Geotechnical Engineering Study

SAWS Cibolo Elevated Storage Tank Project
San Antonio, Texas

SAWS Job No. 02-6015
Arias Job No. 2011-585

ARIAS & ASSOCIATES
Geotechnical • Environmental • Testing

Prepared For
San Antonio Water System

November 21, 2011

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Geotechnical • Environmental • Testing

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Arias Job No. 2011-585

Mr. Vicente Garza, P.E.
Production and Transmission Engineering
San Antonio Water System (SAWS)
2800 U.S. Highway 281 North
San Antonio, Texas 78212

RE: Geotechnical Engineering Study
SAWS Cibolo Elevated Storage Tank Project
TP C Parkway and Bulverde Green
SAWS Job No. 02-6015
San Antonio, Texas

Dear Mr. Garza:

Arias & Associates, Inc. (Arias) is pleased to submit the results of a Geotechnical Engineering Study for the proposed SAWS Cibolo Elevated Storage Tank Project 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 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, pavement installation, 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
AUREA MUÑIZ MARTINEZ
104705
LICENSED
PROFESSIONAL ENGINEER
11/21/11

Aurea M. Martinez, P.E.
Geotechnical Project Engineer

Dexter Bacon, P.E.
Senior Vice President

1295 Thompson Rd
Eagle Pass, Texas 78852
(830) 757-8891
(830) 757-8899 Fax

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

5233 IH 37, Suite B-12
Corpus Christi, Texas 78408
(361) 288-2670
(361) 288-4672 Fax

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INTRODUCTION

The results of a Geotechnical Engineering Study for the proposed SAWS Cibolo Elevated Storage Tank Project in San Antonio, Texas are presented in this report. This project was authorized on October 7, 2011, by Mr. Jim Pedraza, P.E. of SAWS by means of the 2008 Geotechnical Engineering Design Services Contract (Production, Recycle and Treatment Engineering) between SAWS and Arias & Associates, Inc. (Arias). Our scope of work was performed in general accordance with the services outlined in Arias Proposal No. 2011-585, dated Sepember 27, 2011 and revised Octover 5, 2011.

SCOPE OF SERVICES

The purpose of this geotechnical engineering study was to conduct a subsurface exploration and perform laboratory testing to establish geotechnical engineering properties of the subsurface soil and groundwater conditions present at the site. This information was used to develop geotechnical engineering criteria for use by design engineers in preparing the foundation designs for the proposed elevated storage tank. The criteria provided in this report can also be used to assist in the design of the proposed site pavements. Environmental studies or analyses of slopes and/or retaining structures were beyond our authorized scope of services for this project.

PROJECT DESCRIPTION AND SITE DESCRIPTION

The planned project will consist of the construction of a new 2.5 million gallon (MG) elevated water tank northeast of TPC Parkway and Bulverde Green in San Antonio Texas. We understand that the new elevated water tank will be a composite structure with an approximate 120-foot diameter steel water tank atop a 60-foot diameter reinforced concrete pedestal. The planned development will also include the construction of a new access road and 2,050 linear foot water main. The 24-inch water main will extend from the new tank structure to the existing water system along TPC Parkway. It is anticipated that the new water main will be placed with about 5 to 7 feet of cover. The access road will be constructed concurrently with the adjacent residential development. Preliminary pavement sections are provided in the report based on the assumption that fill will be placed beneath the planned roadway. A Vicinity Map depicting the approximate site locations is included as Figure 1 in Appendix A of this report.

Based on our correspondence with the project design team, we understand that the proposed tank structure will be supported on either: (1) a shallow concrete ringwall foundation with non-structural slab-on-grade (constructed at the ground surface above the ringwall foundation), or (2) a deep drilled pier foundation system. It should be noted that final grading plans were not available for our review in preparation of our recommendations. Once final grading plans become available, we should be notified in writing to determine if changes to our recommendations are needed.

Geographically, the project area is situated in the Long Creek watershed in an area typified by low well-rounded ridges. Locally, the existing ground surface within the project area has a gentle eastward descending slope. At the time of our field exploration conducted on October 13, 2011 and October 14, 2011, the project site was observed in an undeveloped condition and may have been used in the past for agricultural purposes. The existing vegetation consisted of a dense cover of juniper and oak trees, wild grass and weeds. Onsite utilities are present. Site photographs are included in Appendix A of this report.

SOIL BORINGS AND LABORATORY TESTING

Six (6) soil borings were drilled at the approximate locations shown on the Boring Location Plan included as Figure 2 in Appendix A. A description of the boring locations and corresponding structures, boring depths and coordinates are summarized in Table 1. The boring depths are referenced below the existing ground surface between October 13, 2011 and October 14, 2011. Drilling was performed in general accordance with ASTM D1586 procedures for Split Spoon 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 grouted with non-shrink grout.

Table 1: Approximate Boring Locations, Depths and Coordinates

Boring No.	Proposed Structure	Depth Drilled	Survey Point	Northing	Easting
B-1	Elevated Storage Tank	25’	70000	13793478.0683	2154460.4632
B-2	Elevated Storage Tank	23.5’	70001	13793436.5759	2154438.1391
B-3	Elevated Storage Tank	27’	70002	13793438.6024	2154485.7576
B-4	24-inch water main	13.5’	70003	13792686.1690	2154664.5680
B-5	24-inch water main	15’	70004	13791796.2056	2155053.3379
B-6	Access Drive	4.5’	Note 3		

Notes:

1. Depth is measured from existing ground surface at the time of the geotechnical study (October 2011).
2. Topographic survey data provided by SAWS (Pape-Dawson Engineers, Inc., Design Staking Sheet, dated November 4, 2011).
3. Topographic survey data was not provided for Boring B-6.

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 with zipper seal. The samples were then placed into wax-coated cardboard sample boxes designed for transporting soil specimens to the laboratory.

Soil classifications and borehole logging were conducted during the exploration by one of our Professional Geologists 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 at the project site are discussed in the following sections. The subsurface conditions presented 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 Edwards Limestone Group of the lower Cretaceous Period of the Geological Time Scale. Locally, the materials encountered in the borings consist of approximately 1 to 3 feet of natural surface soils overlying limestone bedrock; however, at Boring B-3 the limestone was encountered at a depth of approximately 7 feet and at the ground surface at Boring B-5. The surface soils consisted of dark brown clay (CH) or clayey gravel (GC) in a stiff to very hard and medium dense to very dense condition. The limestone was found to contain some red clay filled fractures.

Generalized Site Stratigraphy and Engineering Properties

The general stratigraphic conditions at the boring locations are provided in Table 2 below. *The presence and thickness of the various subsurface materials can be expected to vary away from and between the exploration locations.* The descriptions conform to the Unified Soils Classification System.

Table 2: Generalized Soil Conditions

Stratum	Depth, ft	Material Type	PI range / PI Avg.	No. 200 range / No. 200 Avg.	N Range / N Avg.
I	0 to (0.5-2)	FAT CLAY (CH) and Clayey GRAVEL (GC) with limestone fragments, dark brown, stiff to very hard and medium dense to very dense	29-43	18-75	10-50/1”
			34	47	>50
II	(0.5-1) to (3-7)	Clayey GRAVEL (GC) with sand limestone fragments, tan and reddish brown, very dense	8-10	18	--
			9	[illegible]	>100
III	(0-7) to 27	LIMESTONE, cemented, light tan, very hard	--	--	>100

Where: Depth -Depth from existing ground surface during geotechnical investigation, feet
PI -Plasticity Index, %
No. 200 -Percent passing #200 sieve, %
N -Standard Penetration Test (SPT) value, blows per foot

Depth to Limestone

Based on the results of our field exploration, formational Limestone bedrock was encountered at relatively shallow depths. The Limestone bedrock stratum was observed at the existing ground surface to a depth of about 7 feet below the existing ground surface at the time of the field exploration (October 2011). A detailed evaluation of the excavatibility of the Limestone bedrock was beyond our authorized scope of services. However, based on our experience in this area, we anticipate that drilling/excavating in these areas will likely encounter conditions requiring heavy-duty rock excavating equipment. Heavy-duty excavation equipment is defined as equipment capable of cutting/excavating very hard clay, clay marl, marlstone, claystone and limestone. *The contractor should be prepared for such conditions.*

With regard to the formational material, it is important to note that solution cavities or voids, and clay seams may exist in the limestone formational material in this area. While voids were not observed within the borings, their potential presence is an important consideration with regard to the foundation type chosen for the proposed project.

Groundwater

A dry soil sampling method was used to obtain the soil samples. Groundwater was not observed within the borings during or after sampling activities between October 13, 2011 and October 14, 2011. 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.

Groundwater levels will often change significantly over time due to seasonal conditions, rainfall, drought, or temperature effects and should be verified immediately prior to construction. 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 at strata interfaces, particularly at clay/gravel or soil/rock interfaces.

The means and methods for dewatering the site are solely the responsibility of the contractor. We should note that subsurface soil and groundwater conditions can vary away from the boring locations.

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 27-foot depth.

Subsurface materials having similar consistency were extrapolated to be present between 27 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 sites as Site Class C.

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 78261 and Site Class C. Seismic design parameters for the site are summarized in the following table.

Table 3: Seismic Design Parameters

Site Classification	F_a	F_v	S_s	S_1
C	1.2	1.7	0.096g	0.030g

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

ENGINEERING EVALUATION FOR SITE IMPROVEMENTS

The foundation systems being considered for the proposed tank structure should be designed with an appropriate factor of safety to reduce the possibility of soil failure when subjected to axial and lateral load conditions. The data obtained from the soil borings suggests that the subsurface materials are generally competent to support the proposed construction. The potential for foundation and pavement movements from soil heaving/shrinking will need to be considered in the foundation and pavement designs. Furthermore, site subgrade modifications and preventative design measures should be implemented to aid in reducing the impacts of potential expansive soil-related movement to within the allowable and operational limits of the proposed improvements.

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.

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 about ***1-inch*** 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. However, soil movements in the field depend on the initial moisture contents and the actual changes over time. Thus, the PVR could be more than the TXDOT estimated value due to extended droughts, flooding, “perched” groundwater infiltration, poor surface drainage, the presence of trees or other large vegetation, and/or leaking irrigation lines or plumbing.

Both shallow and deep foundation types are utilized in this area. Deep drilled piers are suited to structures with moderate to heavy loading conditions, or for more movement–sensitive structures. The piers, when properly designed, can reduce foundation movement of the superstructure. Grade beams or pier/pile caps, isolated from the soil, typically span between the piers to allow for shrink/swell movements of the subgrade soils to occur without applying load to the pier/pile cap and structure. The deep foundation option is used when excellent operational and aesthetic performance is expected from the structure in terms of reducing the chances for differential movement in the foundation and structure. Each approach has its advantages and disadvantages in terms of cost and overall performance. Structures founded on expansive soils can be expected to experience some distress.

Recommended Foundation Types

Based on our discussions with SAWS, we understand that both shallow and deep foundation systems are currently being considered for the proposed Cibolo Composite Elevated Tank. Based on both the proposed structure and on the soil conditions encountered at this site, the proposed ringwall foundation will need to be designed to provide adequate resistance against potential expansive soil heave or potential vertical rise (PVR), potential settlement, and overturning moments. The proposed elevated water tank can be supported on either a ringwall foundation or a pile cap and drilled pier foundation provided that the structure and foundation can be designed for the estimated soil movements presented in this report, and provided that the recommendations included herein are followed.

Option I for Tank Structure – Ringwall Foundation

The proposed tank can be supported on a ringwall foundation provided it is designed specifically for the soil conditions encountered at this site. The ringwall foundation should be founded at least 5 feet below existing grade within competent limestone bedrock (*i.e.,* neglect seams of clay or shattered limestone). We should note that the bearing depth may need to be deeper to resist uplift and overturning moments induced by wind loading. The allowable bearing pressure for the tank is 10,000 psf based on total load conditions, and includes a factor of safety of 3.0 against bearing failure. This bearing value assumes that the ringwall footing bears uniformly on competent limestone bedrock.

Lateral loads may be resisted by the friction between the foundation bottom and the supporting subgrade. An allowable friction coefficient of 0.5 between the foundation and supporting subgrade may be used.

Overturning moments and uplift loading can be resisted by the weight of the foundation, weight of the structure, and any soil overlying the ringwall. A soil unit weight of 125 pounds per cubic foot (pcf) may be assumed for on-site soils or select fill that are placed above the ringwall and compacted as recommended in this report. We recommend that backfill within the annular space of the ringwall be conducted using select fill as recommended in the following "Non-structural Slab-on-Grade" section. Backfill above the footing on the outside of the footing's stem wall should consist of the excavation Strata I or II soils. The onsite soil backfill should be placed in 8-inch maximum loose lifts that are moisture conditioned to between -1 and +3 percentage points of optimum moisture content and compacted to at least 95 percent of ASTM D698.

Total settlement of the tank foundation is anticipated to be about 1-inch. Differential settlement from one side of the tank to the other is anticipated to be about ½ to ¾-inch. Based on the recommended minimum ringwall bearing depth, the potential expansive soil-related movements or PVR associated with seasonal moisture change should be negligible when the tank is at capacity. However, the PVR could be about 1 inch for an empty tank condition. If the PVR values are considered excessive, we can provide recommendations for over-excavating a portion of the expansive clay soils from beneath the ringwall and replacing these soils with select fill material.

The ringwall footing excavation should have a firm bottom and be free from excessive slough prior to concrete or reinforcing steel placement. Based on the results of the field exploration and the recommended minimum ringwall bearing depth, it appears the ringwall footing will bear on limestone bedrock. Under no circumstances should water be allowed to adversely affect the quality of the bearing surface. If bearing soils are exposed to drying or wetting cycles that result in either desiccated or softened soils, the unsuitable soil must be re-conditioned or removed as appropriate and replaced with compacted select fill before concrete is placed. The foundation bearing soils should be observed by the geotechnical engineer or his representative prior concreting.

Where utility trenches are to be located adjacent to the ringwall foundation, the bottom of the footing should be located below an imaginary 1:1 (horizontal:vertical) plane projected upward from the nearest bottom edge of the utility trench. The footing excavations should be observed by a representative of Arias prior to placement of reinforcing steel or concrete to evaluate the exposed soil conditions.

Pilot Holes Specifications for Ringwall Foundation

As previously noted, there is the potential for the existence of solution cavities within the limestone formation at this site. This creates a concern with regard to the potential that the ringwall foundation could be installed immediately above a void without detection of the void. For the ringwall footing, potential voids under the footing can be compensated for during construction through a pilot hole program where holes are drilled to evaluate the presence of voids within footing location to a level beneath the bearing surface.

The pilot hole program should be performed at increments of 20 linear feet minimum along the footing to a depth of 10 feet beneath the footing bearing surface. If a void is encountered during pilot hole drilling, we recommend that the pilot be advanced at least 15 feet into competent rock beyond the void. Pilot holes should be grouted using a 3,000 psi lean concrete. The pilot hole program should be monitored by a representative of Arias. Proper placement of pier reinforcing steel, and concrete observations and tests should also be conducted

Non-Structural Slab-on-Grade

Non-structural slab-on-grade for the project should be a minimum thickness of 5 inches. Slab thickness and reinforcing should be determined by the project structural engineer. A Subgrade Modulus of 125 psi/in may be used for the design. Special care should be taken to insure that reinforcement is placed at the slab mid-height. The floor slab should be separated from the footings, structural walls, and utilities, and provisions made to allow for settlement or swelling movements at these interfaces. If this is not possible from a structural design standpoint, it is recommended that the slab connection to footings be reinforced such there will be resistance to potential differential movement.

Backfill within the annular space of the ringwall foundation below the non-structural slab-on-grade should consist of select fill meeting the following criteria: (1) be free of organic and deleterious material, (2) have a plasticity index (PI) between 7 and 20, and (3) not contain particles exceeding 3 inches in maximum dimension. The select fill should be placed in 8-inch maximum loose lifts that are moisture conditioned to between -2 and +3 percentage points of optimum moisture content and compacted to at least 95 percent of ASTM D698.

Consideration can be given to using a TXDOT Item 247, Type A, Grade 1or 2 crushed limestone flexible base material immediately beneath the proposed slab-on-grade to help create a more “all-weather” working surface.

Measures to Reduce Soil Moisture Change

The following design measures are recommended to help reduce potential soil shrink/swell foundation movements.

- The ground surface adjacent to the ringwall foundation perimeter should be graded and maintained at a minimum of 5 percent downward slope away from the foundation for a horizontal distance of at least 10 feet to cause positive surface flow or drainage away from the structure perimeter.
- Hose bibs, sprinkler heads, overflow weirs, and other external water connections should be preferably eliminated if possible, or alternatively, placed well away from the foundation perimeter such that surface leakage cannot readily infiltrate into the subsurface or compacted fills placed under the proposed foundations and slabs.
- Trees should not be planted closer to structures than a distance approximately equal to their estimated mature height. Shrubs or other plants, which require large quantities of water, should not be planted close to structures.
- Utility bedding should not include gravel within 4 feet of the perimeter of the foundation. Compacted clay or flowable fill trench backfill should be used in lieu of permeable bedding materials between 2 feet inside the building to a distance of 4 feet beyond the

exterior of the building edge to reduce the potential for water to infiltrate within utility bedding and backfill material.

- If possible, use paved areas around the structure. These areas help to reduce variations in soil water content.
- Flower bed curbing and planter boxes should be drained or water tight to prevent trapped water near the building perimeter.
- Site work excavations should be protected and backfilled without delay in order to minimize changes in the natural moisture regime.

Option II for Tank Structure – Drilled Pier Foundation

Based upon the subsurface conditions observed at Borings B-1 to B-3 and the results of the laboratory testing performed on the soil samples, straight-shaft drilled pier foundations be used to support the proposed tank structure. Applicable geotechnical foundation design parameters are discussed below for this foundation system. Recommendations for evaluation of axial capacity and lateral capacity are presented below. 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.

The piers can be sized using a net allowable bearing pressure of **60 ksf** based on total load conditions. The recommended bearing pressures includes a factor of safety of 3.0. The pier diameter should be 18 inches or larger. Each pier should be embedded a minimum of **20 feet** into sound formational materials (*i.e.,* competent limestone, <u>not</u> shattered limestone). In addition to end bearing, an allowable skin friction value of **4 ksf** can be used for that portion of the pier in contact with sound limestone (FS=2.0). *Actual pier lengths will vary depending on the design pier loads and the location of sound bedrock at the actual pier location.*

Zones of highly weathered rock, voids, and shattered limestone layers should not be considered as part of the pier embedment length. For example, once the limestone formation is encountered and a two-foot thick shattered zone is then encountered within the limestone formation during drilling, the pier should be extended an additional 2 feet into the underlying sound limestone bedrock formation.

To assess the general condition of the limestone, we recommend that prior to pier installation that a pilot hole program be performed at all of the pier locations to a 10 foot depth beyond the bearing depth. The pilot hole specifications for this site are discussed in the subsequent section below.

Uplift Force due to Expansive Soils and Resistance

The potential soil uplift force, in kips, along the shaft of the pier can be estimated as being negligible when the piers are completely founded in the limestone bedrock.

Estimated Settlement

Post construction settlements of properly constructed drilled piers should be approximately ½ inch or less, assuming proper construction practice. The settlement response of the piers will be more dependent upon the quality of construction than upon the response of the limestone bedrock formation to foundation loads.

Installation Monitoring

Arias should continuously monitor pier installation activity and verify that each pier is installed at the proper depth and that the bottom of the piers are free and clean of loose and/or soft material.

Lateral Pile Analyses

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 the following table, Arias presents geotechnical input parameters for the encountered soils.

Table 4: Drilled Pier Geotechnical Input Parameters for LPILE Analyses for Tank (Borings B-1 to B-3 only)

Depth (ft)	Material	γ_e	C_u	ϕ	K (cyclic loading)	e_{50}
0 to 5	FILL: FAT CLAY (CH), Clayey GRAVEL or LIMESTONE	Neglect Contribution				
5 to 7	Clayey GRAVEL	125	0	32	225	--
	LIMESTONE	125	9,000	0	800	0.004
7 to 27	LIMESTONE	125	9,000	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 below 27 feet based on boring data

Pilot Holes Specifications for Drilled Pier Foundations

As previously noted, there is the potential for the existence of solution cavities within the limestone formation at this site. This creates a concern with regard to the potential that a drilled pier type foundation could be installed immediately above a void without detection of the void. For the drilled piers designed using an end bearing component, potential voids under piers can be compensated for during construction through a pilot hole program where holes are drilled to evaluate the presence of voids at each pier location to a level beneath the bearing surface. Alternatively, pilot holes can be eliminated if the drilled piers are designed as frictional units using skin friction only without end bearing. If desired, we can provide recommendations for piers designed for skin friction only.

The pilothole program should incorporate small diameter holes that are performed at each pier location to a depth of 10 feet beneath the bearing surface. If a void is encountered during pilot hole drilling, we recommend that the pilot be advanced at least 15 feet into competent rock beyond the void. Pilot holes should be grouted using a 3,000 psi lean concrete. The pilot hole program should be monitored by a representative of Arias. Proper placement of pier reinforcing steel, and concrete observations and tests should also be conducted

PAVEMENT RECOMMENDATIONS

The proposed site development will include the construction of a new access drive. The access road will be constructed concurrently with the adjacent residential development. Pavement sections are provided in the report based on the assumption that compacted onsite fill (*i.e.,* CLAY (CL-CH)) will be placed beneath the planned roadway. No specific design traffic information was received for this project. Therefore, the design parameters and assumptions included in Table 5 were used in our analysis. The pavement recommendations were prepared in accordance with the 1993 AASHTO Guide for the Design of Pavement Structures for asphalt and the ACI Design Guide 330R for Design and Construction of Concrete Parking Lots for concrete.

Table 5: Pavement Design Assumptions

Traffic Load for Light Duty Pavement	15,000 equivalent single axle loads (ESALs)
Traffic Load for Medium Duty Pavement	50,000 equivalent single axle loads (ESALs)
Average Daily Truck Traffic vehicle with at least 6 Wheels	One (1)
Concrete Compressive Strength	4,000 psi
Raw Subgrade California Bearing Ratio (CBR)	2 for moderate to high plasticity compacted clay (CL-CH) FILL
Raw Subgrade Modulus of Subgrade Reaction, k in pci	75 for moderate to high plasticity compacted clay (CL-CH) FILL

Accumulation of water beneath the asphaltic surface course can cause progressive and rapid deterioration of the pavement section. Similarly, pavement surfaces should be well drained to eliminate ponding with a two-percent minimum slope, as possible.

Options for section thickness for flexible and rigid pavements are provided in Table 6. Note that the truck lane traffic sections correspond to only one heavy-duty truck per day. If more heavy-duty truck traffic is anticipated, we should be contacted to provide additional recommendations. A truck traffic section is recommended for use at loading docks, entrances, driveways, dumpsters pads and channeled traffic areas. Areas subjected to truck traffic stopping, starting, loading, unloading or turning should <u>not</u> utilize asphalt pavement. For these areas, we recommend rigid concrete pavements.

Table 6: Recommended Pavement Sections

Layer	Material	Flexible Asphaltic Concrete				Rigid Concrete			
		Light Duty		Medium Duty		Light Duty		Medium Duty	
Surface	HMAC/PCC	2”	2”	2½”	2½”	5”	5½”	5½”	6”
Base	Flexible Base	7”	10”	9”	12”	--	--	--	--
Subgrade	Lime Treatment	6”	--	6”	--	6”	--	6”	--
	Moisture Conditioned	--	6”	--	6”	--	6”	--	6”

Notes:

1. Recommended pavement sections are based on the assumption that the subgrade will consist of moderate to high plasticity clay (CL-CH) fill. The Geotechnical Engineer’s representative should verify the final subgrade conditions at the time of construction once the adjacent site grading is complete.
2. Light duty areas include parking and drive lanes that are subjected to passenger vehicle traffic only.
3. Medium duty areas include entrance aprons and drives into the site, single access route drive lanes to parking areas, and areas where paving will be subjected to truck traffic.
4. Heavy duty areas include areas subjected to “truck traffic” including 18-wheel tractor trailers, trash collection vehicles, dumpster pads including loading and unloading areas, and areas where truck turning and maneuvering may occur. **Seven (7)-inch thick concrete pavement is recommended for heavy duty areas.**

Table 7: Additional Concrete Pavement Recommendations

Pavement Thickness	Dowel Diameter	Total Dowel Length	Maximum Control Joint Spacing
5-inch	5/8-inch	12 inches	12.5 feet
5½-inch	¾-inch	14 inches	12.5 feet
6-inch	¾-inch	14 inches	15 feet
7-inch	7/8-inch	14 inches	15 feet

The concrete pavement should include as a **minimum** the following:

1. Reinforcing Steel - #4 @ 16-inch each way placed D/3 from top of slab
2. Construction Joint Dowels – Spaced at 12-inch O.C. lubricated both sides @ mid depth
3. Control Joint Depth – D/3 from top

4. Min. 28 day compressive strength – 4,000 psi
5. Maximum Slump of 5-inches
6. Proper curing practices of concrete in accordance with ACI and PCA recommendations

Traffic can be allowed on the new concrete once required compressive strength is obtained but not sooner than seven (7) days from the time of placement. Mixture design using high early strength concrete is allowed. In general concrete, should be designed and placed in accordance with ACI 330R-92. Hot weather concreting should be performed in accordance with ACI 305R-91 and Cold Weather Concreting should be performed in accordance with ACI 306R-88.

Rigid Concrete Pavement Joints

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

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

The use of sawed and sealed joints should be designed in accordance with current Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. Research has proven that joint design and layout can have a significant effect on the overall performance of concrete pavement.

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

Pavement Performance

Successful long-term performance will depend in part on the implementation of good drainage, proper subgrade preparation, and good construction practices. Accumulation of water can cause: (1) weakening of the subgrade, (2) induce soil subgrade heave, and (3) weakening of the bonds within the pavement section materials. These conditions can each lead to progressive and rapid deterioration of the pavement section. Similarly, pavement surfaces should be well drained to eliminate ponding with a two-percent minimum slope, as possible.

PAVEMENT CONSTRUCTION

Site Preparation for Pavement Construction

Stripping should be performed as needed to remove existing organic materials, loose soils, vegetation, roots, and stumps. Additional excavation may be required due to encountering deleterious materials such as concrete, organics, debris, soft materials, loose fill, etc.

Lime Stabilized Subgrade

The upper 6 inches of high plasticity clay subgrade may be stabilized with lime by dry weight in accordance with City of San Antonio Standard Specifications for Construction, Item 108, “Lime Treated Subgrade”. The quantity of lime required should be determined after the site is stripped of the loose soil and the subgrade soils are exposed. **We anticipate that approximately 5 to 8 percent lime will be required depending upon the material encountered. However, the quantity of lime should be sufficient to: (1) result in a pH of at least 12.4 when tested in accordance with ASTM C977, Appendix XI; and (2) reduce the PI of the clay subgrade to 20 or less.** The target lime content and optimum moisture content should be determined in accordance with TxDOT test procedure TEX-120-E.

For the purposes of lime stabilization, the dry weight of the high plasticity clay soils may be taken as 105 pounds per cubic foot (pcf). The amount of lime required may vary over the site. The limed soil should be compacted to at least 95 percent of the maximum dry density as evaluated by TEX-114-E at moisture contents ranging from optimum to plus four (+4) percentage points of optimum moisture content. As a guideline, at least one in-place density test should be performed for every 100 linear feet of each lift, with a minimum of 3 tests per lift. Any areas not meeting the required compaction should be recompacted and retested until compliance is met.

Fill Requirements

The general fill used to increase sections of the roadway grade should consist of onsite materials meeting or exceeding the existing subgrade CBR at each particular location. The general fill should be placed in accordance with City of San Antonio Standard Specifications for Construction, Item 108, “Embankment”. The compaction should be performed in accordance with the “Density Control” method. Onsite material may be used provided it is placed in maximum 8” loose lifts and compacted to at least 95 percent of the maximum dry density as evaluated by TEX-114-E to within optimum to plus four (+4) percent of optimum moisture. This fill should not have any organics or deleterious materials. When fill material includes rock, the maximum rock size acceptable shall be 4-inches. No large rocks (>4 inches) shall be allowed to nest, and all voids must be carefully filled with small stones and fine-grained soils, and be properly compacted.

The CBR of all fill materials used should be equal to or exceed the existing subgrade CBR at each particular location. The suitability of all fill materials should be approved by the Geotechnical Engineer. Conformance testing during construction to assure quality will be necessary for this process. If fill is required to raise paving grades, the above compaction criteria should be utilized with the fill placed in maximum 8" thick loose lifts. It should be noted that if fill materials with lower CBR values are placed, then a higher Structural Number and a thicker pavement section would be necessary.

Flexible Base Course

The base material should comply with City of San Antonio Standard Specifications for Construction, Item 200, "Flexible Base", Type A, Grade 1 or 2. The compaction should be performed in accordance with the "Density Control" method. The flexible base should be compacted in maximum 8-inch loose lifts to at least 95 percent of the maximum dry density as evaluated by TEX-113-E within plus or minus 3 percent of optimum moisture content. As a guideline, at least one in-place density test should be performed for every 100 linear feet of each lift, with a minimum of 3 tests per lift. Any areas not meeting the required compaction should be recompacted and retested until compliance is met.

Asphaltic Base Course

The asphalt should comply with City of San Antonio Standard Specifications for Construction, Item 205, "Hot Mix Asphaltic Concrete Pavement", Type B, Base Course. As a guideline, at least one in-place density test should be performed for every 100 linear feet of each lift, with a minimum of 3 tests per lift.

Asphaltic Concrete Surface Course

The asphalt should comply with City of San Antonio Standard Specifications for Construction, Item 205, "Hot Mix Asphaltic Concrete Pavement", Type C or D, Surface Course. Our design thickness may require the surface course to be placed in multiple compacted lifts. Compaction tests should be performed during construction in accordance with the project documents. On a daily basis, the asphaltic concrete should be tested for oil content, gradation, and stability to verify compliance with the job mix formula, which should be submitted by the manufacturer for approval.

Curb and Gutters

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 properly designed and constructed curb and gutters be used 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 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 as the roadways are being installed. This consideration should be included in the project specifications.

Maintenance Considerations

The pavements will be subject to expansive soil-related movements on the order of the estimated site PVR previously noted. These movements could lead to pavement distress and some cracking should be expected. 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 that 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 successfully used by the Bexar County, City of San Antonio, TxDOT, and other government agencies to aid in prolonging pavement life by reducing accelerated wear and deterioration. Failure to provide routine crack-sealing will increase the potential for pavement failures and potholes to develop.

STRUCTURES BELOW GRADE

Lateral Earth Pressures – Trench Shoring

Lateral earth pressure for design of trench shoring can utilize the following soil design parameters shown in Table 8 for short term conditions:

Table 8: Trench Shoring Parameters for Short Term Conditions

Stratum	Description	γ_e	C	φ	k_a
I	Dark Brown, FAT CLAY (CH)	125	750	0	0.55
I	Dark Brown Clayey GRAVEL (GC)	125	0	28	0.36
II	Reddish Brown and Tan, Clayey GRAVEL (GC)	125	0	28	0.36
III	Light Tan, LIMESTONE	125	9,000	0	0.55

where: **γ_e** = effective soil unit weight, pcf
C = undrained soil shear strength, psf
φ = angle of internal friction, deg.
k_a = coefficient of active earth pressure

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, soil stockpiles and hydrostatic pressures should be added to this value as required.

Excavations

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

Table 9: OSHA Soil Classifications

Stratum	Description	OSHA Classification
I	Dark Brown, FAT CLAY (CH)	C
I	Dark Brown Clayey GRAVEL (GC)	C
II	Reddish Brown and Tan, Clayey GRAVEL (GC)	C
III	Light Tan, LIMESTONE	A

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

For excavations less than 20 feet deep, the maximum allowable slope for Type "C" soils is 1.5H:1V (34°), 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 temporary slopes. Permanent slopes at this site should be sloped no steeper than 4H:1V and flatter slopes may be required in gravelly/sandy areas. Flatter slopes may also be desired for mowing purposes.

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

The soils to be penetrated by excavations may vary significantly across the site. Our preliminary soil classification is based solely on the materials encountered in widely spaced exploratory test borings. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, we recommend that Arias be contacted immediately to evaluate the conditions encountered.

Trenches less than 5 feet deep are generally not required to be sloped back or braced following federal OSHA requirements for excavations. Sides of temporarily vertical excavations less than 5 feet deep may stay open for short periods of time, however, the soils that will be encountered in trench excavations are subject to random caving and sloughing. If side slopes begin to slough, the sides should be either braced or be sloped back to at least 1V: 1H.

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

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

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

The bottoms of trench excavations should expose strong competent soils, and should be dry and free of loose, soft, or disturbed soil. If fill soils are encountered at the base of trench excavations, their competency should be verified through probing and density testing. Soft, wet, weak, or deleterious materials should be overexcavated to expose strong competent soils.

CONSTRUCTION CRITERIA

Site Preparation and Grading

Site stripping should be performed as needed to remove existing asphalt, concrete, abandoned buried utilities, foundations, vegetation, and deleterious debris. Exposed subgrade from excavations or grading operations within tank, building and pavement areas should be prepared as previously discussed in this report. A loaded dump truck weighing at least 20 tons should be utilized to proofroll over the given subgrade areas and a representative of the Geotechnical Engineer should be present to observe proofrolling operations. Areas of deflection should be removed, recompacted and/or replaced as per the representative of the Geotechnical Engineer. We recommend that one of our representatives be scheduled to observe that the site preparation operations are performed in accordance with our recommendations. If existing structures or deleterious materials are discovered during excavation, we should be informed immediately to determine the impact of those structures on our recommendations.

Fill materials required for general mass grading in pavement and general/common areas should consist of clean onsite materials or import materials meeting the requirements of general fill as defined herein. Import general fill should be a relatively uniform material: (1) free of roots, debris, or other deleterious materials, (2) have a maximum Plasticity Index (PI) of 25, and (3) not contain stones, clay clods, or particles exceeding 4 inches in maximum dimension. General fill should be placed in loose lifts with a maximum 8-inch thickness. Each lift should be compacted should be compacted to at least 95 percent of the maximum dry density determined by ASTM D 698 (standard effort). The moisture content during placement and compaction for each lift should be between optimum moisture content and plus four (+4) percentage points of the optimum moisture content (ASTM D 698).

Recommendations for select fill to be used in structural areas are presented in Table 6.

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

Recommended installation procedure	USACE refers to FHWA (FHWA-NHI-10-016, May 2010)
High-torque drilling equipment anticipated	Yes; high torque, high powered drilling equipment will be required to penetrate the very dense Clayey GRAVEL (GC) and very hard Limestone
Groundwater anticipated	Not Anticipated
Temporary casing anticipated	Not Anticipated
Slurry installation anticipated	Not Anticipated
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 verify the installed depth, should verify 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.

Earthwork and Foundation Acceptance

Exposure to the environment may weaken the soils at the foundation bearing level if the excavation remains open for long periods of time. Therefore, it is recommended that all foundation excavations be extended to final grade and constructed as soon as possible in order to help reduce potential damage to the bearing soils. If bearing soils are exposed to severe drying or wetting, the unsuitable soil must be re-conditioned or removed as appropriate and replaced with compacted fill, prior to concreting. The foundation bearing level should be free of loose soil, ponded water or debris and should be observed prior to concreting by the geotechnical engineer or his representative.

Foundation concrete 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 prior to placement of concrete.

Subgrade preparation and fill placement operations should be observed by the geotechnical engineer or his/her representative. As a guideline, at least one in-place density test should be performed for each 5,000 square feet of compacted surface per lift or a minimum of three tests per lift. Any areas not meeting the required compaction should be recompacted and retested until compliance is met.

Excavations

Excavations should comply with OSHA Standard 29CFR, Part 1926, Subpart P and all State of Texas and local requirements. Trenches 20 feet deep or greater require that the protective system be designed by a registered professional engineer. A trench is defined as a narrow excavation in relation to its depth. In general, the depth is greater than the width, but the bottom width of the trench is not greater than 15 feet. Trenches greater than 5 feet in depth require a protective system such as trench shields, trench shoring, or sloping back the excavation side slopes.

The Contractor's "Competent Person" shall perform daily inspections of the trench to verify that the trench is properly constructed and that surcharge and vibratory loads are not excessive, that excavation spoils are sufficiently away from the edge of the trench, proper ingress and egress into the trench is provided and all other items are performed as outlined in these OSHA regulations. It is especially important for the inspector to observe the effects of changed weather conditions, surcharge loadings, and cuts into adjacent backfills of existing utilities. The flow of water into the base and sides of the excavation and the presence of any surface slope cracks should also be carefully monitored by the Trench Safety Engineer. The Geotechnical Engineer should be made aware of any surface slope cracks that develop.

Although the geotechnical report provides an indication of soil types to be anticipated, actual soil and groundwater conditions will vary along the trench route. The "Competent Person" must evaluate the soils and groundwater in the trench excavation at the time of construction to verify that proper sloping or shoring measures are performed.

Appendix B to the regulations has sloping and benching requirements for short-term trench exposure for various soil types up to the maximum allowable 20-foot depth requirement.

GENERAL COMMENTS

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

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.

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, foundation construction and pavement installation are consistent with those encountered during this study. 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. We also should verify that the materials used as part of subgrade improvement, foundation construction, pavement installation, and other pertinent elements conform to the project specifications and that placement of these materials is in conformance with the specifications. 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.

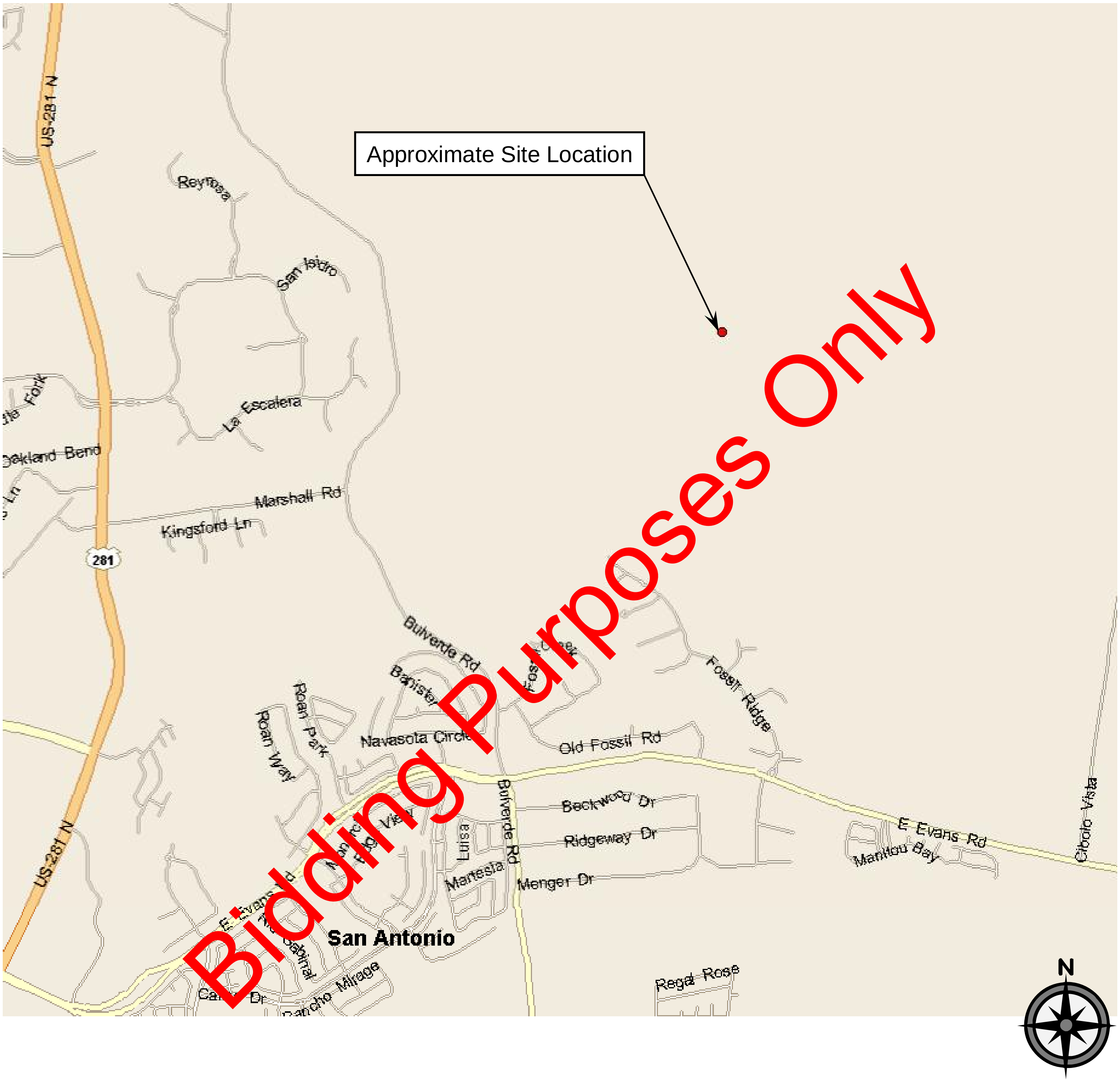
Subsurface Variations

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



ARIAS & ASSOCIATES, INC.

Geotechnical • Environmental • Testing
TBPE Registration No. F-32

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

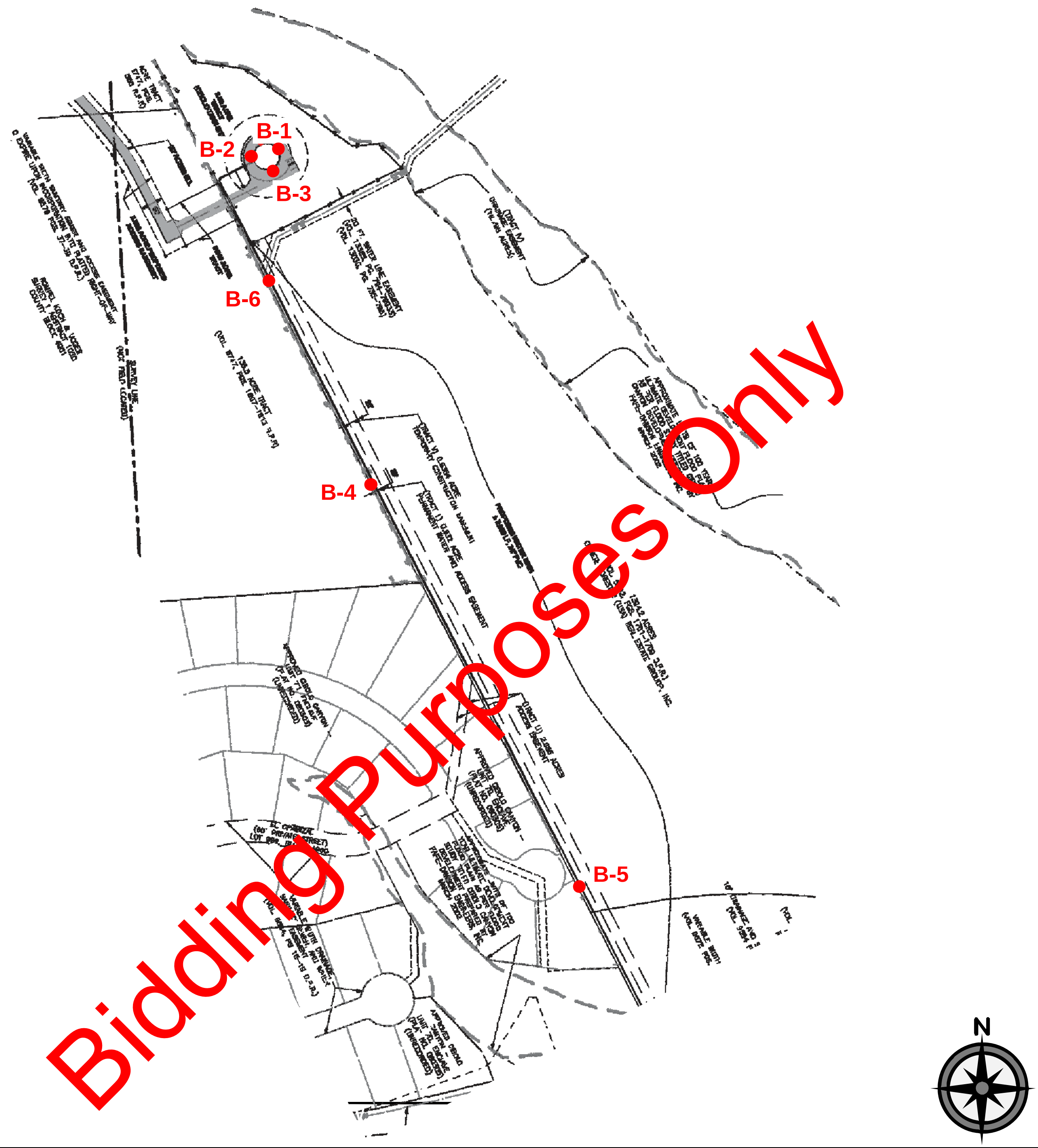
VICINITY MAP

SAWS Cibolo Composite Elevated Storage Tank Project
TPC Parkway and Bulverde Green
San Antonio, Bexar County, Texas

Date: October 31, 2011	Job No.: 2011-585
Drawn By: TAS	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Figure 1

1 of 1



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Office: (210) 308-5884 Fax: (210) 308-5886

BORING LOCATION PLAN

SAWS Cibolo Composite Elevated Storage Tank Project
TPC Parkway and Bulverde Green
San Antonio, Bexar County, Texas

REVISIONS:			Date: October 31, 2011	Job No.: 2011-585
No.:	Date:	Description:	Drawn By: TAS	Checked By: AMM
			Approved By: SAH	Scale: N.T.S.
			Figure 2	
				1 of 2

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142 Chula Vista
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Office: (210) 308-5884 Fax: (210) 308-5886

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			Figure 2	
				2 of 2

Photo 1 – Looking at area of proposed tank

Photo 2 – Looking at area of proposed line

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Geotechnical • Environmental • Testing
TBPE Registration No. F-32
142 Chula Vista
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Office: (210) 308-5884 Fax: (210) 308-5886

SITE PHOTOS

SAWS Cibolo Composite Elevated Storage Tank Project
TPC Parkway and Bulverde Green
San Antonio, Bexar County, Texas

Date: October 31, 2011	Job No.: 2011-585
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: **SAWS Cibolo Composite Elevated Storage Tank** **TPC Parkway and Bulverde Green** **San Antonio, Texas**	Sampling Date: 10/14/11
	Coordinates: N: 13793478.0683 E: 2154460.4632
Location: See Boring Location Plan	Backfill: Cement-bentonite grout

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
FAT CLAY (CH) with gravel, very hard, dark brown		SS	25				50/1"	75
Clayey GRAVEL (GC) with fractured limestone fragments, very dense, tan		SS	4	23	31	8	10/0"	18
LIMESTONE, very hard, light tan, cemented		SS	0				10/0"	
	5	SS	3				10/0"	
		SS	3	17	20	3	10/0"	
	10	GB	1					
		SS	2				10/0"	
	15							
		SS	2				10/0"	
	20							
		SS	2				10/0"	
	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: Survey

Nomenclature Used on Boring Log

Split Spoon (SS) Grab Sample (GB)

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

-200 = % Passing #200 Sieve

Boring Log No. B-2

Project: **SAWS Cibolo Composite Elevated Storage Tank**
TPC Parkway and Bulverde Green
San Antonio, Texas

Sampling Date: 10/14/11

Coordinates: N: 13793436.5759 E: 2154438.1391

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
Clayey GRAVEL (GC) with fractured limestone fragments, very dense, dark brown		SS	22	26	55	29	32/6"	22
LIMESTONE, very hard, light tan, cemented		SS	1				10/0"	
	5	SS	2				10/0"	
		SS	3	17	24	7	10/0"	
		SS	2				10/0"	
	10	GB	5					
		SS	3				10/0"	
	15							
		SS	3				10/0"	
	20							
		SS	2				10/0"	

Borehole terminated at 23.5 feet

Bidding Purposes Only

Groundwater Data:
During drilling: Not encountered

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

Nomenclature Used on Boring Log

Split Spoon (SS) Grab Sample (GB)

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

-200 = % Passing #200 Sieve

2011-585.GPJ 11/10/11 (BORING LOG SA11-01,ARIASSA10-01.GDT,LIBRARY2010.GLB)

Boring Log No. B-3

Project:	**SAWS Cibolo Composite Elevated Storage Tank** **TPC Parkway and Bulverde Green** **San Antonio, Texas**	Sampling Date: 10/14/11
		Coordinates: N: 13793438.6024 E: 2154485.7576
Location:	See Boring Location Plan	Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
FAT CLAY (CH), stiff, dark brown		SS	37				11	71
Clayey GRAVEL (GC) with sand and limestone fragments, very dense, reddish brown		SS	9				72	
	5	SS	6	19	29	10	50/6"	
		SS	12				50/6"	
LIMESTONE, very hard, light tan, cemented		SS	5	18	33	15	10/0"	
	10							
		GB	3					
		SS	2				10/0"	
	15							
		SS	1				10/0"	
	20							
		SS	1				10/0"	
	25							
		GB	1					

Borehole terminated at 27 feet

Groundwater Data:
During drilling: Not encountered

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

Nomenclature Used on Boring Log

Split Spoon (SS) Grab Sample (GB)

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

-200 = % Passing #200 Sieve

Boring Log No. B-4

Project: **SAWS Cibolo Composite Elevated Storage Tank**
TPC Parkway and Bulverde Green
San Antonio, Texas

Location: See Boring Location Plan

Sampling Date: 10/14/11

Coordinates: N: 13792686.1690 E: 2154664.5680

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
Clayey GRAVEL (GC) with limestone fragments, medium dense, dark brown		SS	38	33	76	43	10	48
LIMESTONE, very hard, light tan, cemented		SS	2				10/0"	
	5	SS	5				10/0"	
		SS	5	19	20	1	10/0"	
		SS	3				10/0"	
	10	GB	6					
		SS	3				50/1"	

Borehole terminated at 13.6 feet

Groundwater Data:
During drilling: Not encountered

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

Nomenclature Used on Boring Log

Split Spoon (SS) Grab Sample (GB)

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

-200 = % Passing #200 Sieve

Boring Log No. B-5

Project: **SAWS Cibolo Composite Elevated Storage Tank**
TPC Parkway and Bulverde Green
San Antonio, Texas

Sampling Date: 10/14/11

Coordinates: N: 13791796.2056 E: 2155053.3379

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N
LIMESTONE, very hard, light tan, cemented		SS	2				50/0"
		SS	1	20	18	NP	50/1"
	5	SS	2				10/0"
		SS	2				10/0"
		SS	2				10/0"
	10	GB	4				
	15	GB	4				

Borehole terminated at 15 feet

Groundwater Data:
During drilling: Not encountered

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

Nomenclature Used on Boring Log

Split Spoon (SS) | Grab Sample (GB)

WC = Water Content (%)
PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
NP = Non-plastic

N = SPT Blow Count

Boring Log No. B-6

Project: **SAWS Cibolo Composite Elevated Storage Tank**
TPC Parkway and Bulverde Green
San Antonio, Texas

Sampling Date: 10/14/11

Coordinates: N29°40'4.4" W98°24'56.3"

Location: See Boring Location Plan

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
Clayey GRAVEL (GC), medium dense, dark brown		SS	17	23	52	29	59/7"	18
LIMESTONE, very hard, light tan, cemented		SS	6				10/0"	
		SS	5				10/0"	

Borehole terminated at 4.5 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)

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

-200 = % Passing #200 Sieve

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

Arias & Associates, Inc.

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), using a thin-walled tube sampler (ASTM D 1587), 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. Penetrometer readings recorded for thin-walled tube samples that remained intact also are shown on the soil boring log.

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

Test Name	Test Method	Log Designation
Water (moisture) content of soil and rock by mass	ASTM D 2216	wc
Liquid limit, plastic limit, and plasticity index of soils	ASTM D 4318	LL, PL, 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: 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

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

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