Geotechnical Engineering Study

SAWS Regional Carrizo Water Integration
Pump Station Project
Schertz and San Antonio, Texas

Arias Job No. 2010-895

ARIAS & ASSOCIATES
Geotechnical • Environmental • Testing

Prepared For
Tetra Tech, Inc.

November 30, 2011

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Arias Job No. 2010-895

Mr. Fernando Roman, P.E.
Tetra Tech, Inc.
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San Antonio, Texas 78205

RE: Geotechnical Engineering Study
SAWS Regional Carrizo Water Integration Pump Station Project
Schertz and San Antonio, Texas

Dear Mr. Roman:

Arias & Associates, Inc. (Arias) is pleased to submit the results of a Geotechnical Engineering Study for the proposed SAWS Regional Carrizo Water Integration Pump Station Project in Schertz and 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

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INTRODUCTION

The results of a Geotechnical Engineering Study for the proposed SAWS Regional Carrizo Water Integration Pump Station Project in Schertz and San Antonio, Texas are presented in this report. This project was authorized on March 11, 2011, by Mr. Fernando Roman, P.E. of Tetra Tech, Inc. by means of the Standard Agreement between Tetra Tech, Inc. and Arias & Associates, Inc. (Arias). Our scope of work was performed in general accordance with the services outlined in Arias Proposal No. 2010-895, dated October 6, 2010 and revised Octover 19, 2011. The Notice-to-Proceed for the geotehcinal sevices for this project was isssued by Mr. Josh Sherman, P.E. with Tetra-Tech, Inc. on September 28, 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 building, pump station, control valve station, and ground 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 development of a new Pump Station facility and upgrades to an existing Pump Station facility included as a part of the planned improvements associated with the SAWS Regional Carrizo Project. The new Pump Station facility will be located at the SAWS Schertz Parkway site, northwest of FM 3009 and FM 78 in Schertz, Texas. This development will include the construction of a new 2.0 million gallon (MG) concrete ground storage tank, pump station pad, electrical and security building, chemical storage building and associated parking and access drives. The planned upgrades to the existing NACO Pump Station facility include the construction of a new control valve station pad. The NACO site is located about 7 miles due west of the Schertz site near Nacogdoches Road and O’Connor Road in San Antonio, Texas. 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 pump and control valve stations and buildings will be supported on shallow stiffened slab-on-grade foundation systems. The proposed tank structure will be supported on either: (1) a shallow concrete (monolithic floor slab and perimeter wall footing) foundation, or (2) a structurally suspended pile cap and deep drilled pier foundation system. Site improvements for the pump and control valve stations and buildings have been requested by the design team for design potential vertical rise (PVR) options of 1 inch and 1½ inches. Site improvements for the tank structure have been requested by the design team for design potential vertical rise (PVR) options of ¾ inch to 1-inch. 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.

The following foundations options are presented and discussed in this report:

1. Option I for Pump and Control Valve Stations and Buildings - Stiffened beam and slab-on-grade foundation systems constructed over engineered fill pads designed for a 1-inch or 1½-inch Potential Vertical Rise (PVR),

2. Option II for Pump and Control Valve Stations and Buildings - Mat foundation systems constructed over engineered fill pads designed for a 1-inch or 1½-inch PVR,

3. Option I for Tank Structure - Shallow concrete (monolithic floor slab and perimeter wall footing) foundation system constructed over an engineered fill pad designed for a ¾ inch to 1-inch PVR, and

4. Option II for Tank Structure – Structurally suspended pile cap on drilled pier foundations. The pile cap would need to be separated from the in-situ expansive soil with the use of a positive void space.

At the time of our field exploration conducted on October 6, 2011 and October 7, 2011, the Schertz site was observed in a relatively flat and undeveloped condition. The existing vegetation consisted of low grass and weeds. Onsite utilities are present. The NACO site consisted of an active pump station facility with associated pump station pads, ground storage tank, access drives and parking. Site photographs are included in Appendix A of this report.

SOIL BORINGS AND LABORATORY TESTING

Seven (7) soil borings were drilled at the approximate locations shown on the Boring Location Plan included as Figure 2 in Appendix A. One (1) boring was drilled at the NACO site and six (6) borings were drilled at the Schertz site. A description of the boring locations and boring depths are summarized in Table 1. The boring depths are referenced below the existing ground surface between October 6, 2011 and October 7, 2011. Drilling was performed in general accordance with ASTM D1586 and ASTM D1587 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.

Table 1: Approximate Boring Locations and Depths

Boring No.	Site Location	Proposed Structure	Depth Drilled
B-1	NACO Site	Control Valve Station Pad	15’
B-2	Schertz Site	Ground Storage Tank	48.5’
B-3	Schertz Site	Ground Storage Tank	44.9’
B-4	Schertz Site	Ground Storage Tank	43.5’
B-5	Schertz Site	Chemical Storage Building	33.5’
B-6	Schertz Site	Pump Station Pad	15’
B-7	Schertz Site	Electrical & Security Building	15’

Notes:

1. Depth is measured from existing ground surface at the time of the geotechnical study (October 2011).

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 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 – NACO Site

The earth materials underlying the project site have been regionally mapped as the Pecan Gap Chalk Formation Kpg) of the upper Cretaceous Period of the Geological Time Scale. Locally, the materials encountered in the boring consisted of approximately 6 feet of man-made fill soils, approximately 4 feet of natural surface soils, both overlying ancient marine deposits of the Pecan Gap Chalk.

The fill soils consist of gray brown clay (CL) with gravel in a hard to very hard condition. The natural topsoils consist of dark brown clay (CH) in a hard condition. The underlying marine deposits consist of tan and gray clay (CH) in a hard and weathered condition.

Geology – Schertz Site

The earth materials underlying the Schertz site have been regionally mapped as the alluvial Terrace (Qt) deposits of the Pleistocene Epoch overlying ancient marine deposits of the Pecan Gap Chalk Formation (Kpg) of the upper Cretaceous Period of the Geological Time Scale. The contact between the alluvial and marine deposits represents a significant erosional time gap and could be irregular with depth within the project area.

Locally, the materials encountered in the borings consist of approximately 3 to 5 feet of man-made fill soils and approximately 25 to 27.5 feet of alluvial terrace deposits overlying the ancient marine deposits. The man-made fill soils consist of brown and tan clay (CL-CH) generally in a very stiff to hard condition. The alluvial deposits are comprised of dark brown clay (CL-CH) and tan calcareous clay (CL) generally in a hard to very hard condition. The underlying marine deposits consist of gray claystone in a very hard condition.

Generalized Site Stratigraphy and Engineering Properties

The general stratigraphic conditions at the boring locations are provided in Tables 2 and 3 below. Table 2 presents the generalized stratigraphy encountered in the single boring drilled at the NACO site. Table 3 presents the generalized stratigraphy encountered within the 6 borings drilled at the Schertz site. *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 – NACO Site (*i.e.,* Boring B-1)

Stratum	Depth, ft	Material Type	PI range	No. 200 range	PP range	N Range
FILL	0 to 6	LEAN CLAY (CL) with gravel, very hard to hard, gray brown with light gray	31	90	--	28-53
I	6 to 10	FAT CLAY (CH) hard, dark brown	38	--	--	29-31
II	10 to 15	FAT CLAY (CH) hard, tan and gray	44	99	--	28-49

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

Table 3: Generalized Soil Conditions – Schertz Site (*i.e.,* Borings B-2 thru B-7)

Stratum	Depth, ft	Material Type	PI range	No. 200 range	PP range	N Range
			PI Avg.	No. 200 Avg.	PP Avg.	N Avg.
FILL	0 to (3-5)	LEAN CLAY (CL) and FAT CLAY (CH) with varying amounts of gravel, sand and debris, stiff to very hard, gray brown and tan	21-39	72-96	4.5+	15-63
			32	86	--	29
I	(3-5) to (6-18)	LEAN CLAY (CL), Sandy LEAN CLAY (CL), FAT CLAY (CH) and Sandy FAT CLAY (CH) with varying amounts of calcareous deposits, gravel and sand, hard to very hard, dark brown, dark reddish brown and reddish brown	21-41	50-96	--	29-64
			28	84	4.5+	45
IIa	(6-18.5) to (15-30)	Calcareous LEAN CLAY (CL) with sand, Lean CLAY (CL) with calcareous deposits and Sandy LEAN CLAY (CL) hard to very hard, tan	16-24	77-98	4-4.5+	29-85/10”
			22	87	4.5	>50
IIb	(8.5-22) to (18.5-31)	Clayey GRAVEL (GC) with sand and Clayey SAND (SC) with gravel, medium dense to very dense	14-22	25-30	--	21-50/5”
			18	28	--	>50
III	(28-31) to 48.5	CLAYSTONE, very hard, gray *Only observed at Borings B-2 thru B-5.*	--	--	--	>100

Fill Considerations

Based on the results of our field exploration, fills were observed to depths of approximately 3 to 5 feet (Schertz Site) and 6 feet (NACO site) below the existing ground surface (October 2011). The SPT N-values recorded during the sampling activities suggest that the fills are competent to support the proposed construction. However, without documentation of proper fill construction, there are inherent risks to the owner for the presence of buried rubble, debris and waste; and/or zones of loose, soft, or organic soils within the fill. Any of these conditions, if present, could adversely impact the proposed construction with the potential for increased construction costs and/or distress to newly constructed overlying structures. The risks can be reduced by excavating and recompacting onsite clean soils; however, they cannot be eliminated without completely removing and replacing the onsite fill.

We should note that the site recommendations presented in this report include removing 3 to 7 feet of onsite soils from within the proposed structure areas. Based on these recommendations, the majority, and in some areas all, of the existing fill will be removed from the proposed structure areas. The risks associated with undocumented fill can be lessened by having the Geotechnical Engineer or his/her representative present to: (1) observe foundation pad excavations, (2) observe the recommended proofrolling operations, and (3) perform earthwork compaction testing.

Groundwater

A dry soil sampling method was used to obtain the soil samples. Groundwater was observed within the two of the seven borings during sampling activities between October 6, 2011 and October 7, 2011. The borings were left open for a 24-hour period in order to obtain delayed groundwater readings and the depth to borehole caving/sloughing. Groundwater observations made during drilling and following a 24-hour wait are noted on the individual borings logs and summarized in the following table. 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.

Table 4: Groundwater Measurements

Boring No.	Approximate Location and Proposed Structure	Depth Drilled	Groundwater Depth During Drilling	Delayed Groundwater Reading	Depth at which Borehole Caved
B-1	NACO Site, Control Valve Station Pad	15'	Not Observed	Backfilled Upon Completion of Drilling	
B-2	Schertz Site, Ground Storage Tank	48.5'	Not Observed	Backfilled Upon Completion of Drilling	
B-3	Schertz Site, Ground Storage Tank	44.9'	28'	27'	37'
B-4	Schertz Site, Ground Storage Tank	43.5'	20'	27'	29.2'
B-5	Schertz Site, Chemical Storage Building	33.5'	Not Observed	Backfilled Upon Completion of Drilling	
B-6	Schertz Site, Pump Station Pad	15'	Not Observed	Backfilled Upon Completion of Drilling	
B-7	Schertz Site, Electrical & Security Building	15'	Not Observed	Backfilled Upon Completion of Drilling	

Notes:

1. Depth is measured from existing ground surface at the time of the geotechnical study (October 2011).

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, clay/sand or soil/claystone 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 NACO and Schertz site locations were explored to a maximum 15-foot and 48.5 foot depth, respectively.

Subsurface materials having similar consistency were extrapolated to be present between 15 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 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 78233 (NACO Site), zip code 78154 (Schertz Site) and Site Class D. Seismic design parameters for the site are summarized in the following table.

Table 5: Seismic Design Parameters for NACO and Schertz Sites

Site Classification	F_a	F_v	S_s	S_1
D for NACO Site	1.6	2.4	0.104g	0.031g
D for Schertz Site	1.6	2.4	0.103g	0.031g

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, pump and control valve stations and buildings 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. Considering the plasticity of the site soils, these soils would have a moderate to very 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 about ***3 to 4 inches at the NACO Site and about 1½ to 3 inches at the Schertz 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. 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. Similarly for building foundations, grade beams, isolated from soil, typically span between the piers and either a structurally suspended slab or soil supported slab-on-grade is used at the ground floor level. 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. Minimal aesthetic distress, such as floor tile, foundation and wall movement/cracking would be anticipated with the use of the deep foundation option. 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.

Based on our discussions with Tetra Tech, Inc., we understand that only shallow foundation systems are currently being considered for the proposed pump and control valve stations and buildings. For these structures, a shallow foundation type consisting of a stiffened beam and slab-on-grade is a common approach for lightly-loaded foundations/structures founded over expansive soils. Reinforced mat foundations may also be considered to support the proposed lightly-loaded structures. The proposed tank structure is planned to be supported on either a shallow concrete (monolithic floor slab and perimeter wall footing) foundation, or a pile cap and drilled pier foundation.

Subgrade improvement is recommended in the area of the proposed tank (shallow foundation system option only), pump and control valve stations and buildings to reduce potential soil and foundation movements to a magnitude within operational and structural tolerances.

We should note that although recommendations are provided in this report to help reduce the chances for significant expansive-soil related movement, the owner and design team should be cognizant that some movement should be expected for structures supported on shallow foundations.

Engineered Fill Pad Preparation

As a result of the anticipated expansive soil-related movement, site ground improvement will be needed to reduce the PVR or shrink-swell potential in the area of the proposed structures. The design PVR for the proposed pump and control valve stations, buildings and tank will be dependent upon both structural and operational tolerances. As requested by the design team, we are providing recommendations for: (1) an improved site PVR of about 1-inch and 1 ½ inches for the pump and control valve stations and buildings, and (2) an improved site PVR of about ¾ inch to 1-inch for the tank structure. We understand that the proposed structures can tolerate this amount of differential movement, and that aesthetic distress as previously discussed is acceptable for the buildings. *If project requirements dictate a different magnitude of PVR for higher levels of performance, we should be informed so that modifications to our recommendations can be made.*

The recommended site improvements are summarized in Table 6 below.

Table 6: Pad Recommendations for Proposed Tank, Pump and Control Valve Stations and Building Foundations

Site Improvement Method:	Undercut & Replace with Imported Select Fill				
Proposed Site:	**Schertz Site**			**NACO Site**	
Proposed Structure:	New Tank Structure	Pump Station and Buildings		Control Valve Station	
Client-Preferred Foundation Type:	Concrete Slab and Perimeter Wall Footing Foundation	Stiffened Beam and Slab-on-Grade or Mat Foundation		Stiffened Beam and Slab-on-Grade or Mat Foundation	
Estimated Site PVR Condition:	2 to 3 inches	1½ to 2 inches		3 to 4 inches	
Anticipated Desired Site Condition (PVR):	¾ inch to 1-inch	1 inch	1½ inch	1 inch	1½ inch
Minimum Undercut Depth:	**8 feet**	**4 feet**	**3 feet**	**7 feet**	**5 feet**
Minimum Select Fill Thickness:	**8 feet**	**4 feet**	**3 feet**	**7 feet**	**5 feet**
Select Fill Type:	TxDOT Item 247, Type A, Grade 1 or 2 Crushed Limestone Material	Pit Run – Liquid Limit <40%, PI 10-20, max. 4” particle size *or* TxDOT Item 247, Type A, Grade 1 or 2 Crushed Limestone Material			
Scarify, Moisten & Compact Exposed Subgrade:	12 inches				
Exposed Subgrade Treatment (See Note 4):	Proof roll with 20 ton loaded dump truck for a minimum of 20 passes under direction of Geotechnical Engineer’s Representative				
Pumping/Rutting Areas Discovered During Proofrolling	Remove to firmer materials and replace with compacted general or select fill under direction of Geotechnical Engineer’s representative				
Clay Cap Requirement:	See Note 7				

Notes:

1. The existing soils in area of the proposed structures should be undercut (over-excavated) as necessary to allow for the placement of the minimum select fill shown above for the desired design PVR.

2. If site final grades are above existing grade, the select fill thickness will need to be increased to accommodate the grade change.

3. The undercut zone should extend at least 3 feet laterally beyond the perimeter of the proposed structure foundation or greater than 3 feet to include any movement sensitive flatwork or adjacent ancillary structures. This recommended overbuild will aid in providing more uniform compaction of the soils beneath the structures.

4. The upper 12 inches of exposed subgrade at the base of the excavation should then be moisture conditioned to between optimum moisture content and +4 percentage

points of optimum moisture content and compacted to at least 95 percent of ASTM D 698.

5. After moisture conditioning and compacting the subgrade, imported select fill should be placed within the excavation in loose lifts with a maximum 8-inch thickness. Each lift should be compacted to at least 98% of the maximum dry density determined by ASTM D 698 at moisture contents between -2 and +3 percentage points of optimum moisture content. The select fill should be placed within 48 hours of completion of the subgrade compaction.

6. Select fill should be: (1) free of roots, debris, or other deleterious materials, (2) have a PI between 10 and 20, and (3) not contain stones, clay clods, or particles exceeding 4 inches in maximum dimension. For the tank pad, the entire select fill thickness should consist of crushed limestone meeting the requirements outlined in TxDOT Item 247, Type A, Grade 1 or 2. For the pump and control valve stations and buildings, consideration should be given to completing the final 6 inches of the pad with crushed limestone base to help provide a more "all-weather" working platform.

7. To reduce the potential for surface water infiltration into the select fill body, we recommend that a 24-inch thick clay "cap" be constructed over the previously noted select fill overbuild. Clean onsite Stratum I CLAY (CL-CH) soils can be used to construct this clay "cap". The clay "cap" should be placed in 6-inch compacted lifts. Each lift should be moisture conditioned to between -2 and +3 percentage points of optimum moisture content. The moisture conditioned lift should be compacted to at least 95 percent of ASTM D 698. This clay "cap" is not needed where pavement or flatwork abuts the structure and covers the select fill overbuild.

Measures to Reduce Soil Moisture Change

The following design measures are recommended to help reduce potential soil shrink/swell foundation movements. Although subgrade modification through excavation and select fill replacement is recommended to reduce potential shrink/swell related foundation movements, the design and construction of a shallow foundation should also include the following elements:

- The ground surface adjacent to the 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.
- Site work excavations should be protected and backfilled without delay in order to reduce changes in the natural moisture regime for soils used to backfill the site and achieve design grades
- Roof drainage should be controlled by gutters and carried well away from the structure.
- 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.
- Paved areas around the structure are helpful in maintaining equilibrium within the soil water content. If possible, pavement and sidewalks should be located immediately adjacent to the structure.
- Flower beds and planter boxes should be piped or water tight to prevent water infiltration under the building. Experience indicates that landscape irrigation is a common source of foundation movement problems and pavement distress.
- Clay “cap” construction should be provided as previously recommended.

Grade-Supported Flatwork and other Ancillary Structures

Minor differential movements between the planned structures and abutting sidewalks/ancillary structures should be expected if the flatwork/structure is supported on similar engineered fill pad conditions. Thus, we recommend that the flatwork/structure be supported entirely on the improved building pad. Flatwork or other grade-supported elements supported on unimproved, natural site conditions will result in movements on the order of the magnitudes reported in the PVR section that can result in significant cracking, joint separations, and a reversal in drainage as discussed subsequently.

We recommend that the flatwork and the buildings be designed to include details that permit foundation movements without resulting in vertical separations and without distressing either element. Control joints should be included that include steel reinforcing to prevent vertical shear, but to allow bending.

The flatwork and abutting sidewalks should be designed and constructed to allow for positive drainage to be maintained away from the building foundations. The planned site grading should allow for potential future differential movements and should never be allowed to reach a level or negative slope that promotes drainage toward the foundation. This reversal in drainage can direct moisture back towards the building and can become a constant nuisance and maintenance issue. If the potential differential movements cannot be tolerated, the Owner may wish to consider extending the foundation pad beneath the planned sidewalks and other movement-sensitive ancillary structures.

Option I for Pump and Control Valve Stations & Buildings – Stiffened Beam and Slab-on-Grade Foundation

A grid type beam and slab-on-grade foundation is generally used to support relatively light structures upon expansive soils where soil conditions are relatively uniform, and where uplift and settlement can be tolerated. The intent of a stiffened beam and slab-on-grade foundation is to allow the structure and foundation to move up and down with soil movements while providing sufficient stiffness to limit differential movements within the superstructure to an acceptable magnitude.

A stiffened grid type beam and slab-on-grade foundation may be utilized for the proposed lightly-loaded structures provided they are designed specifically for these soil conditions and the building pad and/or site is improved as outlined in Table 6.

There are various design methods for use by the structural engineer to select the grade beams depths and beam spacings for the project. The foundation may be designed using the Building Research Board No. 33 (BRAB Report) as a guideline. Alternatively, the foundation may be designed based on the Design of Slab-On-Ground Foundations published by the Wire Reinforcement Institute, Inc. (August 1981). Provided in the following table are design criteria for both methods.

Table 7: BRAB and WRI Foundation Design Criteria

Proposed Site:	**Schertz Site**		**NACO Site**	
Proposed Structures:	Pump Station and Buildings		Control Valve Station	
Existing PVR Site Condition:	1½ to 3 inches		3 to 4 inches	
Improved PVR Site Condition (See Table 6):	1 inch	1½ inch	1 inch	1½ inch
Climatic Rating (Cw)	17	17	17	17
Effective Plasticity Index	28	30	30	34
BRAB – Support Index (C)	0.87	0.85	0.85	0.78
WRI – Soil/Climatic Rating Factor (1-C)	0.13	0.15	0.15	0.22
BRAB – Unconfined Compressive Strength (tsf)	1.0	1.0	1.0	1.0

Notes:

1. The above design values assume that the pad has been improved as outlined in this report for an improved 1-inch or 1½-inch PVR site condition.

A stiffened beam and slab type foundation may also be designed using the 3rd Edition of the Design of Post-Tensioned Slabs-on-Ground published by the Post-Tensioning Institute. These values were estimated from the "Volflo" computer program in consideration of the soil conditions in the building area. Provided in the following table are design criteria for this method.

Table 8: PTI Slab-on-Grade Soil Design Criteria (3rd Edition)

Proposed Site:	**Schertz and NACO Sites**			
Proposed Structures:	Pump and Control Valve Stations and Buildings			
Improved PVR Site Condition (See Table 6):	1 inch		1½ inch	
Soil Moisture Variation Case:	Dry to Wet *Soil Heave*	Wet to Dry *Soil Shrinkage*	Dry to Wet *Soil Heave*	Wet to Dry *Soil Shrinkage*
Mode:	Edge Lift	Center Lift	Edge Lift	Center Lift
Edge Moisture Variation Distance, e_m (ft):	4.5	9.0	4.3	8.3
Differential Movement, Y_m (in):	1.3	1.0	1.7	1.2

Notes:

1. The above design values assume that the pad has been improved as outlined in this report for an improved 1-inch or 1½-inch PVR site condition.

Table 9: Allowable Bearing Pressure and Beam Penetration

Allowable Bearing Pressure - (Continuous Wall Load)	2,400 psf
Allowable Bearing Pressure - (Point Load)	2,800 psf
Bearing Stratum at Bottom of Grade Beams	Compacted Select Fill
Min. Penetration of Beams Below Final Grade	30 inches

Notes:

1. The above design values assume that the pad has been improved as outlined in this report for an improved 1-inch or 1½-inch PVR site condition.

We are providing design values for BRAB, WRI, and PTI methods for the Structural Engineer's consideration and possible use. Arias recommend the final design methodology for the planned foundations be selected by the project Structural Engineer based on his knowledge and experience with similar foundation conditions.

Grade beams based at the recommended depth and founded within the existing fill soils or compacted select fill, should be designed for the allowable soil bearing capacity provided above. Grade beams may be thickened and widened at concentrated loads to serve as spread footings. The beams and widened columns should be a minimum of 10 and 12 inches wide, respectively, for shear resistance. The grade beams should extend at least 30 inches below final grade within the existing fill soils or compacted select fill.

We recommend that at least a 10-mil vapor barrier be used under the slab. The vapor barrier should conform to ASTM E1745, Class C or better and shall have a maximum water vapor permeance of 0.044 perms when tested in accordance with ASTM E96. A 10 mil Stego Wrap by Stego Industries LLC or other similar products meeting these requirements would be acceptable.

Option II for Pump and Control Valve Stations & Buildings – Mat Foundation

A mat foundation may be used for the proposed pump and control valve stations and buildings provided that: (1) they are founded on an engineered fill pad as outlined in Table 6 of this report, (2) they are designed with adequate reinforcement and of sufficient foundation stiffness to resist the potential differential shrink/swell movements, and (3) measures to reduce soil moisture change are implemented as outlined in the previous report sections. A mat foundation can be designed for a net allowable soil bearing pressure of **3,000 psf** for a mat bearing at least two (2) feet below the final grade in compacted engineered fill. The provided allowable bearing pressure includes a factor of safety against bearing capacity failure of at least 2. The modulus of the subgrade reaction can be taken as k = 50 pci.

It may be advantageous to use a "turned down" perimeter beam to help protect the foundation edges from erosion.

Option I for Tank Structure – Floor Slab with Perimeter Wall Footing (Schertz Site only)
The proposed tank can be supported on a monolithic floor slab and perimeter wall footing foundation provided it is founded on an engineered fill pad (See Table 6) and that the measures to reduce soil moisture change are implemented as outlined previously. The shallow footing foundation should be designed as required in accordance with ACI 372R-03. The footing foundation should bear in compacted select fill at least 2.5 feet below final grade. The allowable bearing pressure for the perimeter tank footing is **4,500 psf** based on total load conditions, and includes a factor of safety of 3.0 against bearing failure. This bearing value assumes that the footing bears uniformly on properly placed and compacted select fill consisting of TxDOT Item 247, Type A, Grade 1 or 2 Crushed Limestone Material.

The backfill behind the tank wall and above the perimeter footing should consist of onsite clay (CL-CH). The intention of the clay backfill is to reduce the potential for surface water infiltration into the select fill body. The backfill should extend at least 3 feet laterally beyond the perimeter footing and should be placed in 6-inch compacted lifts. Each lift should be moisture conditioned to between -2 and +3 percentage points of optimum moisture content. The moisture conditioned lift should be compacted to at least 95 percent of ASTM D 698.

Total settlement of the center of the tank foundation is anticipated to be between about 1½ and 2 inches. Differential settlement from one side of the tank to the other is anticipated to be about ¾ to 1-inch. Differential heave is estimated to be about ¾ to 1 inch provided that measures are taken to reduce the potential for localized soil wetting.

Slab thickness and reinforcing should be determined by the project structural engineer. A Subgrade Modulus k = 50 pci may be used for the slab design.

Option II for Tank Structure – Drilled Pier Foundation (Schertz Site only)
Based upon the subsurface conditions observed at Borings B-2 to B-4 and the results of the laboratory testing performed on the soil samples, straight-shaft drilled pier foundations with a structurally suspended pile cap may be used to support the proposed tank. Applicable geotechnical foundation design parameters are discussed below for this foundation system. Recommendations for evaluation of axial capacity and lateral capacity are presented in the following tables. Pier capacities for axial loading were evaluated based on design methodologies included in FHWA-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 10: Drilled Pier Design Parameters for Tank (Borings B-2 to B-4 only) – Axial Capacity

Depth	Material	Recommended Design Parameters		
		Allowable Skin Friction, psf ($\alpha c/FS$)	**Allowable End Bearing, psf (cN_c/FS)**	**Uplift Force, kips**
0 to 5	FILL: CLAY (CL-CH)	Neglect Contribution		
5 to 12	CLAY (CL-CH)	650	--	60D
12 to 22	CLAY (CL-CH), Clayey SAND (SC) or Clayey GRAVEL (GC)	750	--	
22 to 30	Clayey SAND (SC) or Clayey GRAVEL (GC)	800	--	
30 to 48.5	CLAYSTONE	2,250	25,000	
Constraints to be Imposed During Pier Design				
Minimum embedment depth	35 feet below grade or 3 feet into CLAYSTONE – whichever results in a deeper pier			
Minimum shaft diameter	24 inches			
Minimum Void Space beneath Pile cap/Ring Beam	8 inches			

Notes:

1. Because of the high potential for expansive soil-related movement associated with high plasticity clays in a dry condition, the pile cap should be separated from the expansive soil subgrade. We recommend that an 8-inch positive void space be provided between the cap bottom and underlying subgrade.

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. 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 35 feet may be required to resist expansive soil uplift forces. Uplift tensile loads induced from wind loading and swelling soils can be resisted by applying the allowable skin friction resistance using the parameters in Table 10 below a 15-foot depth.

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. Deeper pier depths may be required to resist swelling soils. Pier vertical reinforcing steel should be designed to resist the uplift forces from swelling soils. A minimum of 1.5% 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. The loading (shear force and bending moment) may dictate the pier steel 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 Schertz/San Antonio area. A detailed settlement estimate is outside of the scope of this service.

It should be noted that high torque, high powered drilling equipment will be required to penetrate the very hard Claystone and very dense Clayey Sand/Clayey Gravel encountered at this site.

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 11: Drilled Pier Geotechnical Input Parameters for LPILE Analyses for Tank (Borings B-2 to B-4 only)

Depth (ft)	Material	γ_e	C_u	ϕ	K (cyclic loading)	e_{50}
0 to 5	FILL: CLAY (CL-CH)	Neglect Contribution				
5 to 12	CLAY (CL-CH)	120	2,500	0	1,000	0.005
12 to 22	CLAY (CL-CH)	120	3,500	0	1,000	0.005
	Clayey SAND (SC) or Clayey GRAVEL (GC)	120	0	32	225	--
22 to 30	Clayey SAND (SC) or Clayey GRAVEL (GC)	63	0	32	225	--
30 to 48.5	CLAYSTONE	73	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 about 27 feet based on boring data

PAVEMENT RECOMMENDATIONS FOR SCHERTZ SITE

The proposed Schertz site development will include the construction of new access drives and parking areas. No specific design traffic information was received for this project. Therefore, the design parameters and assumptions included in Table 12 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 12: 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 (CH) subgrade
Raw Subgrade Modulus of Subgrade Reaction, k in pci	75 for moderate to high plasticity compacted clay (CH) subgrade

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 13. 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 not utilize asphalt pavement. For these areas, we recommend rigid concrete pavements.

Table 13: 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. Light duty areas include parking and drive lanes that are subjected to passenger vehicle traffic only.
2. 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.
3. 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 14: 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 Bexar County 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 Bexar County 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 Bexar County 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 Bexar County 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 Bexar County 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.

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 15: 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 hard Claystone and very dense Clayey Sand and Clayey Gravel.
Groundwater anticipated	Yes; groundwater observed at 20 to 28 feet during sampling activities; delayed groundwater measured at 27 feet below the existing ground surface at the time of the field exploration. Possibly shallow "perched" water in gravelly and sandy soils underlain by Claystone.
Temporary casing anticipated	Probable, possible caving sand and gravel and influx of groundwater
Slurry installation anticipated	Possible depending upon effectiveness of casing
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 Tetra Tech, Inc., 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. 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, 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 Regional Carrizo Water Integration
Pump Station Project
San Antonio & Schertz, Bexar & Guadalupe County, Texas

Date: October 17, 2011	Job No.: 2010-895
Drawn By: TAS	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Figure 1

ARIAS & ASSOCIATES, INC.
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San Antonio, Texas 78232
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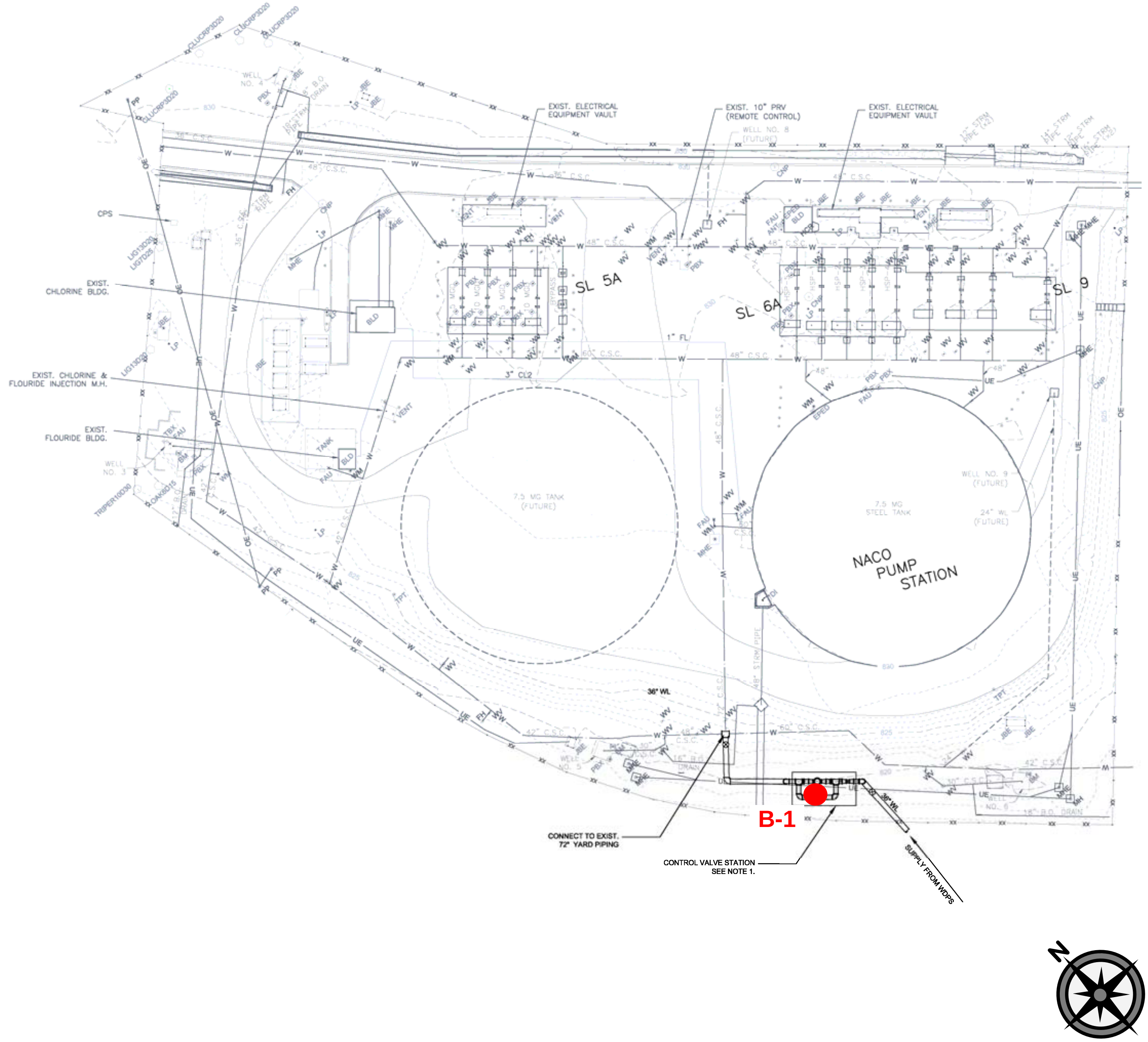
BORING LOCATION PLAN

SAWS Regional Carrizo Water Integration
Pump Station Project, NACO Site
San Antonio, Bexar County, Texas

REVISIONS:				
No.:	Date:	Description:	Date: October 17, 2011	Job No.: 2010-895
			Drawn By: TAS	Checked By: AMM
			Approved By: SAH	Scale: N.T.S.

Figure 2



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TBPE Registration No. F-32

142 Chula Vista
San Antonio, Texas 78232
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BORING LOCATION PLAN

SAWS Regional Carrizo Water Integration
Pump Station Project, NACO Site
San Antonio, Bexar County, Texas

REVISIONS:			Date: October 17, 2011	Job No.: 2010-895
No.:	Date:	Description:	Drawn By: TAS	Checked By: AMM
			Approved By: SAH	Scale: N.T.S.
			Figure 2	
				2 of 4

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TBPE Registration No. F-32

142 Chula Vista
San Antonio, Texas 78232
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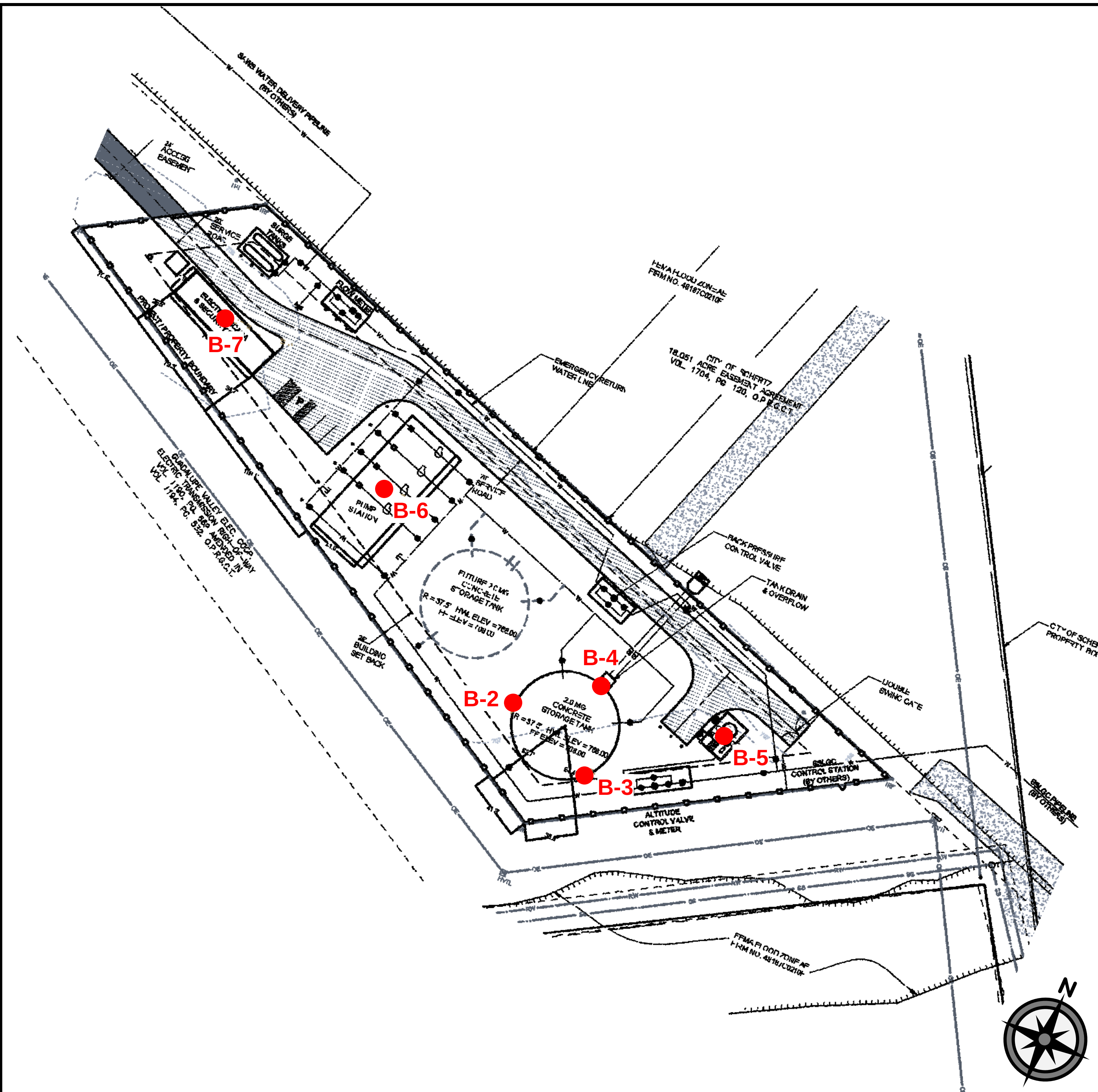
BORING LOCATION PLAN

SAWS Regional Carrizo Water Integration
Pump Station Project, Schertz Site
Schertz, Guadalupe County, Texas

REVISIONS:			Date: October 17, 2011	Job No.: 2010-895
No.:	Date:	Description:	Drawn By: TAS	Checked By: AMM
			Approved By: SAH	Scale: N.T.S.
			Figure 2	
				3 of 4



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TBPE Registration No. F-32
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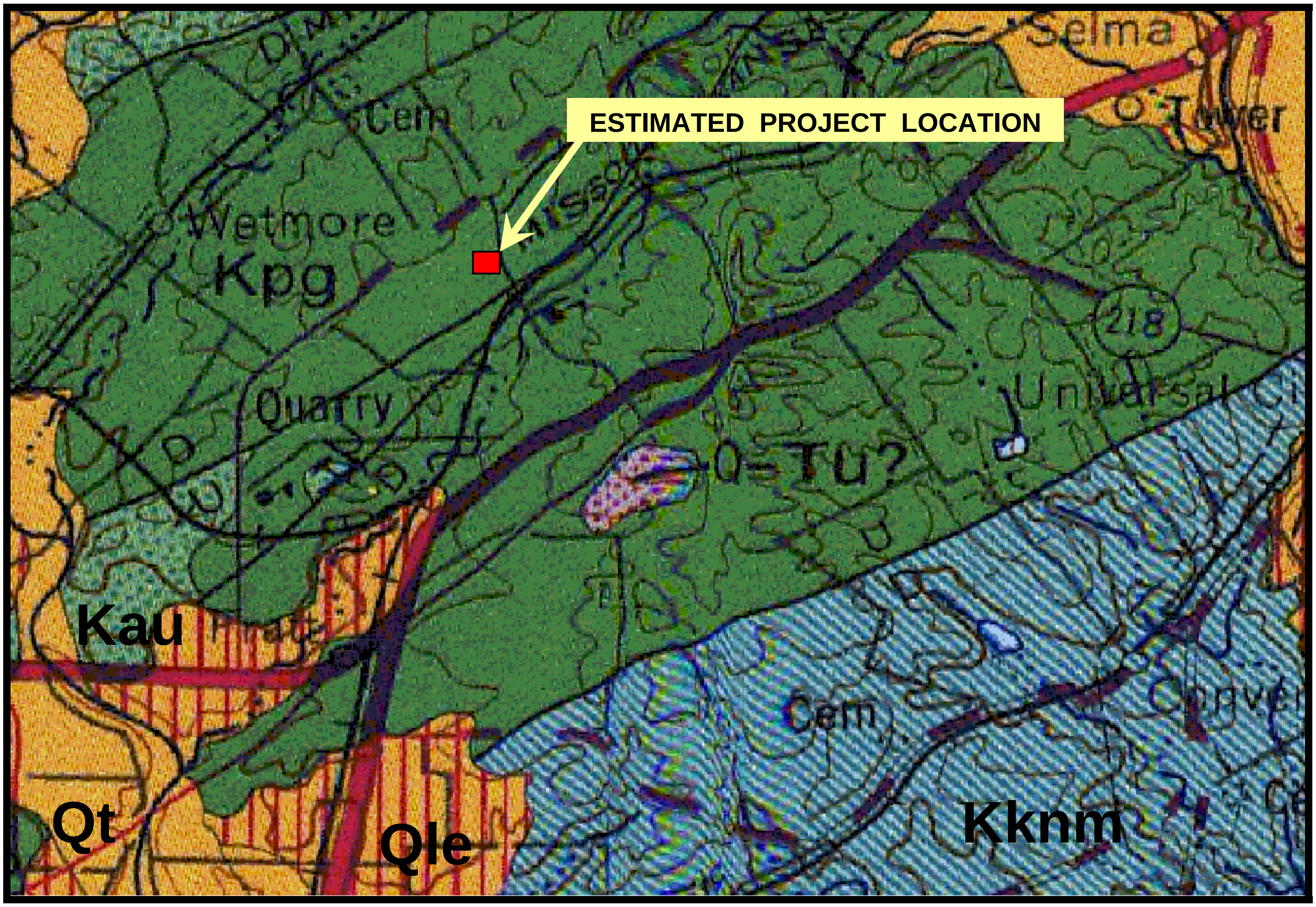
BORING LOCATION PLAN

SAWS Regional Carrizo Water Integration
Pump Station Project, Schertz Site
Schertz, Guadalupe County, Texas

REVISIONS:			Date: October 17, 2011	Job No.: 2010-895
No.:	Date:	Description:	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
Qt	Alluvial Terrace Deposits	Quaternary Period / Pleistocene Epoch
Qle	Leona Formation (Alluvium)	Quaternary Period / Pleistocene Epoch
Q-Tu	Uvalde Gravel Formation	Quaternary – Tertiary Periods
Kknm	Navarro Formation	Upper Cretaceous Period
Kpg	Pecan Gap Chalk Formation	Upper Cretaceous Period
Kau	Austin Chalk Formation	Upper Cretaceous Period

U D — Fault Segment with Indication of Relative Movement

N

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San Antonio, Texas 78232
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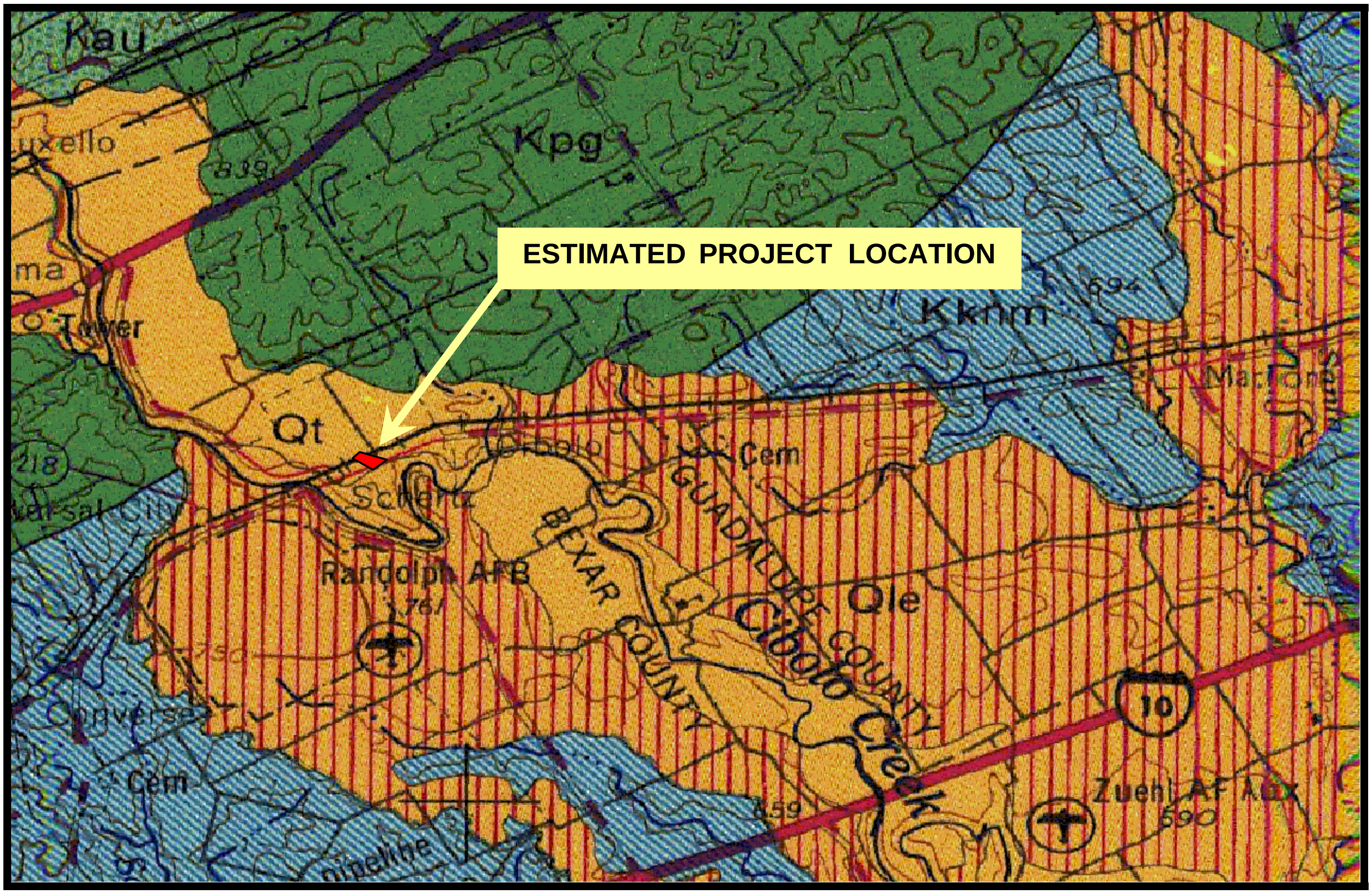
GEOLOGIC MAP

SAWS Regional Carrizo Water Integration
Pump Station Project
San Antonio, Bexar County, Texas

Date: October 27, 2011	Job No.: 2010-895
Drawn By: JLK	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Figure 3



PORTION OF GEOLOGIC ATLAS OF TEXAS

LEGEND

Symbol	Name	Age
Qt	Alluvial Terrace Deposits	Quaternary Period / Pleistocene Epoch
Qle	Leona Formation (Alluvium)	Quaternary Period / Pleistocene Epoch
Kknm	Navarro Formation	Upper Cretaceous Period
Kpg	Pecan Gap Chalk Formation	Upper Cretaceous Period
Kau	Austin Chalk Formation	Upper Cretaceous Period

U D Fault Segment with Indication of Relative Movement

N

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TBPE Registration No. F-32
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San Antonio, Texas 78232
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GEOLOGIC MAP

SAWS Regional Carrizo Water Integration
Pump Station Project
Schertz, Guadalupe County, Texas

Date: October 26, 2011	Job No.: 2010-895
Drawn By: JLK	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Figure 3

Photo 1: Naco Site

Photo 2: Schertz Site

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TBPE Registration No. F-32
142 Chula Vista
San Antonio, Texas 78232
Office: (210) 308-5884 Fax: (210) 308-5886

SITE PHOTOS

SAWS Regional Carrizo Water Integration
Pump Station Project
San Antonio & Schertz, Bexar & Guadalupe County, Texas

Date: October 27, 2011	Job No.: 2010-895
Drawn By: TAS	Checked By: AMM
Approved By: SAH	Scale: N.T.S.

Appendix A

1 of 2

Photo 3: Schertz Site

Photo 4: Schertz Site

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

SITE PHOTOS

SAWS Regional Carrizo Water Integration
Pump Station Project
San Antonio & Schertz, Bexar & Guadalupe County, Texas

Date: October 27, 2011	Job No.: 2010-895
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 Regional Carrizo Project** **See Boring Location Plan** **San Antonio and Schertz, Texas**	Sampling Date: 10/6/11
	Coordinates: N29°33'54.1" W98°23'9.1"
Location: Nacogdoches: See Boring Location Plan	Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	N	-200
FILL: LEAN CLAY (CL) with gravel, very hard to hard, gray brown with light gray		SS	8				53	
		SS	10	16	47	31	51	90
	5	SS	11				28	
FAT CLAY (CH), hard, dark brown		SS	16	17	55	38	31	
	10	SS	15				29	
FAT CLAY (CH), hard, tan and gray		SS	17	20	64	44	28	99
- color changes to brown and gray at 14ft.	15	SS	18				49	

Borehole terminated at 15 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Logged By: J. Kniffen
Driller: Eagle Drilling, Inc.
Equipment: Track-mounted drill rig
Dry-auger drilling: 0 ft to 15 ft
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

2010-895.GPJ 10/28/11 (BORING LOG SA11-01,ARIASSA10-01.GDT,LIBRARY2010.GLB)

Boring Log No. B-2

Project: **SAWS Regional Carrizo Project**
See Boring Location Plan
San Antonio and Schertz, Texas

Location: Schertz: See Boring Location Plan

Sampling Date: 10/6/11

Coordinates: N29°33'59.1" W98°15'51.4"

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200	DD	Uc
FILL: LEAN CLAY (CL) with sand, hard, brown and tan		SS	11					30			
		SS	10	16	48	32		33	72		
LEAN CLAY (CL) with calcareous deposits, hard, dark brown	5	SS	15					29			
		T	15	21	49	28	4.5+		91		
LEAN CLAY (CL) with sand and calcareous deposits, hard, reddish brown		T	15				4.5+				
	10	T	16	18	44	26	4.5+		90	109	4.06
	15	T	17				4.5+				
Calcareous LEAN CLAY (CL) with sand, hard, tan	20	T	16	14	30	16	4		77	114	3.53
Clayey SAND (SC) with calcareous deposits and partially cemented seams, very dense, tan to light tan	25	SS	6	13	27	14		50/5"	29		
	30	SS	10					50/5"			
CLAYSTONE, very hard, light gray	35	SS	8					50/0"			
	40	SS	8					50/0"			
	45	SS	9					10/0"			
		SS	8					10/0"			
Borehole terminated at 48.5 feet											

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Logged By: J. Kniffen
Driller: Eagle Drilling, Inc.
Equipment: Track-mounted drill rig
Dry-auger drilling: 0 ft to 48.5 ft
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)

Boring Log No. B-3

Project: **SAWS Regional Carrizo Project**
See Boring Location Plan
San Antonio and Schertz, Texas

Location: Schertz: See Boring Location Plan

Sampling Date: 10/6/11

Coordinates: N29°33'58.9" W98°15'50.7"

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FILL: FAT CLAY (CH) with gravel and debris, very stiff, brown and tan		SS	10	19	53	34		17	
FAT CLAY (CH) with calcareous deposits, hard, dark brown		T	12				4.5+		
	5	T	18	23	51	28	4.5+		92
		T	11				4.5+		
Sandy LEAN CLAY (CL) with gravel and calcareous deposits, hard, reddish brown	10	T	9	20	44	24	4.5+		50
		SS	10					64	
Clayey GRAVEL (GC) with sand, dense to medium dense, tan	15	SS	6					36	26
	20	SS	8					28	
	25	SS	6					40	
- dense									
CLAYSTONE, very hard, light gray	30	SS	14	14	30	16		10/0"	
	35	SS	17					10/0"	
	40	SS	13					10/0"	
		SS	11					50/5"	

Borehole terminated at 44.9 feet

Groundwater Data:
First encountered during drilling: 28-ft depth
After 24 hr.: At 27-ft depth (37-ft open borehole depth)

Field Drilling Data:
Logged By: J. Kniffen
Driller: Eagle Drilling, Inc.
Equipment: Track-mounted drill rig
Dry-auger drilling: 0 ft to 44.9 ft
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: **SAWS Regional Carrizo Project**
See Boring Location Plan
San Antonio and Schertz, Texas

Location: Schertz: See Boring Location Plan

Sampling Date: 10/7/11

Coordinates: N29°33'59.5" W98°15'50.9"

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FILL: FAT CLAY (CH), trace sand, very hard to hard, brown and tan		SS	15	19	52	33		63	85
		SS	13					27	
FAT CLAY (CH), hard, dark reddish brown to reddish brown	5	T	18	21	62	41	4.5+		94
		T	17				4.5+		
	10	T	18	20	50	30	4.5+		96
		T	19				4.5+		
Calcareous LEAN CLAY (CL) with sand, hard, tan	15	T	13	17	41	24	4.5+		
Clayey SAND (SC) with gravel, dense to very dense, tan	20	SS	7					40	25
	25	SS	8					55	
CLAYSTONE, very hard, light gray	30	SS	16					10/0"	
	35	SS	15					10/0"	
	40	SS	14					10/0"	
		SS	27					10/0"	
Borehole terminated at 43.5 feet									

Groundwater Data:
First encountered during drilling: 20-ft depth
After 24 hr.: At 27-ft depth (29.2-ft open borehole depth)

Field Drilling Data:
Logged By: J. Kniffen
Driller: Eagle Drilling, Inc.
Equipment: Track-mounted drill rig
Dry-auger drilling: 0 ft to 43.5 ft
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-5

Project: **SAWS Regional Carrizo Project**
See Boring Location Plan
San Antonio and Schertz, Texas

Location: Schertz: See Boring Location Plan

Sampling Date: 10/7/11

Coordinates: N29°33'59.5" W98°15'49.9"

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FILL: FAT CLAY (CH), very stiff to hard, brown and tan		SS	19					18	
Sandy FAT CLAY (CH) with gravel, hard, dark brown		T	18	23	62	39	4.5+		92
	5	T	13				4.5+		
		T	12	19	50	31	4.5+		69
Clayey GRAVEL (GC) with sand, very dense, tan		T	5						30
	10	SS	11	16	38	22		54	
Clayey SAND (SC) with gravel, medium dense, tan	15	SS	8					21	
Sandy LEAN CLAY (CL), very hard to hard, tan	20	SS	3	14	24	10		56	
	25	SS	6					29	
	30	SS	13					85/10"	
CLAYSTONE, very hard, gray		SS	12					50/0"	

Borehole terminated at 33.5 feet

Groundwater Data:
First encountered during drilling: 28-ft depth
After 24 hr.: At 27.2-ft depth (28.4-ft open borehole depth)

Field Drilling Data:
Logged By: R. Arizola/J. Kniffen
Driller: Eagle Drilling, Inc.
Equipment: Track-mounted drill rig
Dry-auger drilling: 0 ft to 33.5 ft
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-6

Project: **SAWS Regional Carrizo Project**
See Boring Location Plan
San Antonio and Schertz, Texas

Location: Schertz: See Boring Location Plan

Sampling Date: 10/7/11

Coordinates: N29°34'0.1" W98°15'53.1"

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FILL: FAT CLAY (CH), stiff to hard, brown and tan		SS	17	21	53	32		15	96
		SS	13					36	
LEAN CLAY (CL), hard, dark brown	5	SS	11	17	40	23		42	89
LEAN CLAY (CL) with calcareous deposits, very hard to hard, tan		SS	10					59	
	10	T	12	16	40	24	4.5+		98
		T	12				4.5+		
- very hard	15	SS	15					68	

Borehole terminated at 15 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Logged By: R. Arizola
Driller: Eagle Drilling, Inc.
Equipment: Track-mounted drill rig
Dry-auger drilling: 0 ft to 15 ft
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

2010-895.GPJ 10/28/11 (BORING LOG SA11-01,ARIASSA10-01.GDT,LIBRARY2010.GLB)

Boring Log No. B-7

Project: **SAWS Regional Carrizo Project**
See Boring Location Plan
San Antonio and Schertz, Texas

Location: Schertz: See Boring Location Plan

Sampling Date: 10/7/11

Coordinates: N29°34'0.7" W98°15'54.7"

Backfill: Cuttings

Soil Description	Depth (ft)	SN	WC	PL	LL	PI	PP	N	-200
FILL: LEAN CLAY (CL), stiff to very stiff, brown and tan		SS	9	16	37	21		22	
		SS	9					26	83
LEAN CLAY (CL) with calcareous deposits, hard, dark brown	5	T	13	19	40	21	4.5+		82
		T	13				4.5+		
LEAN CLAY (CL) with calcareous deposits, hard, tan		T	15	19	42	23	4.5+		86
	10	T	19				4.5+		
	15	T	21				4		

Borehole terminated at 15 feet

Groundwater Data:
During drilling: Not encountered

Field Drilling Data:
Logged By: R. Arizola
Driller: Eagle Drilling, Inc.
Equipment: Track-mounted drill rig
Dry-auger drilling: 0 ft to 15 ft
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

2010-895.GPJ 10/28/11 (BORING LOG SA11-01,ARIASSA10-01.GDT,LIBRARY2010.GLB)

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), 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.

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