Geotechnical Engineering Study

Leon Creek Water Recycling Center (WRC) Interconnect to Media River Sewer Outfall San Antonio, Texas

Arias Job No. 2010-475

ARIAS & ASSOCIATES
Geotechnical • Environmental • Testing

Prepared For
CP&Y, Inc.

August 31, 2011

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Mr. Josh Marazzini, P.E.
CP&Y, Inc.
300 E. Sonterra Blvd., Suite 1250
San Antonio, Texas 78258

RE: Geotechnical Engineering Study
Leon Creek Water Recycling Center (WRC) Interconnect to Media River Sewer Outfall
San Antonio, Texas

Dear Mr. Marazzini:

Arias & Associates, Inc. (Arias) is pleased to submit the results of a Geotechnical Engineering Study for the proposed Leon Creek Water Recycling Center (WRC) Interconnect to Media River Sewer Outfall in San Antonio, Texas. Our findings and recommendations should be incorporated into the design and construction documents for the proposed development. Please consult with us as needed during any part of the design or construction process. The recommendations provided in this report supersede all previous draft report recommendations.

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. We recommend that the foundation, site work and construction be tested and observed by one of our representatives in accordance with the report recommendations.

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

Sincerely,
ARIAS & ASSOCIATES, INC.
TBPE Registration No: F-32

STATE OF TEXAS
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INTRODUCTION

The results of a Geotechnical Engineering Study for the Leon Creek Water Recycling Center (WRC) Interconnect to Media River Sewer Outfall in San Antonio, Texas are presented in this report. This project was authorized on September 24, 2010 by Mr. David Wiekel, P.E. of CPY&, Inc. by means of the Standard Agreement for Professional Services between CP&Y, Inc. and Arias & Associates, Inc. (Arias). The Notice-to-Proceed for the geotechnical engineering services was issued June 7, 2011 by Mr. Josh Marazzini, P.E. A preliminary report was issued June 28, 2011. Although the preliminary report was completed in June 2011, preparation of the final geotechnical report was delayed in order to incorporate topographic survey data provided by CP&Y, Inc.

SCOPE OF SERVICES

The purpose of this geotechnical engineering study was to establish engineering properties of the subsurface soil and groundwater conditions present at the site. The scope of the study is sufficient to provide geotechnical engineering criteria for use by design engineers in preparing the bridge and pipeline designs. Environmental studies, corrosivity testing, pavement engineering or analyses of slopes and/or retaining walls were beyond our authorized scope of services for this project.

PROJECT DESCRIPTION AND SITE DESCRIPTION

The planned project will consist of an approximate 12,000 linear foot, 60-inch diameter gravity sewer main to convey raw wastewater from the Leon Creek Water Recycling Center (WRC) to the Medina River Sewer Outfall. The current proposed sewer main location begins at the Leon Creek WRC and travels northeast to cross Comanche Creek then turns to the southeast and ends at the Toyota manufacturing plant railroad easement. Our scope of services includes providing geotechnical design criteria for the proposed structures to be constructed along the sewer main alignment to include: (1) expanding an existing Flow Diversion Structure near the Leon Creek WRC, (2) placement of a new Flow Diversion Structure to tie-in the existing Flow Equalization Basin (FEB) drainage system to the new sewer main, (3) an aerial/structural crossing over Comanche Creek supported by drilled pier foundations and (4) trenchless installation methods at the railroad crossing adjacent to the Toyota Property.

The project site is located southwest of Mauermann Road and Old Pleasanton Road in Southern Bexar County, San Antonio, Texas. A Vicinity Map is included as Figure 1 in Appendix A. Geographically, the project area is situated to the west of Mitchell Lake at the confluence of Comanche Creek and Leon Creek. Locally, the existing ground surface within the project area is characterized by flat flood plains and a steep sided drainage course. Based on our observations, the south bank of Comanche Creek, near the area of the proposed pipeline crossing, has a vertical relief visually estimated to be about 20 feet, while the north bank has a visually estimated vertical relief of 40 feet.

At the time of our field exploration, the project area varied from being well developed within the Leon Creek WRC to an open farm field to a natural stream side that is heavily vegetated. Site photographs are included in Appendix A of this report.

SOIL BORINGS AND LABORATORY TESTING

Five (5) soil borings were drilled at the approximate locations shown on the attached Boring Location Plan included as Figure 2 in Appendix A. The borings were drilled to depths of approximately 15 to 50 feet below the existing ground surface on June 17, 2011. The boring depths were selected by CP&Y, Inc. based on the anticipated bearing depth of the proposed project element. Drilling was performed in general accordance with ASTM D1586 and ASTM D 1587 procedures for Split Spoon and Shelby Tube sampling techniques as described in Appendix C. A truck-mounted drill rig using continuous flight augers together with the sampling tools noted were used to secure the subsurface soil samples. After completion of drilling, the boreholes were backfilled using cuttings generated during the drilling process.

Samples of encountered materials were obtained by: (1) using a split-barrel sampler while performing the Standard Penetration Test (ASTM D 1586), (2) using a thin-walled tube sampler (ASTM D 1587), and (3) by taking material from the auger as it was advanced (ASTM D 1452). 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 for those intervals where the sampler was advanced for a 12-inch penetration after the initial 6-inch seating are shown on the individual boring logs included in Appendix B. If the test was terminated during the 6-inch seating interval, or after 25 hammer blows were applied where no advancement of the sampler was noted, the boring logs denote this condition as blow count during seating penetration. Penetrometer readings recorded for thin-walled tube samples that remained intact are also shown on the boring logs.

For each sample, Arias' field representative visually classified the soil within the split-barrel sampler and placed a portion into a plastic bag with zipper seal. The samples were then placed into wax-coated cardboard sample boxes designed for transporting soil specimens to the laboratory.

Subsequent to the drilling activities, ground surface elevations at the boring locations were measured and provided to us by CP&Y, Inc. A summary of the boring number, general location, corresponding project element, approximate ground surface elevation, approximate boring termination depth and approximate bottom of pipe/structure at each of the boring locations is provided in Table 1 below.

Table 1: Approximate Existing and Proposed Grades at New Structures

Boring No.	General Location	Description of Proposed Structure	Approximate Ground Surface Elevation (ft)	Approximate Boring Termination Elevation (ft)	Approximate Bottom of Pipe/Structure (ft)
B-1	A – Leon Creek WRC	Expand Existing Flow Diversion Structure	541.6 (by survey)	526.6	533.5
B-2	B – Leon Creek WRC	New Flow Diversion Structure	529.5 (by survey)	505.5	515.25
B-3	C – Texas A&M Property	Aerial Pipeline Crossing over Comanche Creek, West Side	526 (Note 2)	476	513.6
B-4	D – Texas A&M Property	Aerial Pipeline Crossing over Comanche Creek, East Side	537 (Note 2)	488.5	513
B-5	E – Toyota Property	Directional Boring under Railroad	521 (Note 3)	496	507

Notes:

1. The topographic survey data and approximate ground surface elevations were provided by CP&Y, Inc.
2. The approximate ground surface elevations for Locations C and D are assumed based on the Plan and Profile sheets provided by CP&Y, Inc. (Station 84+00 to 89+00, dated August 2011) and could vary from the actual locations. The approximate bottom of pipe/structure elevation shown for Locations C and D is in reference to the bottom of the diversion structure at the creek crossing.
3. Topographic survey data was not provided by CP&Y, Inc. for Location E (*i.e.,* Boring B-5), therefore an approximate ground surface elevation was provided based on an existing topographic survey. The boring location is referenced from the ground surface where the boring was drilled and not at the top of the railroad track.

Soil classifications and borehole logging were conducted during the exploration by one of our Engineering Technicians working under the supervision of the project Geotechnical Engineer. Final soil classifications, as seen on the attached boring logs, were determined in the laboratory based on laboratory and field test results and applicable ASTM procedures.

As a supplement to the field exploration, laboratory testing to determine soil water content, Atterberg limits, and percent passing the US Standard No. 200 sieve was conducted. The laboratory results are reported in the boring logs included in Appendix B. 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 applicable ASTM procedures with the specifications and definitions for these tests listed in Appendix C.

Remaining soil samples recovered from this exploration will be routinely discarded following submittal of this report.

SUBSURFACE CONDITIONS

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

Geology

The earth materials underlying the project site have been regionally mapped as the alluvial Terrace (Qt) deposits of Pleistocene age underlain by shallow marine or coastal deposits of the Midway Formation (Emi) of Eocene age (approximately 36 to 56 million years before present). The contact between the alluvial and shallow marine deposits represents a significant erosional time gap which could be irregular with depth within the project area. Locally, the materials encountered in the borings consist primarily of alluvial terrace soils comprised of clays, gravelly clays and clayey gravels in a stiff to very hard and medium dense condition. The underlying marine deposits consist of clays and claystone with scattered iron oxide and gypsum deposits and are generally in a hard to very hard condition. A Geologic Map is included as Figure 3 in Appendix A.

Generalized Site Stratigraphy and Engineering Properties

The generalized stratigraphy and soil properties for the interpreted strata are summarized in the following tables.

Table 2: Generalized Soil Conditions, Location A, Boring B-1

Stratum	Approx. Elevation	Depth, ft	Material Type	PI range	No. 200 range	N Range
I	541.6 to 537.6	0 to 4	LEAN CLAY (CL) trace gravel and calcareous deposits, dark brown and brown, very stiff to hard (possible fill)	26	88	23-27
II	537.6 to 533.6	4 to 8	LEAN CLAY (CL) trace calcareous deposits, light brown, hard	23	--	32-39
III	533.6 to 526.6	8 to 15	LEAN CLAY (CL), tan, hard	19-27	91	28-42

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
PP - Pocket Penetrometer (PP), tons per square foot

Notes:

1. Elevations provided by CP&Y, Inc.

Table 3: Generalized Soil Conditions, Location B, Boring B-2

Stratum	Approx. Elevation	Depth, ft	Material Type	PI range	No. 200 range	PP range	N Range
FILL	529.5 to 526	0 to 3.5	LEAN CLAY (CL) with sand and trace gravel, dark brown to brown, very stiff to hard	27	--		24-32
I	526 to 521.5	3.5 to 8	FAT CLAY (CH) dark brown to black, hard	33	95	--	28-34
II	521.5 to 516.5	8 to 13	LEAN CLAY (CL) with calcareous deposits, light brown, very hard	28	92	4.5+	--
III	516.5 to 505.5	13 to 24	LEAN CLAY (CL) with calcareous deposits, tan, very stiff to hard	19	--	3.75-4.5	32

Notes:

1. Elevations provided by CP&Y, Inc.

Table 4: Generalized Soil Conditions, West Side of Comanche Creek, Location C, Boring B-3

Stratum	Approx. Elevation	Depth, ft	Material Type	PI range	No. 200 range	PP range	N Range
I	526 to 516	0 to 10	FAT CLAY (CH) very dark brown to dark brown, stiff to very hard	32-36	95	4.5+	9-26
II	516 to 501	10 to 25	LEAN CLAY (CL) with sand and gypsum crystals, light brown, hard to very hard	26	82	4.25-4.5+	35
IIb	501 to 494	25 to 32	Clayey GRAVEL (GC) with sand, tan, medium dense	34	40	--	24
III	494 to 477.5	32 to 48.5	FAT CLAY (CH) with gravel, tan, very stiff to hard	42	94	--	25-36
IV	477.5 to 476	48.5 to 50	CLAYSTONE with iron oxide seams, gray, very hard	--	--	--	72

Notes:

1. Elevations provided by CP&Y, Inc.
2. The Stratum IIb Clayey GRAVEL (GC) was only observed at this location.

Table 5: Generalized Soil Conditions, East Side of Comanche Creek, Location D, Boring B-4

Stratum	Approx. Elevation	Depth, ft	Material Type	PI range	No. 200 range	PP range	N Range
I	537 to 533	0 to 4	Gravelly FAT CLAY (CH) with sand, brown, stiff to very stiff	28	59	--	13-23
III	533 to 497	4 to 40	FAT CLAY (CH) with iron oxide seams, tan and gray, very stiff to very hard	30-41	96-97	4.5+	20-77
IV	497 to 488.5	40 to 48.5	CLAYSTONE with cemented seams, gray, very hard	--	--	--	>100

Notes:

1. Elevations provided by CP&Y, Inc.
2. The Stratum II light brown, LEAN CLAY (CL) was not observed at this location.

Table 6: Generalized Soil Conditions, Location E, Boring B-5

Stratum	Approx. Elevation	Depth, ft	Material Type	PI range	No. 200 range	PP range	N Range
I	521 to 509	0 to 12	LEAN CLAY (CL) with sand, brown, stiff to hard	22	82-88	4.5+	12-15
III	509 to 486	12 to 25	LEAN CLAY (CL) tan, hard	27	--	3.25-3.75	--

Notes:

1. Elevations provided by CP&Y, Inc.
2. The Stratum II light brown, LEAN CLAY (CL) was not observed at this location.

Groundwater

A dry soil sampling method was used to obtain the soil samples at the project site. Groundwater was observed within one of the five borings during the soil sampling activities which were performed on June 17, 2011. The borings were left open for an approximate 24-hour period in order to obtain delayed groundwater readings and the depth to caving/sloughing. Groundwater observations made during drilling and following an approximate 24-hour period are noted on the individual borings logs and summarized in Table 7.

It should be noted that water levels in open boreholes may require several hours to several days to stabilize depending on the permeability of the soils. Groundwater levels at this site may be subject to seasonal conditions, recent rainfall, drought or temperature affects. Groundwater conditions may vary during construction from the conditions encountered in our soil borings. Importantly, South Texas, including the area of the project site, is currently experiencing drought conditions.

Table 7: Summary of the Groundwater Observations at Boring Locations

Boring No.	General Location	Approximate Ground Surface Elevation (ft)	Depth to Groundwater During Drilling Operations (ft)	Depth to Groundwater Observed After a 24-hour delay (ft)	Approximate Groundwater Elevation (ft)
B-1	A – Leon Creek WRC	541.6 (by survey)	Not Observed	Backfilled upon Completion	n/a
B-2	B – Leon Creek WRC	529.5 (by survey)	Not Observed	Backfilled upon Completion	n/a
B-3	C – Texas A&M Property	526 (Note 2)	**32**	**26** (Borehole caved at ~28.5 ft)	~500
B-4	D – Texas A&M Property	537 (Note 2)	Not Observed	**39.4** (Borehole caved at ~44.2 ft)	~497.6
B-5	E – Toyota Property	521 (Note 3)	Not Observed	Backfilled upon Completion	n/a

Notes:

1. The topographic survey data and approximate ground surface elevations were provided by CP&Y, Inc.
2. The approximate ground surface elevations for Locations C and D are assumed based on the Plan and Profile sheets provided by CP&Y, Inc. (Station 84+00 to 89+00, dated August 2011) and could vary from the actual locations. The approximate bottom of pipe/structure elevation shown for Locations C and D is in reference to the bottom of the diversion structure at the creek crossing.
3. Topographic survey data was not provided by CP&Y, Inc. for Location E (*i.e.,* Boring B-5), therefore an approximate ground surface elevation was provided based on an existing topographic survey. The boring location is referenced from the ground surface where the boring was drilled and not at the top of the railroad track.
4. Groundwater measurements were recorded during field exploration on July 17, 2011. Delayed groundwater measurements were recorded following an approximate 24-hour period.

Clay soils are generally not conducive to the presence of groundwater; however, pockets or seams of gravels, sands, silts or open fractures and joints can store and transmit “perched” groundwater flow or seepage. The gravelly and sandy soils encountered within the borings can store and transmit “perched” groundwater flow or seepage. Perched groundwater seepage can also occur within joints and factures or at strata interfaces, particularly clay/gravel, or soil/claystone interfaces. Seasonal weather conditions or other factors may dictate actual shallow groundwater conditions at the time of construction.

The installation of temporary piezometers (observation wells) can be performed to obtain more accurate groundwater data. Additionally, pump and recharge tests can be performed using the piezometers to aid in estimating groundwater seepage rates. Subsurface water readings and seepage rates will generally provide an indication of groundwater conditions at that respective location and time. If needed, this information can be used to assist the contractor in developing construction dewatering plans. We should note that installing piezometers and performing groundwater testing was beyond our authorized scope of services for this project. We can provide these services if desired.

AERIAL CROSSING FOUNDATION RECOMMENDATIONS

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

We have been informed by CP&Y, Inc. that it is desired to utilize straight-shaft drilled piers to support the proposed aerial crossing over Comanche Creek (*i.e.,* structure corresponding to Locations C and D). The piers when properly founded can help reduce foundation movement of the superstructure. Geotechnical design criteria for this foundation type in consideration of the site's expansive soil conditions are presented herein.

Expansive Soil Considerations

Structural damage can be caused by volume changes in clay soils. Clays can shrink when they lose water and swell (grow in volume) when they gain water. The potential of expansive clays to shrink and swell is typically related to the Plasticity Index (PI). Clays with a higher PI generally have a greater potential for soil volume changes due to moisture content variations. The soils found at this site are capable of swelling and shrinking in volume dependent on potentially changing soil water content conditions during or after construction. The term swelling soils implies not only the tendency to increase in volume when water is available, but also to decrease in volume or shrink if water is removed. Considering the plasticity of the site soils, these soils would have a moderate to high swell potential upon future changes in soil moisture content.

Several methods exist to evaluate swell potential of expansive clay soils. We have estimated potential heave for this site utilizing the TXDOT method (Tex 124-E). Using the TXDOT method, we estimate that the PVR is approximately ***2 to 4½ inches*** at this site considering the existing soil moisture conditions at the time of the sampling activities. This is a soil heave magnitude considering a change from a dry to wet soil moisture condition within the active zone due to climate variations.

Straight-Shaft Drilled Piers

Items influencing the type of foundation selected for the proposed aerial crossing include the design axial and lateral foundation loads, the presence of expansive clays, and the potential presence of groundwater. More specifically, the final pier dimensions, particularly to include the required length of pier, will be determined based on the foundation design loads, the depth of the active zone, the potential uplift force imposed by expansive soils within the active zone and the available side friction capacity and end-bearing capacity allotted to the subsurface stratigraphy (calculated allowable values are provided in Table 8 below). The active zone is the depth of the stratigraphy which is influenced by seasonal moisture variations. At the project site, this depth is estimated at approximately 15 feet. The difference in elevation between the existing ground surface at the boring locations and the final top-of-pier elevation at the bridge abutments and bents will also influence the final pier dimensions. The amount of cut and/or fill at the bridge abutments and bents are unknown at this time; however, the differences must be accounted for in the final pier design.

Recommendations for evaluation of axial capacity and lateral capacity are presented in the following table. Pier capacities for axial loading were evaluated based on design methodologies included in FHWA-IF-99-025 - Drilled Shafts: Construction Procedures and Design Methods. Both end bearing and side friction resistance may be used in evaluating the allowable bearing capacity of the pier shafts.

Table 8: Drilled Pier Axial Design Parameters for Aerial Crossing, Location C and D– Axial Capacity

Depth	Approximate Elevation (ft)	Material	Recommended Design Parameters: Allowable Skin Friction, psf (αc/FS)	Allowable End Bearing, psf (cN_c/FS)	Uplift Force, kips
West Bridge Abutment, Location C, Boring B-3					
0 to 5	526 to 521	FAT CLAY (CH)	Neglect Contribution		
5 to 15	521 to 511	FAT CLAY (CH) and LEAN CLAY (CL)	700	--	65D
15 to 25	511 to 501	LEAN CLAY (CL)	750	--	
25 to 38	501 to 488	CLAYEY GRAVEL (GC) and LEAN CLAY (CL)	800	12,000	
38 to 48.5	488 to 477.5	FAT CLAY (CH)	1,250	15,000	
48.5 to 50	477.5 to 476	CLAYSTONE	1,600	24,000	
East Bridge Abutment, Location D, Boring B-4					
0 to 5	537 to 532	FAT CLAY (CH)	Neglect Contribution		
5 to 15	532 to 522	FAT CLAY (CH)	700	--	75D
15 to 25	522 to 512	FAT CLAY (CH)	800	--	
25 to 40	512 to 497	FAT CLAY (CH)	1,250	15,000	
40 to 50	497 to 488.5	CLAYSTONE	1,600	24,000	
Constraints to be Imposed During Pier Design					
Minimum embedment depth			30 feet below existing ground surface (June 2011)		
Minimum shaft diameter			24 inches		

Notes:

1. Topographic survey information was provided by CP&Y, Inc.
2. *For straight shaft piers, the contribution of the soils for the top 5 feet of soil embedment and for a length equal to at least 1 pier diameter from the bottom of the shaft should be neglected in determination of friction capacity. The recommended design parameters include a factor of safety of 2 for skin friction and of 3 for end bearing.*
3. The uplift force resulting from expansion of soils in the active zone may be computed using the above formula where D is the shaft diameter in feet. For drilled straight-sided piers, the contribution from soils to resist uplift is the allowable skin friction resistance of the soils below the 15-foot deep estimated active zone. Sustained dead loads will also aid in resisting uplift forces. Pier depths greater than 30 feet may be required to resist expansive soil uplift forces.
4. The minimum embedment depth was selected to locate the pier base below the depth of seasonal moisture change and within a specified desired bearing stratum. Pier vertical reinforcing steel should be designed to resist the uplift forces from swelling soils. A minimum of 1% of the gross cross-sectional area should be considered and the final reinforcing requirements should be determined by the project structural engineer. Tensile rebar steel should be designed in accordance with ACI Code Requirements.
5. Total and differential settlement of piers is expected to be less than 1 inch and 0.5 inch, respectively. Estimated settlements are based on performance of properly installed piers in the San Antonio metropolitan area. A detailed settlement estimate is outside of the scope of this service.
6. 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. The grain size analysis curve for the sample retrieved within the creek area is included in Appendix D.

Since groundwater (Borings B-3 and B-4) and granular soils (Boring B-3) were present within the borings performed at the proposed bridge crossing, we anticipate that the construction of piers which extend below this depth range will require either: (1) the temporary casing method should the tip of the pier excavation and casing be extended, as necessary, in a relatively impervious clay stratum to adequately seal the drill hole from the excessive influx of groundwater, or (2) the slurry displacement method should the pier tip bear within water-bearing strata and/or if an adequate casing seal cannot be established. *The presence and location of groundwater and potentially caving soils should be confirmed before construction commences. It should be noted that high-torque drilling equipment capable of drilling in rock would be required at this site due to the very hard and dry clay, claystone, and dense gravel encountered during the drilling operations.* Drilled pier installation considerations are discussed further in Table 10.

Lateral pile analyses including capacity, maximum shear, and maximum bending moment should be evaluated by the project structural engineer using LPILE or similar software. In the following table, Arias presents geotechnical input parameters for the encountered soils. Please note that the depths to the top and bottom of each layer were interpreted using approximate elevation data at the explored boring locations and layer boundaries as shown on the boring logs.

Table 9: Drilled Pier Geotechnical Input Parameters for LPILE Analyses – Aerial Crossing, Location C and D

Depth (ft)	Approximate Elevation (ft)	Material	γ_e	C_u	ϕ	K (cyclic loading)	e_{50}
West Bridge Abutment, Location C, Boring B-3							
0 to 5	526 to 521	FAT CLAY (CH)	Neglect Contribution				
5 to 15	521 to 511	FAT CLAY (CH) and LEAN CLAY (CL)	120	3,000	0	400	0.005
15 to 25	511 to 501	LEAN CLAY (CL)	125	3,500	0	400	0.005
25 to 38	501 to 488	CLAYEY GRAVEL (GC) and LEAN CLAY (CL)	63	4,000	0	400	0.005
38 to 48.5	488 to 477.5	FAT CLAY (CH)	63	5,000	0	800	0.004
48.5 to 50	477.5 to 476	CLAYSTONE	68	8,000	0	800	0.004
East Bridge Abutment, Location D, Boring B-4							
0 to 5	537 to 532	FAT CLAY (CH)	Neglect Contribution				
5 to 15	532 to 522	FAT CLAY (CH)	120	3,000	0	400	0.005
15 to 25	522 to 512	FAT CLAY (CH)	125	4,000	0	400	0.005
25 to 40	512 to 497	FAT CLAY (CH)	63	5,000	0	800	0.004
40 to 50	497 to 488.5	CLAYSTONE	68	8,000	0	800	0.004

Where:

γ_e = effective soil unit weight, pcf
c_u = undrained soil shear strength, psf
ϕ = undrained angle of internal friction, degrees
K = modulus of subgrade reaction, pci
e_{50} = 50% strain value
Design depth to groundwater is 26 feet based on boring data

Drilled Piers Construction Considerations

The contractor should verify groundwater conditions before production pier installation begins. Comments pertaining to high-torque drilling equipment, groundwater, slurry, and temporary casing are based on generalized conditions encountered at the explored locations. Conditions at individual pier locations may differ from those presented and may require that these issues be implemented to successfully install piers. Construction considerations for drilled pier foundations are outlined in the following table.

Table 10: Drilled Pier Installation Considerations for Locations C and D

Recommended installation procedure	USACE refers to FHWA (FHWA-NHI-10-016, May 2010)
High-torque drilling equipment anticipated	Yes
Groundwater anticipated	Yes; groundwater observed at 32 feet during sampling activities; delayed groundwater measured at 26 to 39 feet below the existing ground surface at the time of the field exploration
Temporary casing anticipated	Yes
Slurry installation anticipated	Yes, if casing seal into relatively impervious clay soil cannot be achieved
Concrete placement	Same day as drilling
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 bearing material type at the base of excavation and cleanliness of base, should observe placement of reinforcing steel

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

- The clear spacing between rebar or behind the rebar cage should be at least 3 times the maximum size of coarse aggregate.
- Centralizers on the rebar cage should be installed to keep the cage properly positioned.
- Cross-bracing of a reinforcing cage may be used when fabricating, transporting, and/or lifting. However, experience has shown that cross-bracing can contribute to the development of voids in a concrete shaft. Therefore, we recommend the removal of the cross-bracing prior to lowering the cage in the open shaft.

- The use of a tremie should be employed so that concrete is directed in a controlled manner down the center of the shaft to the shaft bottom. The concrete should not be allowed to ricochet off the pier reinforcing steel nor off the pier side walls.

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

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

IBC Site Classification and Seismic Design Coefficients

Section 1613 of the International Building Code (2009) requires that every structure be designed and constructed to resist the effects of earthquake motions, with the seismic design category to be determined in accordance with Section 1613 or ASCE 7. Site classification according to the International Building Code (2009) is based on the soil profile encountered to 100-foot depth. The stratigraphy at the site location was explored to a maximum 50-foot depth.

Clay soils having similar consistency were extrapolated to be present between 50 and 100-foot depths. On the basis of the site class definitions included in Table 1613.5.2 and 1613.5.5 of the 2009 Code and the encountered generalized stratigraphy, we characterize the site as Site Class D.

Seismic design coefficients were determined using the on-line software, Seismic Hazard Curves and Uniform Response Spectra, version 5.1.0, dated February 10, 2011 accessed at (http://earthquake.usgs.gov/hazards/designmaps/javacalc.php). Analyses were performed considering the 2009 International Building Code. Input included zip code 78221 and Site Class D. Seismic design parameters for the site are summarized in the following table.

Table 11: Seismic Design Parameters

Site Classification	F_a	F_v	S_s	S_1
D	1.6	2.4	0.114g	0.027g

Where:

Fa	=	Site coefficient
Fv	=	Site coefficient
Ss	=	Mapped spectral response acceleration for short periods
S1	=	Mapped spectral response acceleration for a 1-second period

BELOW GRADE STRUCTURES AND PIPELINE DESIGN CONSIDERATIONS

Details regarding excavation, dewatering, site safety, shoring and excavation retention, selection of machinery and equipment, and benching and sloping requirements are considered construction means and methods to accomplish the work, and thus, are the sole responsibility of the Contractor. The information presented herein pertaining to groundwater control and trenching and shoring is for informational purposes only. Additional information should be collected by the Contractor, as they deem appropriate.

We understand that below grade structures and utilities are planned to include: (1) Location A – expand existing concrete diversion structure, (2) Location B – install new diversion structure for FEB drain line and (3) Location E – trenchless installation methods along existing railroad near the Toyota property. The net allowable bearing pressure at each of the proposed structure locations is provided below based on the results of the soil borings and the approximate elevations provided.

Table 12: Recommended Allowable Bearing Pressure for Drainage Structures

Boring No.	General Location	Approximate Ground Surface Elevation (ft)	Approximate Boring Termination Elevation (ft)	Approximate Bottom of Pipe/Structure (ft)	Anticipated Bearing Surface	Net Allowable Bearing Pressure (psf)
B-1	A – Leon Creek WRC	541.6 (by survey)	527	533.5	Tan LEAN CLAY (CL), hard	4,000
B-2	B – Leon Creek WRC	529.5 (by survey)	505.5	515.25	Tan LEAN CLAY (CL), hard	4,000
B-5	E – Toyota Property	521 (Note 2)	496	507	Tan LEAN CLAY (CL), hard	4,000

Notes:

1. The topographic survey data and approximate ground surface elevations were provided by CP&Y, Inc.
2. Topographic survey data was not provided by CP&Y, Inc. for Location E (*i.e.,* Boring B-5), therefore an approximate ground surface elevation was provided based on an existing topographic survey. The boring location is referenced from the ground surface where the boring was drilled and not at the top of the railroad track.

Trenchless Technology Considerations

Based on the information provided by CP&Y, Inc., we understand that trenchless boring methods will be utilized at Location E along the existing railroad near the Toyota Property. We understand that open-cut methods will be employed at all other structure locations.

Trenchless methods may include directional boring techniques or micro-tunneling and pipe-jacking techniques. These methods will be employed to install the pipe below and across the crossing. The size of the pipe and pipe flexibility will often determine what trenchless construction technique will be selected. Our experience is that directional boring techniques are often selected when more flexible, smaller diameter pipes are being installed. If directional boring techniques cannot be accomplished, then micro-tunneling and pipe-jacking may be the more feasible installation method.

The proposed boring depth is anticipated to be near the 15 foot depth but has not been finalized at this time. For roadway crossings, according to Special Provisions for TxDOT – San Antonio District Utility Permits, Revised December 5, 2003, Paragraph 6, Open Trenching or Boring Operations for Utility Work and Paragraph 8 Boring and Jacking and The San Antonio District Minimum Depth of Cover Table dated April 4, 2002, the depth of horizontal earth boring operations should be a minimum of 10 feet (minimum cover over the steel casing) below pavement section and 10 feet beyond the edge of the pavement at a given area. It is also recommended that this minimum depth guideline be used for the railroad crossing.

It is recommended that the Contractor review the boring logs prior to performing trenchless boring operations. The boring logs and Table 14 include our interpreted geotechnical design parameters typically used for trenchless technology. The information included herein indicates the soil consistency at the time of exploration, soil strength (SPT value, Penetrometer value), undrained cohesion (C, shear strength), ϕ value (angle of internal friction), applicable coefficient of active and passive soil pressures (K_a and K_p), and effective soil unit weight (γ'). The soil design parameters as presented in the referenced table are meant to supplement the Boring Logs to provide general baseline information of the anticipated geotechnical conditions and constructability issues in constructing the pipeline, which bidders should take into consideration. *The purpose of this report is not intended as a sole reliance for bid development; additional studies may be required to reduce the risk of unanticipated conditions.*

The soil parameters provided in this report can be used to assess temporary pipe installation techniques. That is, temporary access pits should be designed to consider the short-term lateral earth pressures and base stability. The soil parameters can be used to calculate estimated lateral earth pressures for the short-term construction condition. Furthermore, the soil parameters can be used to assess the passive resistance of a reaction block or anchor associated with pipe-jacking. A factor of safety of at least 2 should be used for a short-term passive resistance condition.

Groundwater Control

During the June 2011 field exploration, groundwater was not encountered at the boring locations performed at the below-grade structures and railroad crossing (groundwater was however, encountered at the aerial crossing near Locations C and D). Provided that similar groundwater conditions are present during construction, mechanical dewatering may not to be required for the planned construction. If water seeps into the excavation, sumps and pumps may be an effective means for removing the water given the clay soil conditions encountered.

Although groundwater was not encountered within the borings performed for the below-grade structures and railroad crossing, groundwater could be encountered during construction due to periods of inclement weather, the presence of granular soils not identified within the borings, and the proximity of the nearby creek. Accordingly, the contractor should be prepared with appropriate dewatering measures to dewater the site, as necessary, to allow for the proposed construction. Dewatering could vary from open sump pumping (as noted) to the use of deep wells that may be required to lower the water table. The dewatering requirements will depend upon the site conditions at the time of construction and the proposed installation methods.

Trenching and Shoring

The Occupational Safety and Health Administration (OSHA) Standards 29 CFR, Part 1926, Subpart P addresses excavation trenching and shoring. The regulations provide options for the design of sloping and benched excavations and for vertical trench excavations. The contractor should be aware that slope height, slope inclination, and/or excavation depths should in no case exceed those specified in local, state, or federal safety regulations, e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations, such regulations are strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties.

Appropriate trench excavation methods and safety design will depend on the soil and groundwater conditions encountered during construction. We emphasize that differing soil conditions may be present at locations and depths than disclosed by the widely-spaced borings. Undisclosed material may be less stable than the soils encountered in our borings and/or groundwater may be present. Such differing conditions could lead to excavation instability. Consequently, flatter slopes and dewatering techniques may be required in these areas.

OSHA requires that the excavations be carefully monitored by a competent person making daily construction inspections. These inspections are required to verify that the excavations are constructed in accordance with the intent of OSHA regulations and the trench safety design. If deeper excavations are necessary or if actual soil conditions vary from the borings, the trench safety design should be reviewed and revisions should be made to the design as needed based on encountered conditions. The effects of changed weather conditions, surcharge loadings, and cuts into adjacent backfills of existing utilities are critical items that should be evaluated by the inspector. The flow of water into the base and sides of the excavation and the presence of any surface slope cracks should also be carefully monitored by the Trench Safety Engineer.

Regardless of excavation depth, we recommend that all vehicles and material stockpiles be located at distance equal to or preferably greater than the trench vertical height. The trench safety design should consider the impacts of surcharge loads that may result from the presence of material stockpiles, equipment traffic, and other loadings in close proximity to the trench.

OSHA Soil Classifications

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

Table 13: OSHA Soil Classifications

Stratum	Description	OSHA Classification
I	Dark Brown to Brown, CLAY (CH-CL)	B
II	Light Brown, LEAN CLAY (CL)	B
IIb	Clayey GRAVEL (GC)	C
III	Tan and Gray, CLAY (CH-CL)	C
IV	Gray, CLAYSTONE	B or C

****It must be noted that layered slopes cannot be steeper at the top than the underlying slope and that <u>all materials that become wet or are situated below the water table must be classified as Type "C" soils</u>. The OSHA publication should be referenced for layered soil conditions, benching, etc.****

For excavations less than 20 feet deep, the maximum allowable slope for Type "C" soils is 1.5H:1V (34°), for Type "B" soils is 1H:1V (45°) and for Type "A" soils is ¾H:1V (53°). It should be noted that the table and allowable slopes above are for <u>temporary</u> slopes.

Lateral Earth Pressures

Lateral earth pressures for design of temporary trench shoring and permanent below grade structures can be assessed utilizing the soil design parameters provided in the following table. Active earth pressure can be used to assess temporary trench shoring. At rest earth pressure should be used to assess permanently buried structures.

Table 14: Lateral Earth Pressure Parameters

Stratum	Description	Soil Unit Weight, γ_e	Undrained Conditions		Drained Conditions		k_a	k_p*	k_o
			C	φ	C'	φ'			
I	CLAY (CH-CL)	125	1,000	0	0	17	0.55	1.8	0.71
II	LEAN CLAY (CL)	125	1,500	0	0	19	0.50	2.0	0.68
IIb	Clayey GRAVEL (GC)	125	0	26	0	26	0.39	2.6	0.56
III	CLAY (CH)	125	2,000	0	0	17	0.55	1.8	0.71
IV	CLAYSTONE	125	5,000	0	0	17	0.55	1.8	0.71

where: **γ_e** = effective soil unit weight, pcf

C = undrained soil shear strength, psf

φ = angle of internal friction, deg.

C' = drained soil shear strength, psf

φ' = effective stress angle of internal friction, deg.

k_a = coefficient of active earth pressure

k_p = coefficient of passive earth pressure

k_o = coefficient of at-rest earth pressure

***** = These are ultimate passive earth pressure coefficients. A factor of safety of at least 2 should be applied when determining passive resistance.

Short-term lateral earth pressures on the trench shoring can be calculated considering a rectangular pressure diagram having a magnitude of:

$$(\gamma)(H)(ka)$$

where γ and k_a are provided above and H is the depth of excavation in feet. Any surcharge loads including equipment loads, and soil stockpiles and hydrostatic pressures should be added to this value as required.

GENERAL COMMENTS

This report was prepared as an instrument of service for this project exclusively for the use of CP&Y, Inc., Inc. and the project design team. If the development plans change relative to the proposed construction or anticipated loading conditions, or if different subsurface conditions are encountered, 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.

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 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.

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, we should be engaged by the Owner to provide quality assurance testing. Our services, as a minimum, will be to observe and confirm that the encountered materials during earthwork for site subgrade improvement and pipeline installation are consistent with those encountered during this study. We also should verify that the materials used as part of subgrade improvement, pipeline installation, and other pertinent elements conform to the project specifications and that placement of these materials is in conformance with the specifications. With regard to drilled pier construction, we should be engaged to observe and evaluate the foundation installation to determine that the actual bearing materials are consistent with those encountered during the field exploration and to observe and document the pier installation process. In the event that Arias is not retained to provide quality assurance 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.

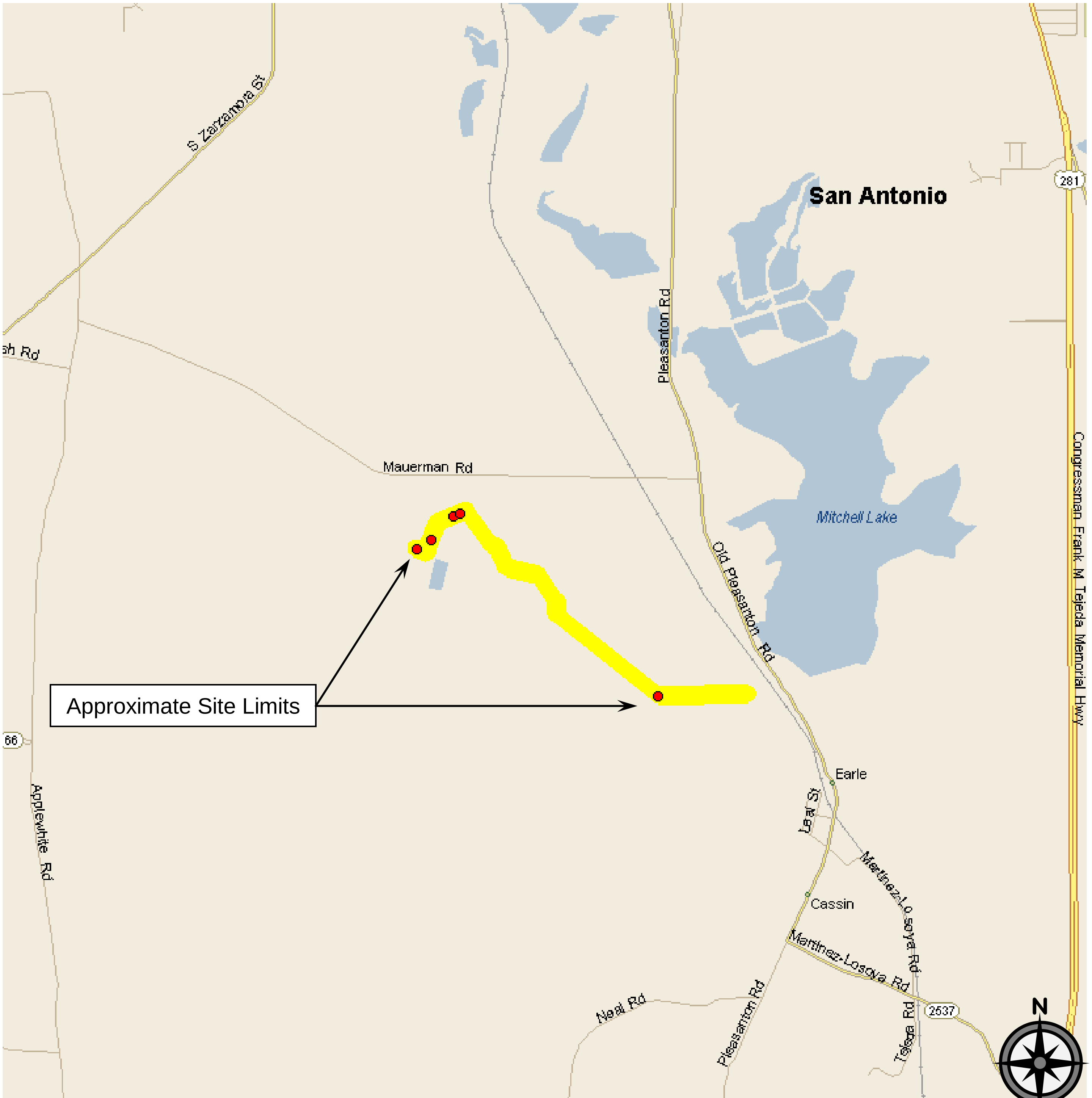
Subsurface Variations

Soil and groundwater conditions may vary away from 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.

Standard of Care

This report has been prepared in accordance with generally accepted geotechnical engineering practice with a degree of care and skill ordinarily exercised by reputable geotechnical engineers practicing in this area and the area of the site.

APPENDIX A: FIGURES AND SITE PHOTOGRAPHS



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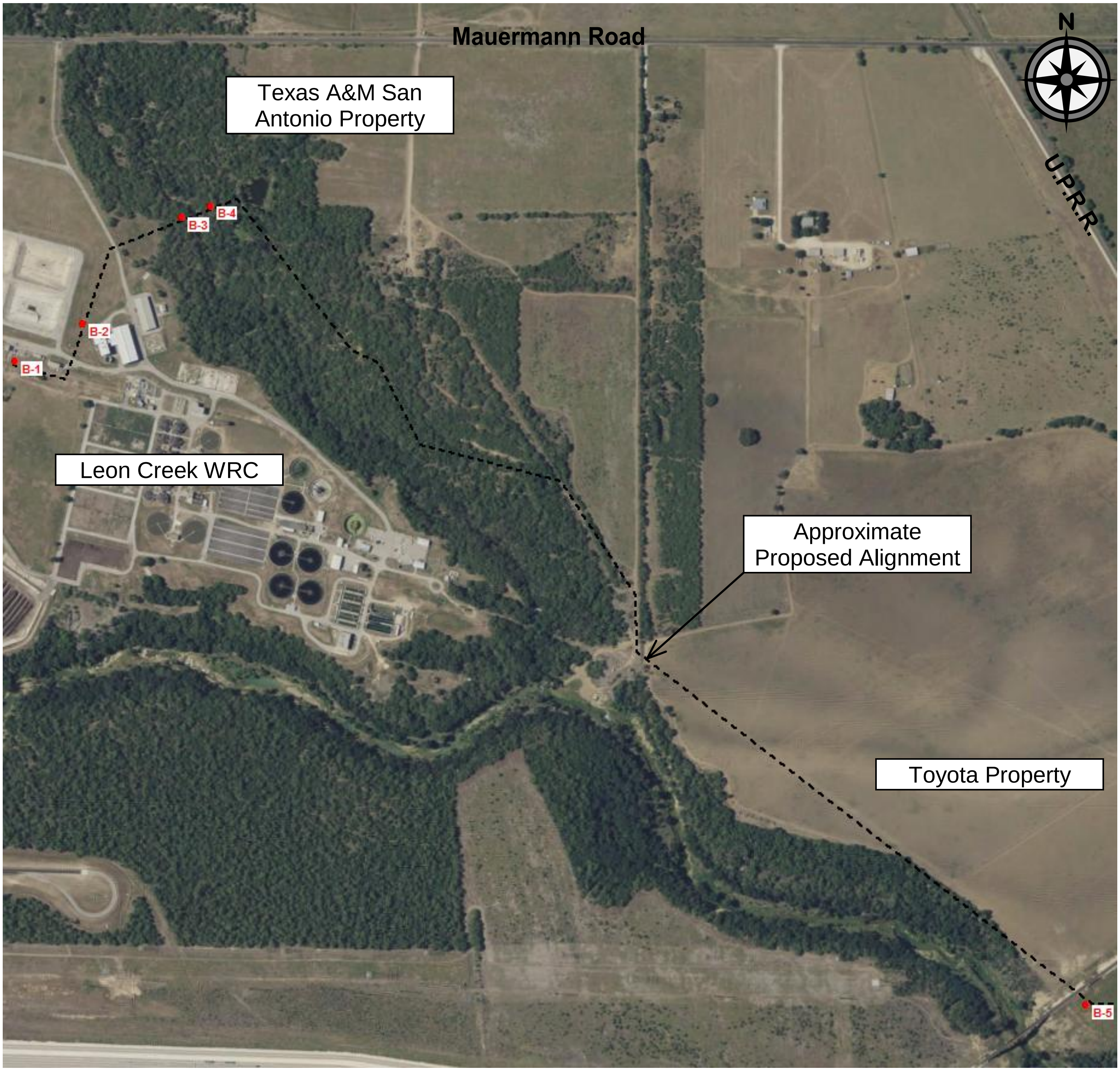
VICINITY MAP

Leon Creek Water Recycling Center (WRC)
Interconnect to Medina River Sewer Outfall
San Antonio, Bexar County, Texas

Date: June 21, 2011	Job No.: 2010-475
Drawn By: TAS	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Figure 1



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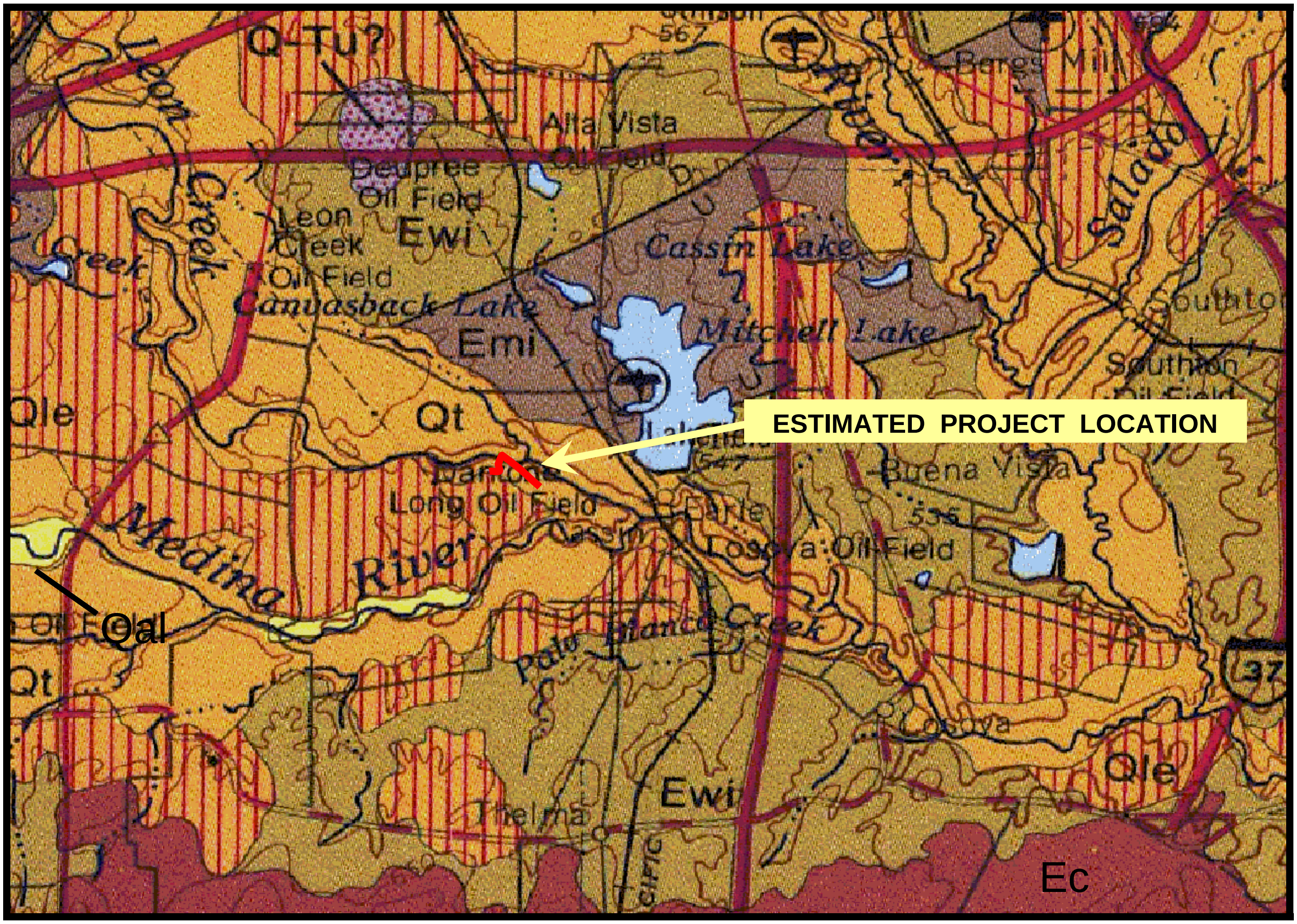
BORING LOCATION PLAN

Leon Creek Water Recycling Center (WRC)
Interconnect to Medina River Sewer Outfall
San Antonio, Bexar County, Texas

REVISIONS:				
No.:	Date:	Description:	Date: June 21, 2011	Job No.: 2010-475
			Drawn By: TAS	Checked By: AMM
			Approved By: SAH	Scale: N.T.S.

Figure 2



PORTION OF GEOLOGIC ATLAS OF TEXAS

LEGEND

Symbol	Name	Age
Qal	Active Alluvial Deposits	Quaternary Period / Holocene Epoch
Qt	Alluvial Terrace Deposits	Quaternary Period / Pleistocene Epoch
Qle	Leona Formation (Alluvium)	Quaternary Period / Pleistocene Epoch
Q-Tu	Uvalde Gravel Formation	Quaternary – Tertiary Periods
Ec	Carrizo Sand Formation	Tertiary Period / Eocene Epoch
Ewi	Wilcox Formation	Tertiary Period / Eocene Epoch
Emi	Midway Formation	Tertiary Period / Eocene Epoch

U
D
Fault Segment with Indication of Relative Movement

N

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GEOLOGIC MAP

Proposed Leon Creek Water Recycling Center (WRC)
Mauerman Road
City of San Antonio, Bexar County, TX

Date: June 27, 2011	Job No.: 2010-475
Drawn By: JLK	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Figure 3

1 of 1

Photo 1: Looking towards Boring B-1

Photo 2: Looking towards Boring B-2

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SITE PHOTOS

Leon Creek Water Recycling Center (WRC)
Interconnect to Medina River Sewer Outfall
San Antonio, Bexar County, Texas

Date: June 22, 2011	Job No.: 2010-475
Drawn By: TAS	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Appendix A

Photo 3: Looking towards the approximate area of Boring B-4

Photo 4: Looking towards the approximate area of B-4

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SITE PHOTOS

Leon Creek Water Recycling Center (WRC)
Interconnect to Medina River Sewer Outfall
San Antonio, Bexar County, Texas

Date: June 22, 2011	Job No.: 2010-475
Drawn By: TAS	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Appendix A

Photo 5: Looking at the area of Boring B-5

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SITE PHOTOS

Leon Creek Water Recycling Center (WRC)
Interconnect to Medina River Sewer Outfall
San Antonio, Bexar County, Texas

Date: June 22, 2011	Job No.: 2010-475
Drawn By: TAS	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Appendix A

APPENDIX B: SOIL BORING LOGS AND KEY TO TERMS

Boring Log No. B-1

Project: **Leon Creek WRC**
Interconnect to Media River Sewer Outfall
San Antonio, Texas

Sampling Date: 6/17/11
Elevation: 541.6 ft (By survey)

Location: See Boring Location Plan
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
LEAN CLAY (CL) trace gravel and calcareous deposits, very stiff to hard, dark brown and brown, (possible fill)		SS	7	19	45	26	23	
		SS	8				27	88
LEAN CLAY (CL) trace calcareous deposits, hard, light brown	5	SS	10	17	40	23	39	
		SS	10				32	
LEAN CLAY (CL), hard, tan	10	SS	11	15	42	27	42	91
		SS	13				30	
	15	SS	14	14	33	19	28	

Borehole terminated at 15 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Logged By: R. Arizola
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

Nomenclature Used on Boring Log

Split Spoon (SS)

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

-200 = % Passing #200 Sieve

2010-475.GPJ 9/2/11 (BORING LOG SA11-01,ARIASSA10-01.GDT,LIBRARY2010.GLB)

Boring Log No. B-2

Project: **Leon Creek WRC**
Interconnect to Media River Sewer Outfall
San Antonio, Texas

Sampling Date: 6/17/11
Elevation: ~529.5 ft (By survey)

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FILL: LEAN CLAY (CL) with sand and trace gravel, very stiff to hard, dark brown and brown		SS	8	19	46	27		24	
		SS	7					32	
FAT CLAY (CH), hard, dark brown to black	5	SS	14	19	52	33		28	95
		SS	14					34	
LEAN CLAY (CL) with calcareous deposits, very hard, light brown		T	14	19	47	28	4.5+		92
	10	T	14				4.5+		
LEAN CLAY (CL) with calcareous deposits, very stiff to hard, tan	15	T	15	15	34	19	3.75		
- gravel seam observed at 18ft.	20	SS	12					32	
		T	14				4.5		

Borehole terminated at 24 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Logged By: R. Arizola
Driller: Eagle Drilling, Inc.
Equipment: Truck-mounted drill rig

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

2010-475.GPJ 9/2/11 (BORING LOG SA11-01,ARIASSA10-01.GDT,LIBRARY2010.GLB)

Boring Log No. B-3

Project: **Leon Creek WRC**
Interconnect to Media River Sewer Outfall
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 6/17/11
Elevation: 526 ft (Estimated)
Coordinates: N29°16'52.4" W98°31'8.7"
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FAT CLAY (CH), stiff to very hard, very dark brown to dark brown		SS	14	20	52	32		9	
		SS	20					26	
	5	T	17	20	56	36	4.5+		95
		T	16				4.5+		
	10	T	14				4.5+		
LEAN CLAY (CL) with sand and gypsum crystals, very hard to hard, light brown		T	14	17	43	26	4.5+		82
	15	T	13				4.5+		
	20	SS	14					35	
- with scattered gravel lenses	25	T	18				4.25		
CLAYEY GRAVEL (GC) with sand, medium dense, tan	30	SS	17	18	52	34		24	40
FAT CLAY (CH) with gravel, very stiff to hard, tan	35	SS	7					25	
	40	SS	26	21	63	42		36	94
	45	SS	24					34	
CLAYSTONE with iron oxide seams, very hard, gray	50	SS	25					72	

Borehole terminated at 50 feet

Groundwater Data:
First encountered during drilling: 32-ft depth
After 24 hr.: At 26-ft depth (28.5-ft open borehole depth)

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

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

Boring Log No. B-4

Project: **Leon Creek WRC**
Interconnect to Media River Sewer Outfall
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 6/17/11
Elevation: 537 ft (Estimated)
Coordinates: N29°16'52.98" W98°31'6.88"
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
GRAVELLY FAT CLAY (CH) with sand, stiff to very stiff, brown		SS	7	22	50	28		13	59
		SS	11					23	
FAT CLAY (CH) with iron oxide seams, very stiff to very hard, tan and gray	5	SS	15	17	57	40		20	
		T	14				4.5+		
	10	SS	15					29	
		SS	16	20	61	41		30	97
	15	T	13				4.5+		
	20	SS	13					39	
	25	SS	14					58	
- gravel seam observed at 26ft.									
	30	SS	21					51	
	35	T	14				4.5+		
- gypsum crystals observed	40	SS	23	20	50	30		77	96
CLAYSTONE with cemented seams, very hard, gray	45	SS	16					50/1"	
		SS	16					25/0"	
Borehole terminated at 48.5 feet									

Groundwater Data:
During drilling: Not encountered
After 24 hr.: At 39.4-ft depth (44.2-ft open borehole depth)

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

Nomenclature Used on Boring Log

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

2010-475.GPJ 9/2/11 (BORING LOG SA11-01.ARIASSA10-01.GDT,LIBRARY2010.GLB)

Boring Log No. B-5

Project: **Leon Creek WRC**
Interconnect to Media River Sewer Outfall
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 6/17/11
Elevation: 521 ft (Estimated)
Coordinates: N29°16'9.8" W98°30'13.2"
Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200	DD	Uc
LEAN CLAY (CL) with sand, stiff to hard, brown		SS	10	17	39	22		14	82		
		SS	11					14			
	5	SS	13					15			
		SS	15					12			
- hard	10	T	15	17	39	22	4.5+		88	115	6.52
		T	16				4.5+				
LEAN CLAY (CL), hard, tan	15	T	17	17	44	27	3.75			110	4.08
	20	T	18				3.25				
	25	T	17				3.25				

Borehole terminated at 25 feet

Groundwater Data:
During drilling: Not encountered

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

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
DD = Dry Density (pcf)
Uc = Compressive Strength (tsf)

KEY TO CLASSIFICATION 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 is 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

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 by taking material from the auger as it was advanced (ASTM D 1452). 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.

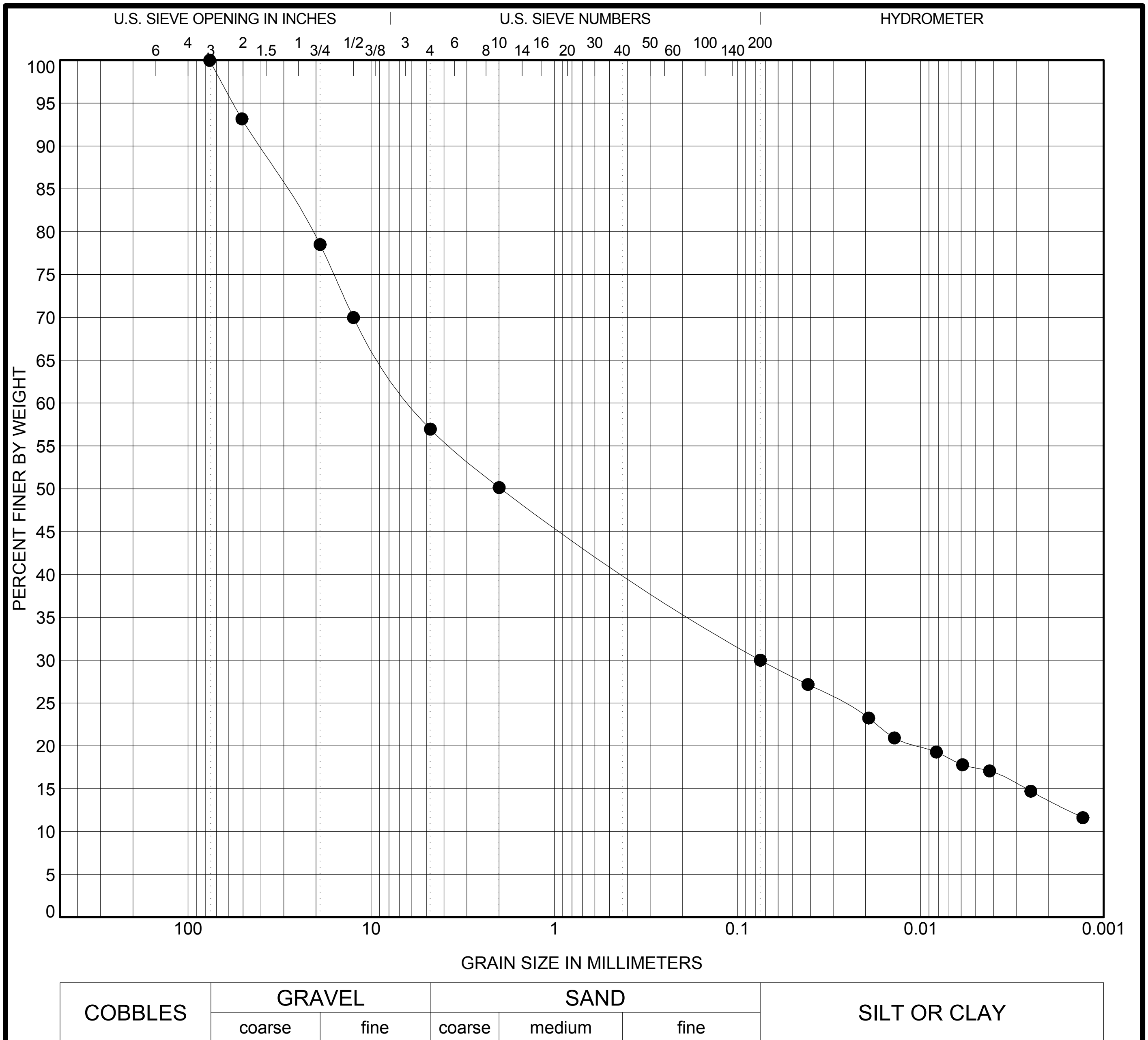
SPT N-values and blow counts for those intervals where the sampler could not be advanced for the required 18-inch penetration are shown on the soil boring log. If the test was terminated during the 6-inch seating interval or after 10 hammer blows were applied used and no advancement of the sampler was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

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

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

The laboratory results are reported on the soil boring log.

APPENDIX D: GRAIN SIZE DISTRIBUTION



Boring	Elev	Depth	Classification	LL	PL	PI	Cc	Cu
● **A**		**0.0**						

Boring	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **A**	**0.0**	**76.2**	**5.953**	**0.075**		**42.8**	**27.0**	**16.3**	**13.7**

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

2010-475.GPJ 8/31/11 (GRAIN SIZE ARIAS.US_LAB.GDT,LIBRARY2010.GLB)

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

GRAIN SIZE DISTRIBUTION

Project: Leon Creek WRC

Location: See Boring Location Plan

Job No.: 2010-475

Arias & Associates, Inc.

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.

ASFE
THE BEST PEOPLE ON EARTH

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