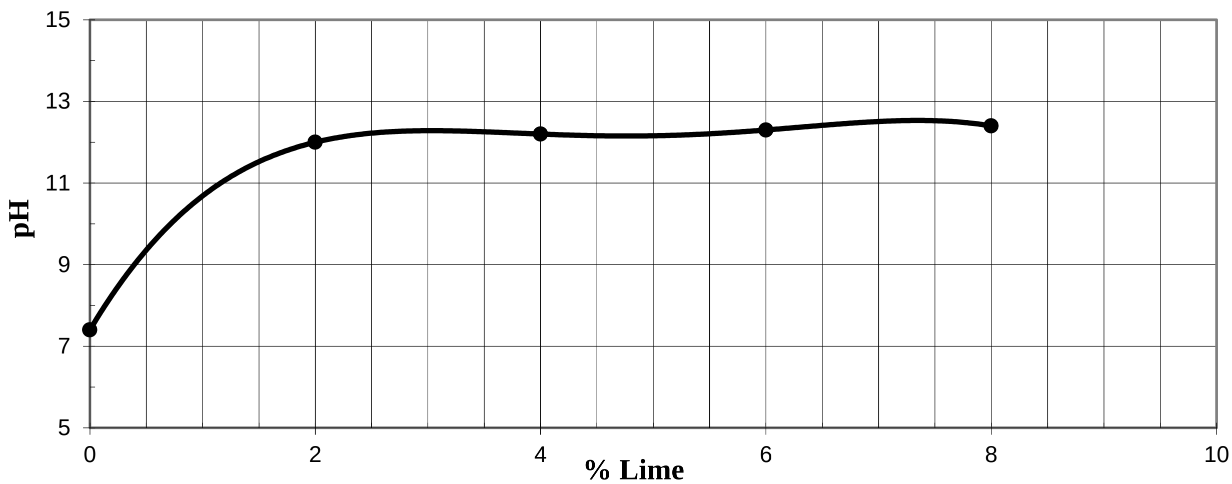
Figure 3

Soil Description: Fat Clay, dark brown
Material Origin: Composite Sample: B-3 (0-2'), B-4(0-2'), B-5 (0-2')
Date Sampled: August 12, 2016
Test Method: ASTM D4318, ASTM D1140, ASTM D6276, and ASTM C977

Application: Treated Subgrade

Trial No.	% Lime	Liquid Limit	PI	PH
1	0	55	38	7.4
2	2	41	9	12.0
3	4	42	6	12.2
4	6	43	6	12.3
5	8	43	5	12.4

Lime Series



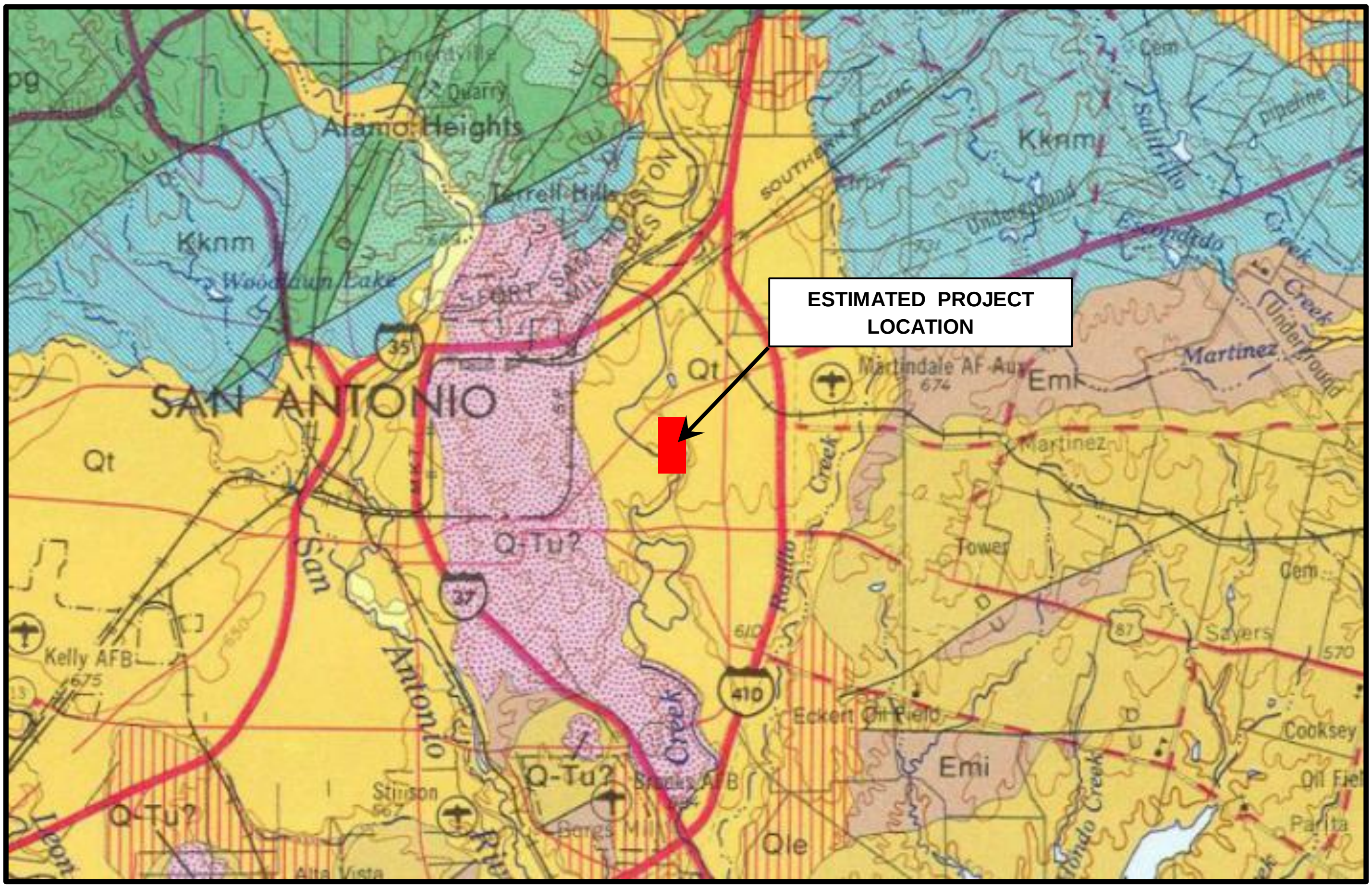
ARIAS GEOPROFESSIONALS
142 Chula Vista, San Antonio, Texas 78232
Phone: (210) 308-5884 • Fax: (210) 308-5886

LIME SERIES

Martin Luther King Park Improvements
San Antonio, Texas

REVISIONS:		Date: August 19, 2016	Job No.: 2015-1018
No.:	Date:	Description:	Drawn By: TAS	Checked By: TJF
			Approved By: SAH	Scale: N.T.S.

Figure 3



PORTION OF GEOLOGIC ATLAS OF TEXAS (San Antonio Sheet)

N

LEGEND

Symbol	Name	Age
Qt	Fluviatile Terrace Deposits	Quaternary Period / Holocene
Qle	Leona Formation	Quaternary Period / Pleistocene
Q-Tu	Uvalde Gravel	Quaternary Period / Pleistocene
Emi	Midway Group	Tertiary Period / Eocene
Kknm	Navarro Group & Marlbrook Marl	Cretaceous Period / Upper Cretaceous
Kpg	Pecan Gap Chalk	Cretaceous Period / Upper Cretaceous
Kau	Austin Chalk	Cretaceous Period / Upper Cretaceous

U D Fault Segment with Indication of Relative Movement

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

GEOLOGIC MAP

Martin Luther King Park Improvements
San Antonio, Texas

Date: October 21, 2015	Job No.: 2015-1018
Drawn By: TAS	Checked By: TJF
Approved By: SAH	Scale: N.T.S.

Figure 4

Photo 1 – View looking towards the east at the approximate location of Boring B-1 (marked with stake).

Photo 2 – View looking towards the north at the low water crossing and the approximate location of Boring B-3 (marked with pink and white paint).

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

SITE PHOTOS

Martin Luther King Park Improvements
San Antonio, Texas

Date: May 19, 2016	Job No.: 2015-1018
Drawn By: TAS	Checked By: TJF
Approved By: SAH	Scale: N.T.S.

Appendix A

Photo 3 – View looking towards the north at the bobcat drilling operation for the offset of Boring B-5.

Photo 4 – View looking towards the northwest at the approximate location of Boring B-7 (marked with pink and white paint).

ARIAS GEOPROFESSIONALS

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

SITE PHOTOS

Martin Luther King Park Improvements
San Antonio, Texas

Date: May 19, 2016	Job No.: 2015-1018
Drawn By: TAS	Checked By: TJF
Approved By: SAH	Scale: N.T.S.

Appendix A

APPENDIX B: BORING LOGS AND KEY TO TERMS

Boring Log No. B-1

Project: **MLK Park**
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 4/26/16
Elevation: 610.08 ft (By survey)
Coordinates: N: 13698309.61 E: 2153433.05
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200	DD	Uc
FAT CLAY with Sand (CH), very stiff to stiff, dark brown **(STRATUM I)**		SS	15					18			
		SS	19	16	52	36		11	83		
LEAN CLAY with Sand (CL), hard, tan brown **(STRATUM II)**	5	T	16	15	46	31	5.9		74	116	4.45
Well-graded GRAVEL with Silt and Sand (GW-GM), medium dense, tan and gray **(STRATUM III)**		SS	1					29	11 (GSD)		
CLAYEY GRAVEL with Sand (GC), dense, tan and light brown **(STRATUM IV)**		SS	13	12	43	31		20	33		
	10										
FAT CLAY (CH), stiff to firm, tan and gray, blocky with silty sand seams **(STRATUM IV)**		SS	26					9			
		SS	35					7			
	15										
FAT CLAY (CH), very stiff to hard, tan and gray **(STRATUM IV)** - blocky to 25 ft.	20	T	28				6.0			94	2.84
	25	T					8.0				
	30	SS						30			

Borehole terminated at 30 feet

Groundwater Data:
First encountered during drilling: 15-ft depth

Field Drilling Data:
Coordinates: Survey
Logged By: J. Ramos
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

Single flight auger: 0 - 30 ft

Nomenclature Used on Boring Log

Split Spoon (SS) | Thin-walled tube (T) | ▽ Water encountered during drilling

WC = Water Content (%)
PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
PP = Pocket Penetrometer (tsf)

N = SPT Blow Count
-200 = % Passing #200 Sieve
DD = Dry Density (pcf)
Uc = Compressive Strength (tsf)

Boring Log No. B-2

Project: **MLK Park**
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 4/26/16
Elevation: 606.53 ft (By survey)
Coordinates: N: 13698843.80 E: 2153658.34
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
ASPHALT, 2-inches; over BASE, 8-inches	0–1	GB	5	11	20	9		36
FAT CLAY (CH), stiff, dark brown **(STRATUM I)**	1–2	SS	20				9	
CLAYEY GRAVEL with Sand (GC), medium dense, brown **(STRATUM II)**	2–4	SS	10	14	45	31	17	22
Poorly-graded GRAVEL with Silt and Sand (GP-GM), dense, tan and gray **(STRATUM III)**	4–10	SS	2				34	12 (GSD)
		SS	2				35	12 (GSD)
- very dense and light tan and gray below 8 ft.		SS	2				52	15

Borehole terminated at 10 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Coordinates: Survey
Logged By: J. Ramos
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

Single flight auger: 0 - 10 ft

Nomenclature Used on Boring Log

Grab Sample (GB) | Split Spoon (SS)

WC = Water Content (%)
PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
N = SPT Blow Count

-200 = % Passing #200 Sieve

Boring Log No. B-3

Project: **MLK Park**
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 4/26/16
Elevation: 600.19 ft (By survey)
Coordinates: N: 13699062.60 E: 2153674.99
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200	DD	Uc
FAT CLAY (CH), stiff, dark brown **(STRATUM I)**		SS	15	21	62	41		8			
LEAN CLAY with Gravel (CL), very stiff, brown **(STRATUM II)**		SS	15					16			
	5	SS	14	16	47	31		20			
		SS	13					16			
SILTY GRAVEL with Sand (GM), dense to medium dense, tan and gray **(STRATUM III)**		SS	3					36	12 (GSD)		
	10	SS	3					33	15 (GSD)		
CLAYEY GRAVEL (GC), medium dense, tan and gray **(STRATUM III)**	15	SS	11					19			
FAT CLAY (CH), stiff to very stiff, tan and gray **(STRATUM IV)**	20	SS	34					13			
- blocky from 23 ft. to 25 ft.	25	T	27	25	82	57	8.25			96	3.26
- hard and tan and dark gray below 28 ft.	30	T	27				8.2				
SHALEY FAT CLAY (CH), hard, dark gray **(STRATUM IV)**	35	SS	26	24	77	53		33	100		
- very hard from 38.5 ft. to 40 ft.	40	SS	25					52			
	45	SS	25					36			
	50	SS	25					37			

Borehole terminated at 50 feet

Groundwater Data:
First encountered during drilling: 15-ft depth
After 15 minutes: 14.8-ft depth
Field Drilling Data:
Coordinates: Survey
Logged By: J. Ramos
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

Single flight auger: 0 - 50 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
PP = Pocket Penetrometer (tsf)

N = SPT Blow Count
-200 = % Passing #200 Sieve
DD = Dry Density (pcf)
Uc = Compressive Strength (tsf)

Boring Log No. B-4

Project: **MLK Park**
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 4/27/16
Elevation: 595.92 ft (By survey)
Coordinates: N: 13699256.83 E: 2153570.56
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200	DD	Uc
ASPHALT, 2.5-inches; over BASE, 6-inches		GB	6	14	22	8			21 (GSD)		
FAT CLAY (CH), soft to firm, dark brown **(STRATUM I)**		SS	24	19	53	34		4			
SANDY FAT CLAY (CH), stiff, gray brown **(STRATUM I)** - gray with brown below 6 ft.		T	26				2.2				
	5										
		T	26	19	59	40	1.75		70		
		SS	25					7			
	10										
		T	14				2.75			100	1.78
SILTY, CLAYEY SAND with Gravel (SC-SM), very loose, gray **(STRATUM III)**		SS	16					3	13		
	15										
FAT CLAY (CH), stiff to very stiff, gray and tan, with horizontal bedding **(STRATUM IV)**		SS	30					11			
	20										
		SS	29					10			
	25										
		SS	29					24			
	30										
SHALEY FAT CLAY (CH), very stiff, dark gray **(STRATUM V)**		SS	23					25			
	35										
SHALEY FAT CLAY (CH), hard, dark gray **(STRATUM V)**		SS	24					45			
	40										
		SS	23					44			
	45										
		SS	24					54			
	50										

Borehole terminated at 50 feet

Groundwater Data:
First encountered during drilling: 15-ft depth
After 20 minutes: 4-ft depth
Field Drilling Data:
Coordinates: Survey
Logged By: J. Ramos
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

Single flight auger: 0 - 50 ft

Nomenclature Used on Boring Log

Grab Sample (GB)
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
PP = Pocket Penetrometer (tsf)
N = SPT Blow Count
-200 = % Passing #200 Sieve
DD = Dry Density (pcf)
Uc = Compressive Strength (tsf)

2015-1018.GPJ 7/7/16 (BORING LOG SA13-02.ARIASSA12-01.GDT,LIBRARY2013-01.GLB)

Arias Geoprofessionals

Job No.: 2015-1018

Boring Log No. B-5

Project: **MLK Park**
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 4/29/16
Elevation: 603.10 ft (By survey)
Coordinates: N: 13699363.07 E: 2153300.33
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FAT CLAY (CH), stiff, dark brown **(STRATUM I)**	1	T	24	23	59	36	2.6		89
	2								
LEAN CLAY (CL), very stiff, dark brown **(STRATUM II)**	3	T	20	19	48	29	5.25		
	4								
	5	T	21				4.25		
- dark brown to brown below 6 ft. - hard from 6 ft. to 8 ft.	6								
	7	T	17	18	45	27	6.25		
	8								
	9	SS	23					7	
	10								

Borehole terminated at 10 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Coordinates: Survey
Logged By: J. Ramos
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

Single flight auger: 0 - 10 ft

Nomenclature Used on Boring Log

Thin-walled tube (T) Split Spoon (SS)

WC = Water Content (%)
PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
PP = Pocket Penetrometer (tsf)
N = SPT Blow Count
-200 = % Passing #200 Sieve

Boring Log No. B-6

Project: **MLK Park** **San Antonio, Texas**	Sampling Date: 4/27/16
	Elevation: 607.09 ft (By survey)
	Coordinates: N: 13699754.18 E: 2153066.79
Location: See Boring Location Plan	Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
ASPHALT, 12-inches	0–1								
FILL: FAT CLAY with Sand (CH), stiff, dark brown	1–2.5	SS	15	17	52	35		12	73
FAT CLAY (CH), stiff, dark brown to brown **(STRATUM I)**	2.5–4	SS	14					7	
LEAN CLAY (CL), very hard, dark brown to brown **(STRATUM II)**	4–6	T	13	16	41	25	12.3		
- hard from 6 ft. to 8 ft.	6–8	T	17				8.2		
- stiff to very stiff below 8 ft.	8–10	SS	18					13	

Borehole terminated at 10 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Coordinates: Survey
Logged By: J. Ramos
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

Single flight auger: 0 - 10 ft

Nomenclature Used on Boring Log

Split Spoon (SS)   Thin-walled tube (T)

WC = Water Content (%)
PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
PP = Pocket Penetrometer (tsf)
N = SPT Blow Count
-200 = % Passing #200 Sieve

Boring Log No. B-7

Project: **MLK Park**
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 4/26/16
Elevation: 656.33 ft (By survey)
Coordinates: N: 13702397.60 E: 2153572.65
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
ASPHALT, 5-inches; over BASE, 6-inches		GB	3	16	21	5			32
FILL: CLAYEY SAND with Gravel (SC), loose, dark brown, brown and light tan		SS	13	17	42	25		9	43
FILL: LEAN CLAY (CL), firm, dark brown, brown and light tan		SS	18	18	44	26		8	
- very stiff below 4 ft.	5	T	20	18	42	24	4		
FAT CLAY (CH), very stiff, dark brown **(STRATUM I)**		T	20				4.25		
	10	SS	19					9	
LEAN CLAY with Sand (CL), hard, light tan and gray **(STRATUM II)**		T	12	13	27	14	9.75		74
	15	T	10						
	20	SS	12					69	
SILTY GRAVEL with Sand (GM), dense, light tan and gray **(STRATUM III)**									
- hard sandy lean clay seams at 24 ft.	25	SS	6	12	25	13		47	67
- very hard sandy, silty lean clay seam at 28 ft.	30	SS	14					50/6"	

Borehole terminated at 30 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Coordinates: Survey
Logged By: J. Ramos
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

Single flight auger: 0 - 30 ft

Nomenclature Used on Boring Log

Grab Sample (GB)
Split Spoon (SS)
Thin-walled tube (T)

WC = Water Content (%)
PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
PP = Pocket Penetrometer (tsf)
N = SPT Blow Count
-200 = % Passing #200 Sieve

2015-1018.GPJ 7/7/16 (BORING LOG SA13-02,ARIASSA12-01.GDT,LIBRARY2013-01.GLB)

Hand Auger No. HA-5

Project: **MLK Park**
San Antonio, Texas

Sampling Date: 5/24/16

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	PL	LL	PI	-200
FAT to LEAN CLAY (CH-CL), dark brown	0-1	GB	14	53	39	89
	1-2	GB	13	40	27	86
	2-3	GB				
	3-4	GB	16	49	33	81
	4-5	GB	17	50	33	84
- brown from 5 ft. to 7 ft.	5-6	GB				
	6-7	GB	15	51	36	76
- tan brown from 7 ft. to 8 ft.	7-8	GB				
- tan below 8 ft.	8-9	GB				
	9-10	GB				

Excavation terminated at 10 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Logged By: J. Ramos
Driler: Arias & Associates, Inc.
Equipment: Hand auger

Hand Auger: 0 - 10 ft

Nomenclature Used on Boring Log

☒ Grab Sample (GB)

PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
-200 = % Passing #200 Sieve

Boring Log No. S-1

Project: **MLK Park**
San Antonio, Texas

Sampling Date:

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)		SN	PL	LL	PI	-200
CLAYEY GRAVEL with Sand (GC)	1		GB	18	72	54	30 (GSD)
	2						
Borehole terminated at 2 feet							

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:

Hand Auger: 0 - 2 ft

Nomenclature Used on Boring Log

☒ Grab Sample (GB)

PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
-200 = % Passing #200 Sieve

Boring Log No. S-2

Project: **MLK Park**
San Antonio, Texas

Sampling Date:

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)		SN	PL	LL	PI	-200
CLAYEY GRAVEL with Sand (GC)	1		GB	22	68	46	22 (GSD)
	2						
Borehole terminated at 2 feet							

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:

Hand Auger: 0 - 2 ft

Nomenclature Used on Boring Log

☒ Grab Sample (GB)

PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
-200 = % Passing #200 Sieve

2015-1018.GPJ 7/7/16 (BORING LOG SA13-02,ARIASSA12-01.GDT,LIBRARY2013-01.GLB)

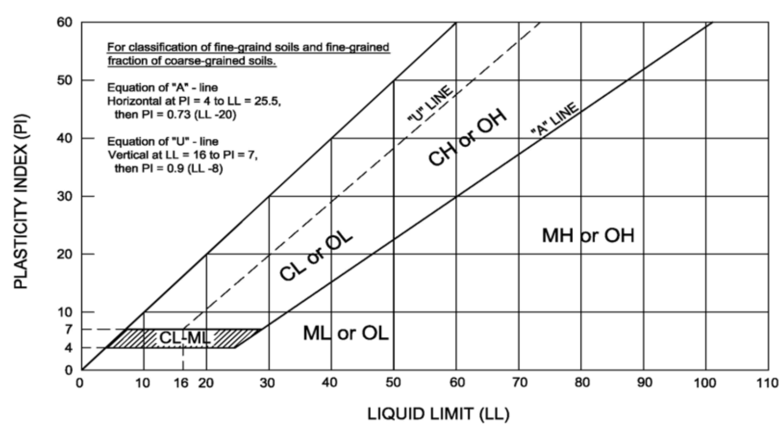
KEY TO TERMS AND SYMBOLS USED ON BORING LOGS

MAJOR DIVISIONS					GROUP SYMBOLS	DESCRIPTIONS
COARSE-GRAINED SOILS	More than half of material LARGER than No. 200 Sieve size	GRAVELS	More than Half of Coarse fraction is LARGER than No. 4 Sieve size	Clean Gravels (little or no Fines)	GW	Well-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines
					GP	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines
				Gravels with Fines (Appreciable amount of Fines)	GM	Silty Gravels, Gravel-Sand-Silt Mixtures
					GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
		SANDS	More than half of Coarse fraction is SMALLER than No. 4 Sieve size	Clean Sands (little or no Fines)	SW	Well-Graded Sands, Gravelly Sands, Little or no Fines
					SP	Poorly-Graded Sands, Gravelly Sands, Little or no Fines
				Sands with Fines (Appreciable amount of Fines)	SM	Silty Sands, Sand-Silt Mixtures
					SC	Clayey Sands, Sand-Clay Mixtures
FINE-GRAINED SOILS	More than half of material SMALLER than No. 200 Sieve size	SILTS & CLAYS		Liquid Limit less than 50	ML	Inorganic Silts & Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity
					CL	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
		SILTS & CLAYS		Liquid Limit greater than 50	MH	Inorganic Silts, Micaceous or Diatomaceous Fine Sand or Silty Soils, Elastic Silts
					CH	Inorganic Clays of High Plasticity, Fat Clays
FORMATIONAL MATERIALS		SANDSTONE				Massive Sandstones, Sandstones with Gravel Clasts
		MARLSTONE				Indurated Argillaceous Limestones
		LIMESTONE				Massive or Weakly Bedded Limestones
		CLAYSTONE				Mudstone or Massive Claystones
		CHALK				Massive or Poorly Bedded Chalk Deposits
		MARINE CLAYS				Cretaceous Clay Deposits
		GROUNDWATER			▼	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[A]				Soil Classification	
				Group Symbol	Group Name[B]
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	Gravels (More than 50% of coarse fraction retained on No. 4 sieve)	Clean Gravels (Less than 5% fines[C])	Cu ≥ 4 and 1 ≤ Cc ≤ 3[D]	GW	Well-Graded Gravel[E]
			Cu < 4 and/or [Cc < or Cc > 3][D]	GP	Poorly-Graded Gravel[E]
		Gravels with Fines (More than 12% fines[C])	Fines classify as ML or MH	GM	Silty Gravel[E,F,G]
			Fines classify as CL or CH	GC	Clayey Gravel[E,F,G]
	Sands (50% or more of coarse fraction passes No. 4 sieve)	Clean Sands (Less than 5% fines[H])	Cu ≥ 6 and 1 ≤ Cc ≤ 3[D]	SW	Well-Graded Sand[I]
			Cu < 6 and/or [Cc < or Cc > 3][D]	SP	Poorly-Graded Sand[I]
		Sands with Fines (More than 12% fines[H])	Fines classify as ML or MH	SM	Silty Sand[F,G,I]
			Fines classify as CL or CH	SC	Clayey Sand[F,G,I]
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean Clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	(Liquid limit - oven dried) / (Liquid limit - not dried) <0.75	OL	Organic Clay[K,L,M,N] / Organi Silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat Clay[K,L,M]
			PI plots on or below "A" line	MH	Elastic Silt[K,L,M]
		organic	(Liquid limit - oven dried) / (Liquid limit - not dried) <0.75	OH	Organic Clay[K,L,M,P] / Organic Silt[K,L,M,Q]
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name

[C] 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

[D] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[E] If soil contains ≥ 15% sand, add "with sand" to group name

[F] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM

[G] If fines are organic, add "with organic fines" to group name

[H] 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

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name

[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay

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

[L] If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name

[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name

[N] PI ≥ 4 and plots on or above "A" line

[O] PI < 4 or plots below "A" line

[P] PI plots on or above "A" line

[Q] PI plots below "A" line

TERMINOLOGY

Boulders	Over 12-inches (300mm)	**Parting**	Inclusion < 1/8-inch thick extending through samples
Cobbles	12-inches to 3-inches (300mm to 75mm)	**Seam**	Inclusion 1/8-inch to 3-inches thick extending through sample
Gravel	3-inches to No. 4 sieve (75mm to 4.75mm)	**Layer**	Inclusion > 3-inches thick extending through sample
Sand	No. 4 sieve to No. 200 sieve (4.75mm to 0.075mm)		
Silt or Clay	Passing No. 200 sieve (0.075mm)		
Calcareous	Containing appreciable quantities of calcium carbonate, generally nodular		

Stratified	Alternating layers of varying material or color with layers at least 6mm thick
Laminated	Alternating layers of varying material or color with the layers less than 6mm thick
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay
Homogeneous	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.

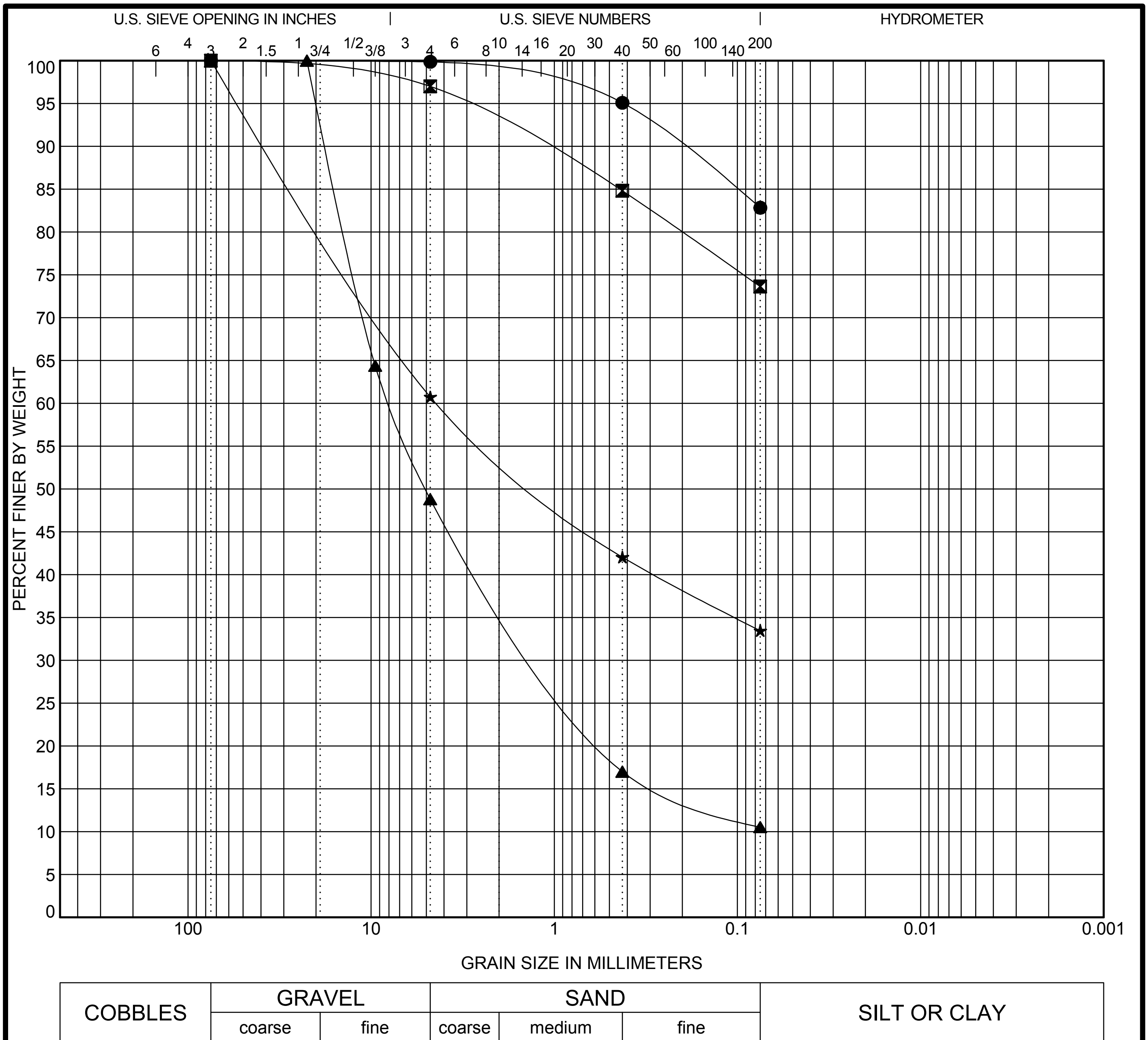
SPT N values and blow counts for those intervals where the sampler could not be advanced for the required 18-inch penetration are shown on the soil boring log. If the test was terminated during the 6-inch seating interval or after 10 hammer blows were applied used and no advancement of the sampler was 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 designations 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
Determining Sulfate Content in Soils	Tex-145-E	n/a
Lime Series	ASTM C 977	n/a

The laboratory results are reported on the soil boring log.

APPENDIX D: GRAIN SIZE DISTRIBUTION CURVES



	Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	**1**		**2.0**	**FAT CLAY with SAND (CH)**	**52**	**16**	**36**		
⊠	**1**		**4.0**	**LEAN CLAY with SAND (CL)**	**46**	**15**	**31**		
▲	**1**		**6.0**	**WELL-GRADED GRAVEL with SILT and SAND (GW-GM)**				**2.55**	**119.95**
★	**1**		**8.0**	**CLAYEY GRAVEL with SAND (GC)**	**43**	**12**	**31**		

	Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**1**	**2.0**	**75**				**0.1**	**17.0**	**82.8**	
⊠	**1**	**4.0**	**75**				**3.0**	**23.4**	**73.6**	
▲	**1**	**6.0**	**22.4**	**7.819**	**1.14**		**51.2**	**38.3**	**10.5**	
★	**1**	**8.0**	**75**	**4.317**			**39.3**	**27.3**	**33.5**	

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

142 Chula Vista
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GRAIN SIZE DISTRIBUTION

Project: MLK Park

Location: See Boring Location Plan

Job No.: 2015-1018

2015-1018.GPJ 5/19/16 (GRAIN SIZE ARIAS.US_LAB.GDT LIBRARY2013-01.GLB)

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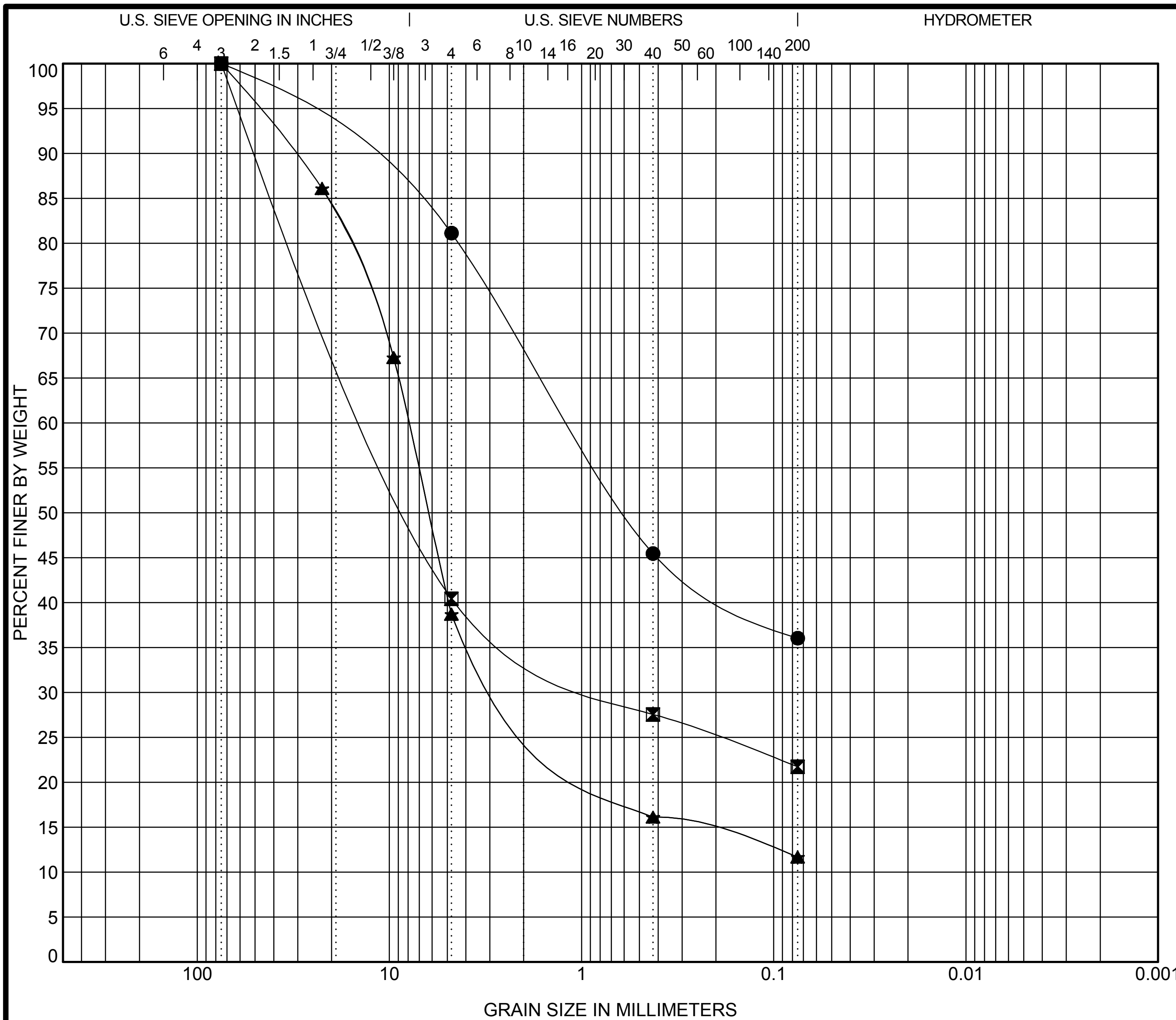

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	**2**		**0.2**	**BASE: CLAYEY SAND with GRAVEL (SC)**	**20**	**11**	**9**		
⊠	**2**		**2.5**	**CLAYEY GRAVEL with SAND (GC)**	**45**	**14**	**31**		
▲	**2**		**4.0**	**POORLY-GRADED GRAVEL with SILT and SAND (GP-GM)**				**11.31**	**203.99**
★	**2**		**6.0**	**POORLY-GRADED GRAVEL with SILT and SAND (GP-GM)**				**11.31**	**203.99**

	Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**2**	**0.2**	**75**	**1.137**			**18.9**	**45.1**	**36.0**	
⊠	**2**	**2.5**	**75**	**11.757**	**0.671**		**59.6**	**18.7**	**21.7**	
▲	**2**	**4.0**	**22.4**	**7.965**	**1.875**		**47.4**	**27.0**	**11.7**	
★	**2**	**6.0**	**75**	**7.965**	**1.875**		**61.3**	**27.0**	**11.7**	

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Location: See Boring Location Plan

Job No.: 2015-1018

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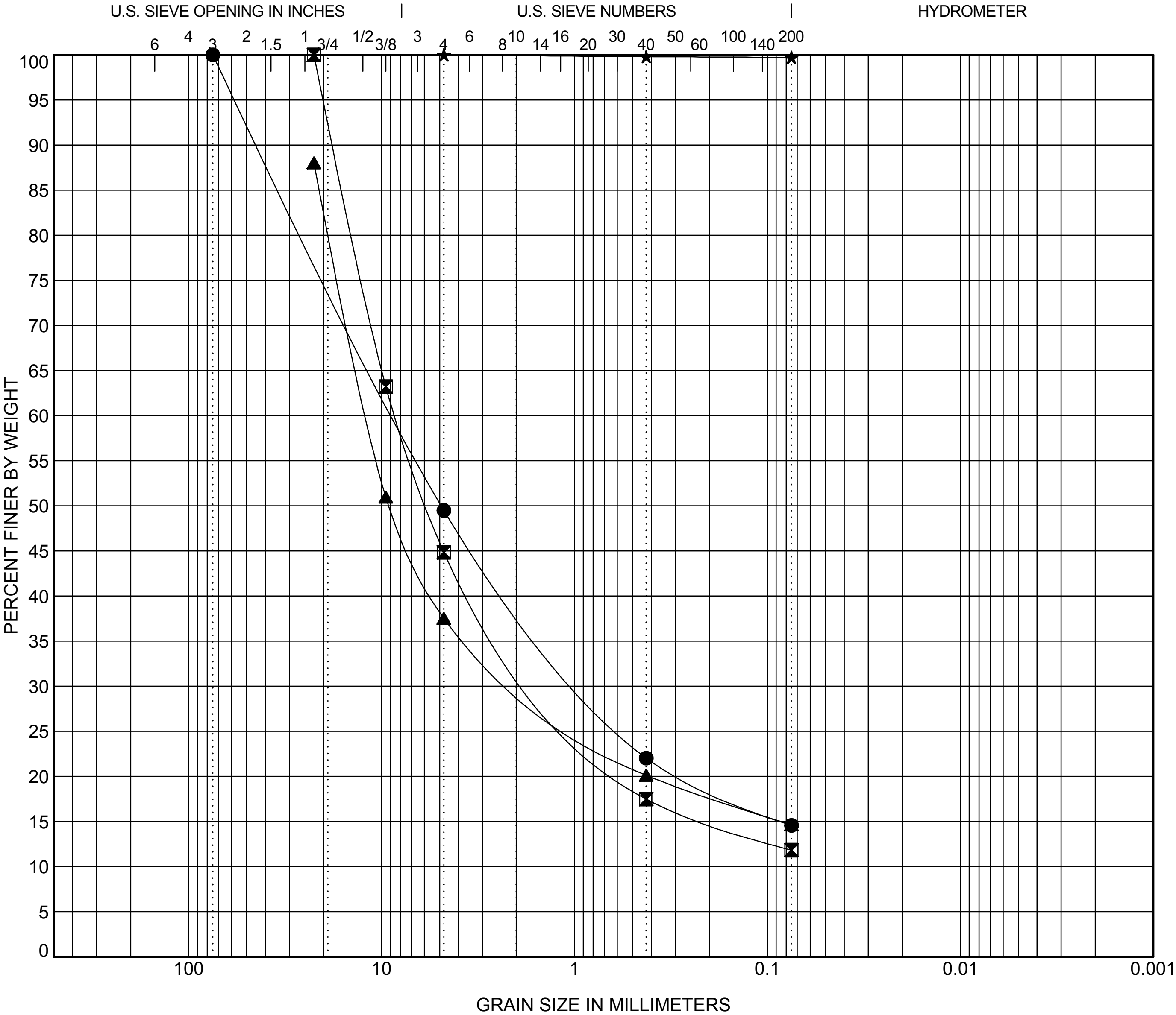

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	**2**		**8.0**	**POORLY-GRADED GRAVEL with SILT and SAND (GP-GM)**					
⧗	**3**		**8.0**	**SILTY GRAVEL with SAND (GM)**				**4.55**	**195.78**
▲	**3**		**10.0**	**SILTY GRAVEL with SAND (GM)**					
★	**3**		**33.0**	**FAT CLAY (CH)**	**77**	**24**	**53**		

	Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**2**	**8.0**	**75**	**8.437**	**0.857**		**50.5**	**34.9**	**14.6**	
⧗	**3**	**8.0**	**22.4**	**8.414**	**1.283**		**55.2**	**33.0**	**11.8**	
▲	**3**	**10.0**	**22.4**	**11.711**	**1.674**		**50.5**	**22.9**	**14.6**	
★	**3**	**33.0**	**4.75**				**0.0**	**0.3**	**99.7**	

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

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GRAIN SIZE DISTRIBUTION

Project: MLK Park

Location: See Boring Location Plan

Job No.: 2015-1018

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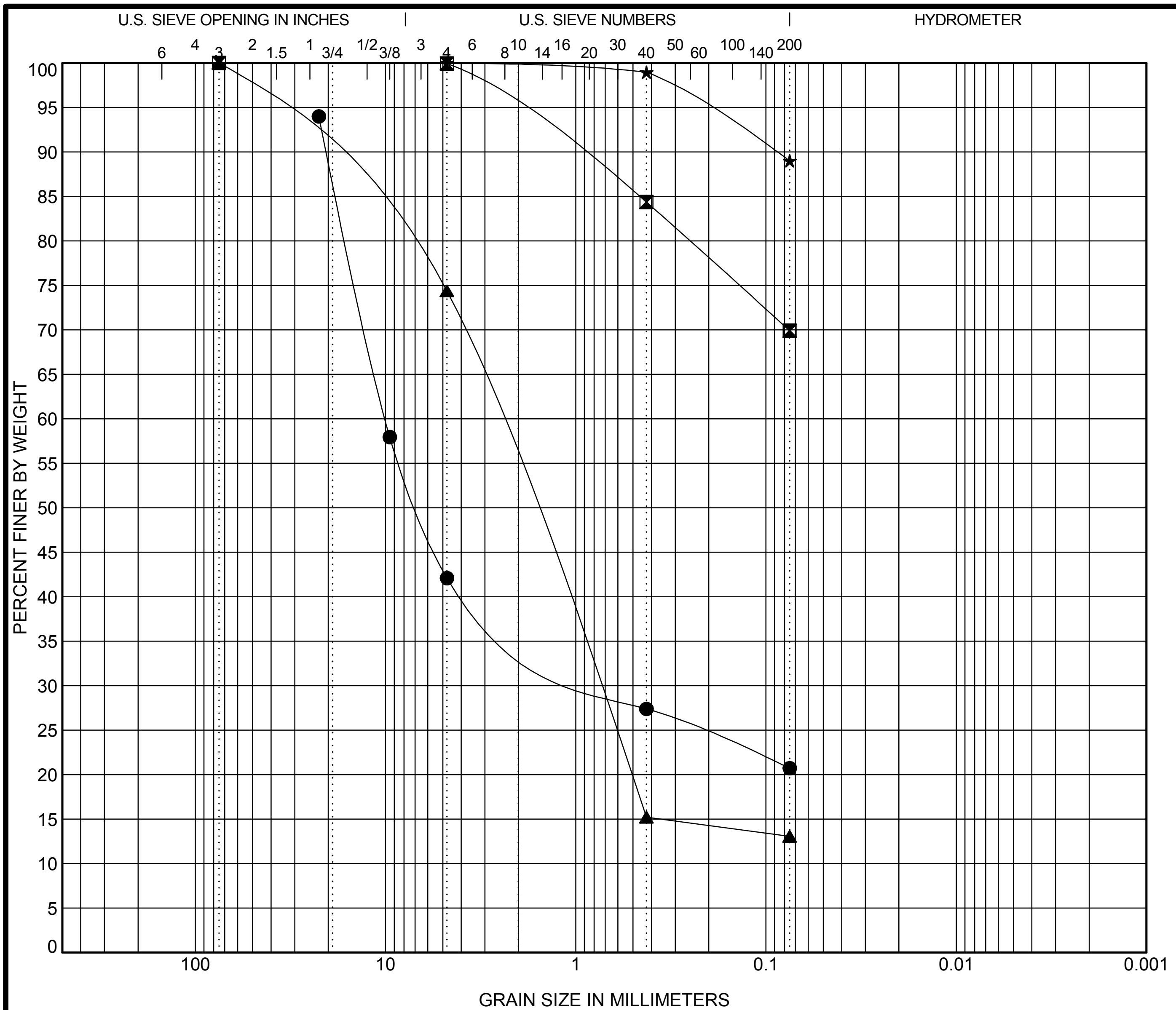

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	**4**		**0.0**	**BASE: CLAYEY GRAVEL with SAND (GC)**	**22**	**14**	**8**		
⊠	**4**		**6.0**	**SANDY FAT CLAY (CH)**	**59**	**19**	**40**		
▲	**4**		**13.0**	**SILTY, CLAYEY SAND with GRAVEL (SC-SM)**					
★	**5**		**0.0**	**FAT CLAY (CH)**	**59**	**23**	**36**		

	Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**4**	**0.0**	**22.4**	**9.977**	**0.653**		**51.9**	**21.4**	**20.7**	
⊠	**4**	**6.0**	**75**				**0.1**	**30.0**	**69.9**	
▲	**4**	**13.0**	**75**	**2.642**	**0.777**		**25.6**	**61.3**	**13.1**	
★	**5**	**0.0**	**4.75**				**0.0**	**11.0**	**89.0**	

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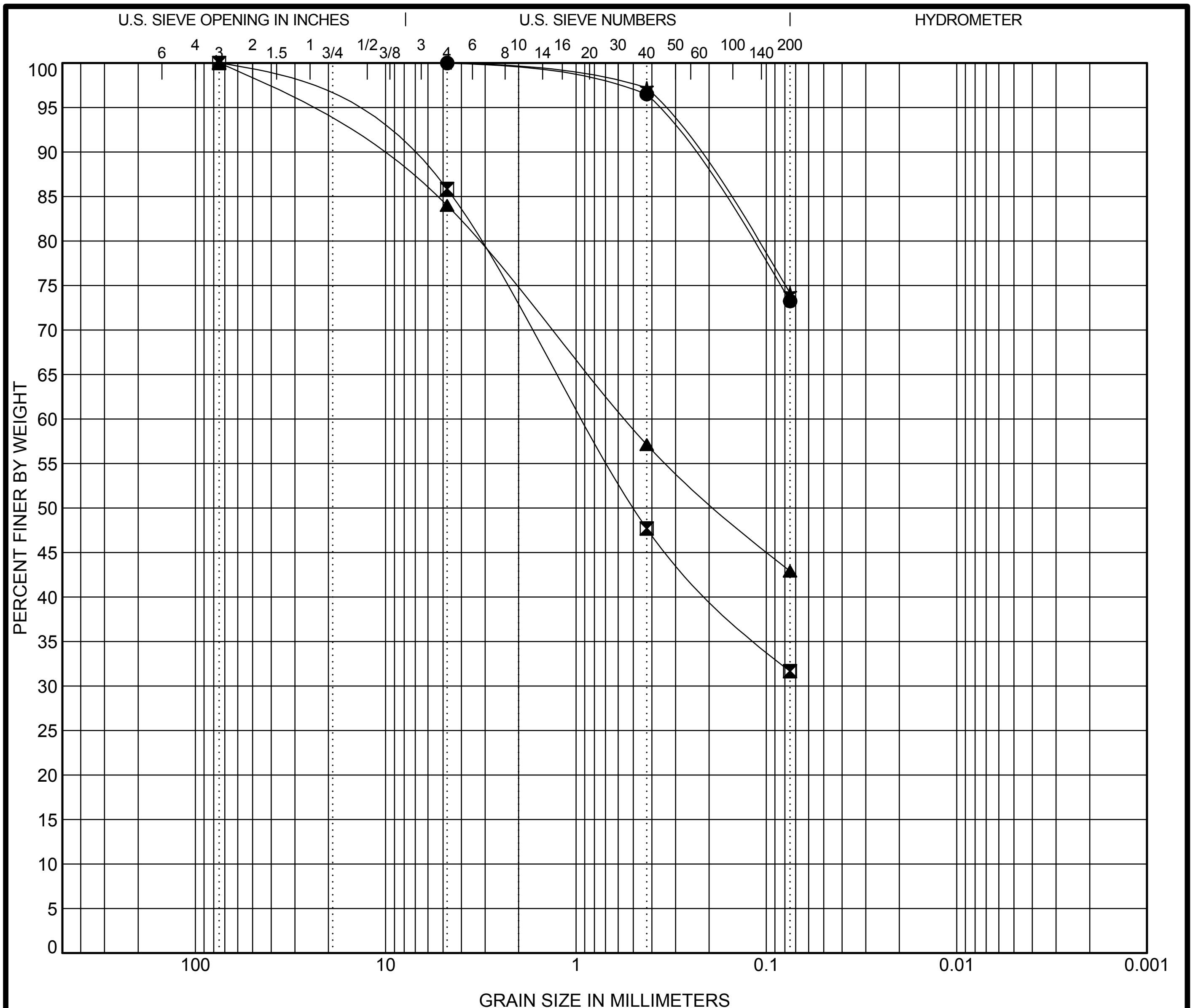

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	6		1.0	**FAT CLAY with SAND (CH)**	52	17	35		
⧗	7		0.4	**BASE: SILTY, CLAYEY SAND (SC-SM)**	21	16	5		
▲	7		1.0	**CLAYEY SAND with GRAVEL (SC)**	42	17	25		
★	7		10.0	**LEAN CLAY with SAND (CL)**	27	13	14		

	Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt / %Clay
●	6	1.0	4.75				0.0	26.8	73.2
⧗	7	0.5	75	0.926			14.2	54.2	31.7
▲	7	1.0	75	0.55			16.0	41.1	42.9
★	7	10.0	4.75				0.0	25.9	74.1

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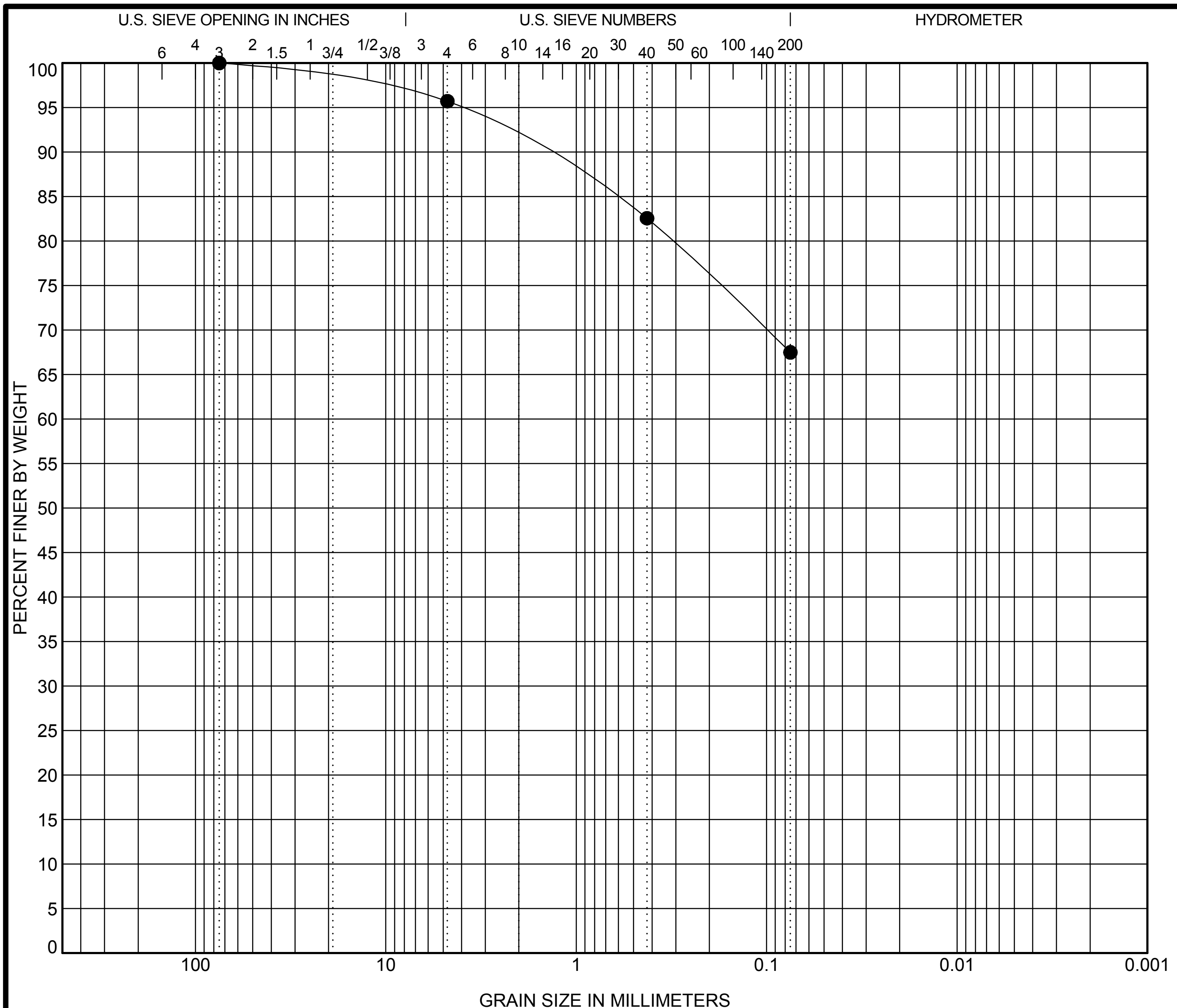

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	**7**		**23.0**	**SANDY LEAN CLAY (CL)**	**25**	**12**	**13**		

	Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**7**	**23.0**	**75**				**4.3**	**28.2**	**67.5**	

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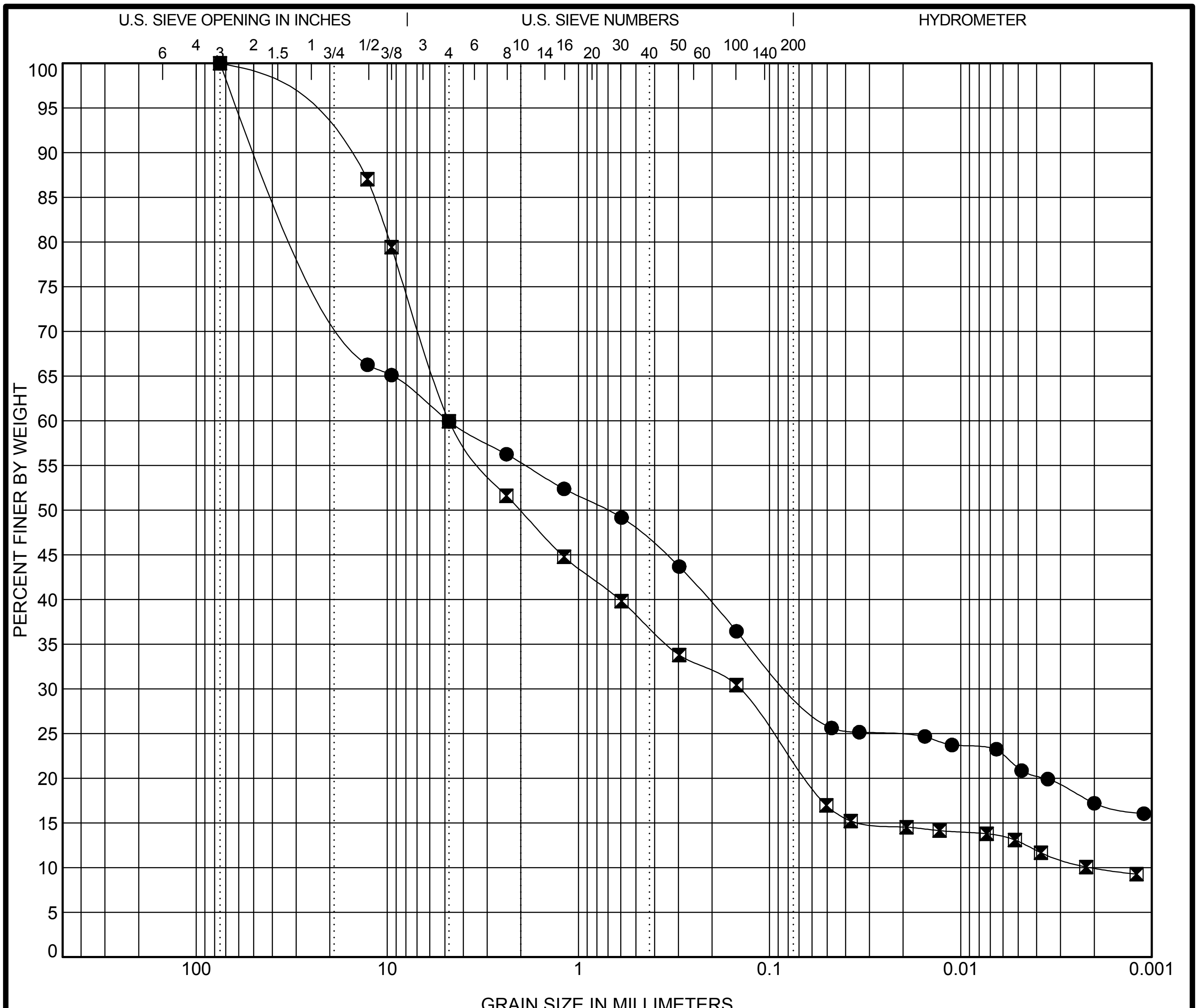

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
●	**Scour 1**		**0.0**	**CLAYEY GRAVEL with SAND (GC)**	**72**	**18**	**54**		
☒	**Scour 2**		**0.0**	**CLAYEY GRAVEL with SAND (GC)**	**68**	**22**	**46**	**2.08**	**2279.66**

	Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**Scour 1**	**0.0**	**75**	**4.803**	**0.075**		**40.1**	**30.0**	**12.8**	**17.2**
☒	**Scour 2**	**0.0**	**75**	**4.759**	**0.144**	**0.002**	**40.1**	**38.0**	**12.0**	**9.9**

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

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GRAIN SIZE DISTRIBUTION

Project: MLK Park

Location: See Boring Location Plan

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APPENDIX E: 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 Geotechncial 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.

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APPENDIX F: PROJECT QUALITY ASSURANCE

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.

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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.***

Divorcing the GER from geotechnical field operations is almost always penny-wise and pound-foolish, helping to explain why "geo" issues are the number-one source of construction-industry claims and disputes.

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.) 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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