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LAREDO, TEXAS 78041
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September 19, 2017

ADDENDUM #1

Ref: **Mirando City Elevated Water Storage Tank (TxCDBG #7216115)**

Bidders are advised of the following changes and/or clarifications to the plans and specifications:

DIVISION D - *Technical Specifications:*

GEOTECHICAL ENGINEERING (SOIL REPORT), Page D39

ADD the attached report in its entirety (33 pages).

BIDDERS MUST ACKNOWLEDGE RECEIPT OF THIS ADDENDUM NO. 1 ON THEIR BID PROPOSAL TO HAVE THEIR BIDS RECOGNIZED.

If you have any questions or concerns contact our office at (956) 724-3097.

Recommended by:

Wayne Nance, PE
Porras Nance Engineering

STATE OF TEXAS
WAYNE NANCE
87006
LICENSED
PROFESSIONAL ENGINEER
9.19.17

GEOTECHNICAL ENGINEERING STUDY

FOR

PROPOSED ELEVATED WATER STORAGE TANK
MIRANDO CITY, WEBB COUNTY, TEXAS

Raba Kistner
Consultants, Inc.
800 E. Hackberry
McAllen, TX 78501
www.rkci.com

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Project No. AMA17-036-00
September 18, 2017

Mr. Wayne Nance, P.E., RPLS
Porras Nance Engineering
304 E. Calton Road
Laredo, Texas 78041

Re: Geotechnical Engineering Study
Proposed 50,000 to 60,000 Gallon Capacity Elevated Water Storage Tank Structure
Along the East Side of Main Street
About 300 ft South of its Intersection with J W Edgar Street
Mirando City, Webb County, Texas

Dear Mr. Nance:

RABA KISTNER Consultants Inc. (RKCI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with **RKCI** Proposal No. PMA17-033-00, dated May 3, 2017. Written authorization for this study was received by our firm via electronic-mail attachment on Tuesday, August 15, 2017. The purpose of this study was to drill borings within the project site, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations for the proposed elevated water storage tank structure.

The following report contains our foundation recommendations and considerations based on our current understanding of finished floor elevation, design tolerances and structural loads. If any of these parameters change, then there may be alternatives for value engineering of the foundation system, and **RKCI** recommends that a meeting be held with Porras Nance Engineering (CLIENT) and design team to evaluate these alternatives.

We appreciate the opportunity to be of professional service to you on this project. Should you have any questions about the information presented in this report, please call. We look forward to assisting Porras Nance Engineering during the construction of the project by conducting the construction materials engineering and testing services (quality assurance program).

Very truly yours,

RABA KISTNER CONSULTANTS, INC.

Saul Cruz
Graduate Engineer

Katrin M. Leonard, P.E.
Associate

STATE OF TEXAS
KATRIN M. LEONARD
95810
LICENSED
PROFESSIONAL ENGINEER

Sept. 18, 2017

Attachments

SC/KML

Copies Submitted: Above (1)

GEOTECHNICAL ENGINEERING STUDY

For

PROPOSED 50,000 to 60,000 GALLON CAPACITY ELEVATED WATER STORAGE TANK
ALONG THE EAST SIDE OF MAIN STREET
ABOUT 300 FT SOUTH OF ITS INTERSECTION WITH J W EDGAR STREET
MIRANDO CITY, WEBB COUNTY, TEXAS

Prepared for

PORRAS NANCE ENGINEERING
Laredo, Texas

Prepared by

RABA KISTNER CONSULTANTS, INC.
McAllen, Texas

PROJECT NO. AMA17-036-00

September 18, 2017

RABAKISTNER

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ATTACHMENTS

INTRODUCTION

RABA KISTNER Consultants Inc. (RKCI) has completed the authorized subsurface exploration and foundation analysis for the proposed elevated water storage tank structure to be located along the east side of Main Street and about 300 ft south of its intersection with J W Edgar Street in Mirando City, Webb County, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations.

PROJECT DESCRIPTION

We understand that the project will consist of the design and construction of a 20-ft diameter and 110-ft tall, 50,000 to 60,000 gallon capacity, elevated, water storage tank structure. The proposed structure is planned to be located along the east side of Main Street and about 300 ft south of its intersection with J W Edgar Street in Mirando City, Webb County, Texas. The proposed water storage tank structure is expected to create heavy loads to be carried by the foundation system, which is anticipated to consist of a deep foundation system.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of South Texas and for the use of the Porras Nance Engineering (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods.

The recommendations submitted in this report are based on the data obtained from three borings drilled at this site, our understanding of the project information provided to us by others, and the assumption that site grading will result in only minor changes in the topography existing at the time of our study. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the subject site. No environmental opinions are presented in this report. **RKCI**'s scope of work does not include the investigation, detection, or design related to the prevention of any biological pollutants. The term “biological pollutants” includes, but is not limited to, mold, fungi, spores, bacteria, and viruses, and the byproduct of any such biological organisms.

If final grade elevations are significantly different from the grades existing at the time of our study (more than plus or minus 1 ft), our office should be informed about these changes. If needed and/or desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the subject site were evaluated by three borings drilled within the site, as shown in the following table:

Proposed Structure	Number of Borings	Depth, ft. *	Boring Identification
Center of the Tank	1	50	B-1
Perimeter of the Tank	2	25	B-2 and B-3

* below the ground surface elevations existing at the time of our study.

The borings (designated as "B-") were drilled on August 24, 2017, at the locations shown on the Boring Location Map, Figure 1. The boring locations are approximate and were located in the field by an **RKCI** representative based on the site plan titled "Tank Site Plan," dated July 21, 2017, and provided to our office by the CLIENT via electronic-mail attachment on August 15, 2017. The borings were drilled to the depths indicated in the previous table using a truck-mounted, rotary-drilling rig. The borings were conducted utilizing straight flight augers and were backfilled with the auger cuttings following completion of the drilling operations. During the drilling operations, Split-Spoon (with Standard Penetration Test, SPT) samples were collected.

The SPT samples were obtained in accordance with accepted standard practices and the penetration test results are presented as "blows per foot" on the boring logs. Representative portions of the samples were sealed in containers to reduce moisture loss, labeled, packaged, and transported to our laboratory for subsequent testing and classification.

In the laboratory, each sample was evaluated and visually classified by a member of our Geotechnical Engineering staff in general accordance with the Unified Soil Classification System (USCS). The geotechnical engineering properties of the strata were evaluated by the following laboratory tests: natural moisture content, Atterberg limits, a corrosivity (Including resistivity, pH, and sulfate and chloride content determinations) test, and percent passing a No. 200 sieve determinations.

With the exception of the laboratory corrosivity (pH, electrical resistivity, and sulfate and chloride content determinations), the results of the laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 4. A key to the classification of terms and symbols used on the logs is presented on Figure 5. The results of the laboratory and field testing are also tabulated on Figure 6 for ease of reference.

Standard penetration test results are noted as "blows per ft" on the boring logs and Figure 6, where "blows per ft" refers to the number of blows by a falling hammer required for 1 ft of penetration into the soil. Where hard and/or very dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the request of the CLIENT.

GENERAL SITE CONDITIONS

SITE DESCRIPTION

The proposed tank structure is planned to be located along the east side of Main Street and about 300 ft south of its intersection with J W Edgar Street in Mirando City, Webb County, Texas. At the time of our field activities, the project site can be described as a heavily-vegetated, uneven tract of land. In general, the topography at the subject site is relatively flat, with an estimated vertical relief of less than 3 ft across the site. Surface drainage is estimated to be poor. The subject site is bounded to the north by J W Edgar Street; to the east by Main Street; and to the south and west by undeveloped tracts of land.

GEOLOGY

A cursory review of the Geologic Atlas of Texas, (Laredo Sheet, dated 1976), published by the Bureau of Economic Geology at The University of Texas at Austin, indicates that the subject site appears to be located within the Catahoula and Frio Formation consisting of mudstone and clay deposits of the Miocene and Oligocene (Tertiary Period).

According to the Soil Survey of Webb County, Texas, published by the United States Department of Agriculture - Soil Conservation Service, in cooperation with the Texas Agricultural Experiment Station, the project site appears to be located within the Delmita-Randado-Cuevitas soil association consisting of moderately deep to very shallow, gently undulating, non-saline, sandy and loamy surface layer. The corresponding soil symbol appears to be JQD, Jimenez-Quemado complex.

SEISMIC COEFFICIENTS

Based upon a review of Section 1613 *Earthquake Loads* of the 2012 International Building Code (IBC), the following information has been summarized for seismic considerations associated with this site.

- Site Class Definition (Chapter 20 of the American Society of Civil Engineers [ASCE] 7): **Class D**. Based on the soil borings conducted for this investigation, the upper 100 feet of soil may be may be characterized as a stiff soil profile.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United Stated of a 0.2-Second, Spectral Response Acceleration (5% of Critical Damping) (Figure 1613.3.1(1)): $\mathbf{S_s = 0.057g}$. Note that the value taken from Figure 1613.3.1(1) is based on Site Class B and is adjusted as per 1613.3.3 below.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of a 1-Second, Spectral Response Acceleration (5% of

Critical Damping) (Figure 1613.3.1(2)): $\mathbf{S_1 = 0.018g}$. Note that the value taken from Figure 1613.3.1(2) is based on Site Class B and is adjusted as per 1613.3.3 below.

- Value of Site Coefficient (Table 1613.3.3 (1)): *from worksheet* $\mathbf{F_a = 1.6}$.
- Value of Site Coefficient (Table 1613.3.3 (2)): *from worksheet* $\mathbf{F_v = 2.4}$.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec., adjusted based on equation 16-37: *from worksheet* $\mathbf{S_{ms} = 0.092g}$.
- 1 sec., adjusted based on equation 16-38: *from worksheet* $\mathbf{S_{m1} = 0.044g}$.

The Design Spectral Response Acceleration Parameters are as follows:

- 0.2 sec., based on equation 16-39: *from worksheet* $\mathbf{S_{DS} = 0.061g}$.
- 1 sec., based on equation 16-40: *from worksheet* $\mathbf{S_{D1} = 0.030g}$.

Based on the parameters listed above, the critical nature of the structure, Tables 1613.3.5(1) and 1613.3.5(2), and calculations performed using a Java program titled, "Seismic Hazard Curves and Uniform Hazard Response Spectra" published by the United States Geological Survey (USGS) website, the Seismic Design Category for both short period and 1 second response accelerations is **A**. As part of the assumptions required to complete the calculations, a Risk Category of **II** was selected.

STRATIGRAPHY

The subsurface stratigraphy at this site can be described by three generalized strata. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. For purposes of this report, we have designated the subsurface strata as Stratum I through Stratum III. The lines designating the interfaces between strata on the boring logs represent approximate boundaries. Transitions between strata may be gradual.

Stratum I consist of brown to light brown, medium dense to dense, silty sand soils and clayey silty sand soils with gravel. This layer was encountered in the borings from the ground surface elevations existing at the time of our study extending down to a depth of about 7 ft. Moisture contents ranging from about 1 to 6 percent were measured for this layer. This stratum is classified as non-plastic to marginally plastic, with measured plasticity indices ranging from 1 to 8 percent and a plasticity index that could not be determined. Percent passing a No. 200 sieve test demonstrates percent fines ranging from about 12 to 43 percent. SPT N-values ranging from 11 blows to 35 blows per foot of penetration were measured for this stratum. These soils are classified as SM soils and/or SC-SM in general accordance with the USCS.

Stratum II consists of light brown to whitish-brown, dense to very dense, sandstone. This layer was encountered in the borings beneath the Stratum I soils extending down to depths ranging from about 13 ft to 17 ft. Moisture contents were measured to range from about 2 to 9 percent for this layer. This stratum is classified as marginally plastic, with a single measured plasticity index of about 5 percent. Two percent passing a No. 200 sieve test demonstrates percent fines of about 21 and 24 percent. SPT N-values ranging from about 40 blows to more than 50 blows per foot of penetration were measured for this stratum.

Stratum III consists of light olive-brown to light grayish-brown, very stiff to hard, lean clay soils and fat clay soils with black ferrous stains, gypsum crystals, and calcareous nodules. This layer was encountered in the borings beneath the Stratum II soils extending down to at least the termination depths of these borings. Moisture contents were measured to range from about 19 to 30 percent for this layer. This stratum is classified as plastic, with two measured plasticity indices of about 27 and 32 percent. SPT N-values ranging from about 22 blows to more than 50 blows per foot of penetration were measured for this stratum. These soils are classified as CL soils and/or CH in general accordance with the USCS.

CORROSIVITY POTENTIAL

The corrosivity characteristics of the upper subsurface strata were preliminarily evaluated using pH test, resistivity test, and sulfate and chloride content tests. These tests were conducted on a single composite soil specimen obtained near Boring B-1, from a depth of about 2 ft below the ground surface elevation existing at the time of our study. Results are summarized in the following table:

Composite Sample Identification	Depth, ft	Electrical Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (mg/kg)
Near Boring B-1	2	1,692.8	8.1	120	8.1

The results of the laboratory electrical resistivity test conducted on the composite soil sample indicate a highly corrosive potential for corrosion to buried metals. The laboratory chloride content test result indicated a marginal corrosive potential for corrosion to buried metals. According to the American Concrete Institute (ACI) document titled “Guide to Durable Concrete” (ACI 201), concrete usually provides protection against rusting of adequately embedded steel because of the highly alkaline environment of the Portland cement paste. The adequacy of that protection is dependent upon the amount of the concrete cover, the quality of the concrete, the details of the construction, and the degree of exposure to chlorides from concrete-making components and external sources. It is recommended that no chloride-containing admixtures be utilized in the concrete mixes for this project. Consideration should also be given to implementing corrosion protection measures for buried metals in direct contact with the subsurface strata, such as coating metal structural elements, pipings, and/or fittings. The pH laboratory test results indicate that the surficial native soils are moderately alkaline. On the basis of the laboratory sulfate content test results, the shallow subsurface strata appear to result in a mild exposure of concrete to corrosion. According to these laboratory test results, the native shallow subsurface strata result in a Class 0 severity of potential exposure of concrete to corrosion. The ACI 201 Guide indicates no special cementitious material requirements for sulfate resistance for a Class 0 exposure.

GROUNDWATER

Groundwater was not observed in the borings either during or immediately upon completion of the field drilling activities. The boreholes were left open for the duration of the field exploration phase to allow monitoring of water levels, and remained dry. However, it is possible for groundwater to exist beneath this site on a transient basis following periods of precipitation. Fluctuations in groundwater levels occur due to variations in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

Based on the findings in the borings and on our experience in this region, we believe that groundwater seepage encountered during site earthwork activities may be controlled using temporary earthen berms and conventional sump-and-pump dewatering methods. For deep foundation excavations, this could include the use of temporary casing to reduce groundwater seepage and sloughing of the subsurface soils.

FOUNDATION ANALYSIS

EXPANSIVE SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at this site were estimated for slab-on-grade construction using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). PVR values on the order of less than 1 inch were estimated for the stratigraphic conditions encountered in the borings. The PVR value was estimated using a surcharge load of 1 pound per square inch (psi) for the concrete slab and dry moisture conditions within the regional zone of seasonal moisture variation.

The TxDOT method of estimating expansive, soil-related movements is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content. If desired, other methods of estimating expansive, soil-related movements are available, such as estimations based on swell tests and/or soil-suction analyses. However, the performance of these tests and the detailed analysis of expansive, soil-related movements were beyond the scope of the current study. It should also be noted that actual movements can exceed the estimated PVR values due to isolated changes in moisture content (such as due to leaks, landscape watering, etc.) or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

Drainage Considerations Considerations of surface and subsurface drainage may be crucial to construction and adequate foundation performance of the soil-supported structure. Several surface and subsurface drainage design features and construction precautions can be used to limit problems associated with fill moisture. These features and precautions may include, but are not limited to, the following:

- Installing berms or swales on the uphill side of the construction area to divert surface runoff away from the excavation/fill area during construction;

- Sloping of the top of the subgrade with a minimum downward slope of 1.5 percent out to the base of a dewatering trench located beyond the structure's perimeter;
- Sloping the surface of the fill during construction to promote runoff of rain water to drainage features until the final lift is placed;
- Sloping of a final, well-maintained, impervious clay or pavement surface (downward away from the proposed structure) over the select fill material and any perimeter drain extending beyond the structure lines, with a minimum gradient of 6 in. in 5 ft;
- Constructing final surface drainage patterns to prevent ponding and limit surface water infiltration at and around the structure perimeter;
- Locating the water-bearing utilities, roof drainage outlets, and irrigation spray heads outside of the select fill and perimeter drain boundaries; and
- Raising the elevation of the ground level floor slab.

Details relative to the extent and implementation of these considerations must be evaluated on a project-specific basis by all members of the project design team. Many variables that influence fill drainage considerations may depend on factors that are not fully developed in the early stages of design. For this reason, drainage of the fill should be given consideration at the earliest possible stages of the project.

FOUNDATION RECOMMENDATIONS

SITE GRADING

Site grading plans can result in changes in almost all aspects of foundation recommendations. We have prepared the foundation recommendations based on the ground surface elevations and the stratigraphic conditions encountered in the borings at the time of our study. If site grading plans differ from the grades existing at the time of our study by more than plus or minus 1 ft, we must be retained to review the site grading plans prior to bidding the project for construction. This will enable us to provide input for any changes in our original recommendations, which may be required as a result of site grading operations or other considerations.

DEEP FOUNDATIONS

Drilled, straight-shaft piers should be considered to support the proposed water storage tank structure. We recommend that piers extend to a minimum depth of 20 ft below the ground surface elevations existing at the time of our study or below final ground surface, whichever is greater. The piers may be designed as both end bearing units and as friction units utilizing the maximum allowable end-bearing pressures and the allowable side shear resistance values tabulated in the following tables:

Approximate Depth Below the Ground Surface Elevation Existing at the Time of our Study (ft)	Allowable End-Bearing Pressure (ksf)
20	8.0
25	10.5
30 and deeper	14.5

Depth Range Below the Ground Surface Elevation Existing at the Time of our Study (ft)	Allowable Side Shear Resistance (ksf)
0 to 10	0
10 to 20	0.6
20 to 30	0.8
30 to 45	1.2

If designed as skin friction units, the side shear resistance value shown previously should be used for the portion of the shaft extending below a depth of 10 ft. To proportion the drilled piers for axial compression, the side shear resistance should be neglected along the portion of the shaft located one shaft diameter from the bottom of the pier. The allowable values for end bearing and side shear resistance were evaluated using factors of safety of 3 and 2, respectively, with respect to the measured soil shear strength. Based on the 50-ft maximum depth of exploration, pier depths should not exceed a depth of 45 ft below the ground surface elevation existing at the time of our study.

It is recommended that the pier shafts be a minimum of 24 inches in diameter to facilitate reinforcing steel placement and shaft observation prior to placing concrete.

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the sustained compressive load carried by the pier. We recommend that each pier be reinforced to withstand this net force or an amount equal to 1/2 percent of the cross-sectional area of the shaft, whichever is greater.

The surficial soils encountered in the boring exhibit relatively low shrink/swell potential within the active zone. Therefore, potential uplift forces acting on the pier shafts due to expansive, soil-related movements are negligible at this site.

PIER SPACING

If more than one pier or a pile-group is utilized to support the water tank structure, we recommend that the drilled, straight-shaft piers be spaced at a distance of at least three shaft diameters on-center. Such spacing will not require a reduction in the load carrying capacity of the individual piers.

If design and/or construction restraints require that piers be spaced closer than the above recommended spacing, **RKCI** must be retained to re-evaluate the allowable bearing capacity presented above for the individual piers. Reductions in load-carrying capacities may be required depending upon individual loading and spacing conditions.

LATERAL RESISTANCE

Resistance to lateral loads and the expected pier behavior under the applied loading conditions will depend not only on subsurface conditions, but also on loading conditions, the pier size(s), and the engineering properties of the piers. Once the structural loading is known, as well as the pier size(s) and properties, the piers should be analyzed to determine the resulting lateral deflections, maximum bending moments, and ultimate bending moments. This type of analysis is typically performed utilizing a computer analysis program and usually requires a trial and error procedure to appropriately size the piers and meet project tolerances.

To assist the structural engineer in this procedure, we are providing the following subsurface parameters for use in analysis. These parameters are in accordance with the input requirements of one of the more commonly used computer programs for laterally-loaded piles, the "L-Pile Plus" program. If a different program is used for analysis, different parameters may be required and different limitations may be required than what was assumed in selecting the parameters given on the following page. Thus, if a program other than "L-Pile Plus" is used, **RKCI** must be notified of the analysis method and the required soil parameters, so that we can review and revise our recommendations if required. The soil-related parameters required for input into the "L-Pile Plus" program are summarized in following table.

Soil Type	**Approximate Depth Range (ft) ***	**c, tsf**	**ϕ (°)**	ε_{50}	**k_s, (pci)**	**k_c, (pci)**	**γ, (pcf)**
Sand Soils (Above the Groundwater Table)	0 to 5	0	29	--	90	90	110
Sand Soils (Above the Groundwater Table)	5 to 15	0	31	--	90	90	115
Clay Soils (Above the Groundwater Table)	15 to 25	1.3	0	0.005	1,000	400	115
Clay Soils (Above the Groundwater Table)	25 to 30	1.6	0	0.005	1,000	400	120
Clay Soils (Above the Groundwater Table)	30 to 45	2.4	0	0.004	2,000	800	125

* Below the ground surface elevations existing at the time of our study.

Where:

c = undrained shear strength
ϕ = angle of internal friction
ε_{50} = strain at 50 percent
k_s = horizontal modulus of subgrade reaction (static)
k_c = horizontal modulus of subgrade reaction (cyclic)
γ = density (effective unit weight)

The values presented above for subgrade modulus are based on recommended values for the "L-Pile Plus" computer program for the strength of the subsurface conditions encountered in the borings, and are not necessarily based on laboratory test results.

The parameters presented above **do not** include factors of safety. Consequently, it is recommended that a factor of safety of at least 2 be introduced to the analysis by doubling the applied lateral loads and moments.

FOUNDATION CONSTRUCTION CONSIDERATIONS

SITE DRAINAGE

Drainage is an important key to the successful performance of any foundation. Good surface drainage should be established prior to and maintained after construction to help prevent water from ponding within or adjacent to the structure's foundation and to facilitate rapid drainage away from the structure's foundation.

Other drainage and subsurface drainage issues are discussed in the *Expansive Soil-Related Movements* section of this report.

SITE PREPARATION

The water tank structure area and all areas to support select fill should be stripped of all vegetation, and/or organic topsoil down to a minimum depth of 8 inches and extending a minimum of 5 ft beyond the structure's footprint area.

Beyond the structure pad footprint, existing utilities and trenches that are not removed should be properly abandoned. This would include grouting abandoned pipes and sealing off granular fill in trenches to prevent the migration and seepage of water into the foundation areas of the new structures.

Exposed subgrades should be thoroughly proofrolled in order to locate and densify any weak, compressible zones. A minimum of 5 passes of a fully-loaded dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or his/her representative to document subgrade conditions and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with a suitable, compacted select fill in accordance with the recommendations presented under the *Select Fill* subsection of this section of the report. Proofrolling operations and any excavation/backfill activities should be observed by **RKCI** representatives to document subgrade preparation.

Upon completion of the proofrolling operations and just prior to fill placement, the exposed subgrades should be moisture-conditioned by scarifying to a minimum depth of 6 inches and recompacting to a minimum of 95 percent of the maximum dry density as determined from the American Society for Testing and Materials (ASTM) D1557, Compaction Test. The moisture content of the subgrade should be maintained within the range of two percentage points below the optimum moisture content to two percentage points above the optimum moisture content until permanently covered.

SELECT FILL

Materials used as select fill for final site grading preferably should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2014 Standard Specification for Construction and Maintenance of Highways, Streets, and Bridges, Item 247, Flexible Base, Type A through Type E, Grades 1, 2, 3, and 5.

Alternatively, the following soils, as classified according to the USCS, may be considered satisfactory for use as select fill materials at this site: SC, GC, and combinations of these soils. In addition to the USCS classification, alternative select fill materials shall have a maximum liquid limit of 40 percent, a plasticity index between 7 and 18 percent, and a maximum particle size not exceeding 4 inches or one-half the loose lift thickness, whichever is smaller. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a minimum rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with pit-run materials.

If the above listed alternative materials are being considered for bidding purposes, the materials should be submitted to the Geotechnical Engineer for pre-approval a minimum of 10 working days or more prior to the bid date. Failure to do so will be the responsibility of the General Contractor. The General Contractor will also be responsible for ensuring that the properties of all delivered alternate select fill materials are similar to those of the pre-approved submittal. It should also be noted that when using alternative fill materials, difficulties may be experienced with respect to moisture control during and subsequent to fill placement, as well as with erosion, particularly when exposed to inclement weather. This may result in sloughing of beam trenches and/or pumping of the fill materials.

Soils classified as CH, MH, ML, SM, GM, OH, OL, and Pt under the USCS and not meeting the alternative select fill material requirements, are **not** considered suitable for use as select fill materials at this site.

Select fill should be placed in loose lifts **not** exceeding 8 in. in thickness and compacted to at least 95 percent of maximum dry density as determined by ASTM D1557. The moisture content of the subgrade should be maintained within the range of two percentage points below the optimum moisture content to two percentage points above the optimum moisture content until permanently covered.

The select fill should be properly compacted in accordance with these recommendations and tested by **RKCI** personnel for compaction as specified.

DRILLED PIERS

Drilled pier excavations must be examined by an **RKCI** representative who is familiar with the geotechnical aspects of the subsurface stratigraphy, the structural configuration, foundation design details, and assumptions prior to placing concrete. This is to observe that:

- The shaft has been excavated to the specified dimensions at the correct depth established by the previously mentioned criteria;
- The shaft has been drilled plumb within specified tolerances along its total length; and
- Excessive cuttings, buildup and soft, compressible materials have been removed from the bottom of the excavation.

Drilled pier excavation observations should be scheduled with the Geotechnical Engineer a minimum of 48 hours prior to pier drilling. Failure to do so will be the responsibility of the General Contractor.

REINFORCEMENT AND CONCRETE PLACEMENT

Reinforcing steel should be checked for size and placement prior to concrete placement. Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the moisture content or the state of stress of the foundation materials. Concrete should not be placed in the pier excavations without the approval of the Engineer. No foundation element should be left open overnight without concreting.

TEMPORARY CASING

Groundwater was not observed during drilling operations. However, groundwater seepage and/or side sloughing may be encountered during pier construction at this site depending on climatic conditions. Therefore, we recommend that the bid documents require the foundation contractor to specify unit costs for different lengths of casing and/or slurry drilling techniques which may be required.

EXCAVATION SLOPING AND BENCHING

Excavations that extend to or below a depth of 5 ft below construction grade shall require the General Contractor to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, are beyond the scope of the current study. Any such designs and safety plans shall be developed in accordance with current Occupational Safety and Health Administration (OSHA) guidelines and other applicable industry standards.

EXCAVATION EQUIPMENT

SPT N-values in excess of 50 blows per foot were recorded at this site at a depth of about 7-1/2 ft below the ground surface elevation existing at the time of our study. Thus, very dense and/or hard subsurface conditions should be anticipated during excavation activities at the site. The boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that General Contractors and their subcontractors interested in bidding on the work perform their own tests in the form of test pits and/or test piers to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

UTILITIES

Utilities which project through slab-on-grade, slab-on-fill, or any other rigid unit should be designed with either some degree of flexibility or with sleeves. Such design features will help reduce the risk of damage to the utility lines as vertical movements occur. These types of slabs will generally be constructed as monolithic, grid type beam and slab foundations.

Our experience indicates that significant settlement of backfill can occur in utility trenches, particularly when trenches are deep, when backfill materials are placed in thick lifts with insufficient compaction, and when water can access and infiltrate the trench backfill materials. The potential for water to access the backfill is increased where water can infiltrate flexible base materials due to insufficient penetration of curbs, and at sites where geological features can influence water migration into utility trenches (such as fractures within a rock mass or at contacts between rock and clay formations). It is our belief that another factor which can significantly impact settlement is the migration of fines within the backfill into the open voids in the underlying free-draining bedding material.

To reduce the potential for settlement in utility trenches, we recommend that consideration be given to the following:

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and all backfilling procedures should be tested and documented; and
- Locating the water-bearing utilities, roof drainage outlets and irrigation spray heads outside of the select fill and perimeter drain boundaries.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, **RKCI** is retained to perform construction observation and testing services during the construction of the project. This is because:

- **RKCI** has an intimate understanding of the geotechnical engineering report's findings and recommendations. **RKCI** understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- **RKCI** knows what subsurface conditions are anticipated at the site.

- **RKCI** is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical workscope. This enables **RKCI** to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.
- **RKCI** has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors' work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.
- **RKCI** cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that **RKCI** and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. **RKCI** looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.

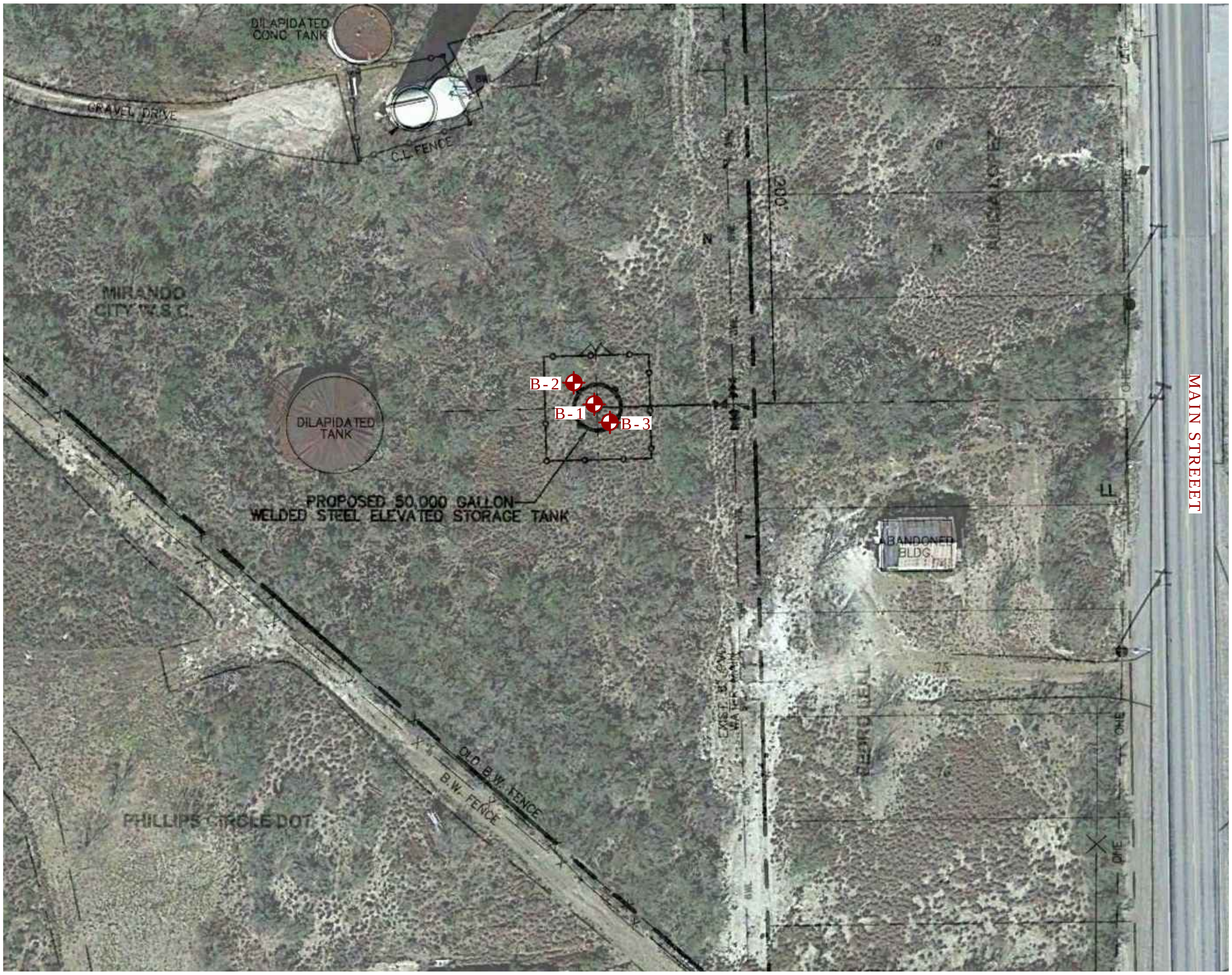
* * * * * * * * * * * * * * * * * *

The following figures are attached and complete this report:

Figure 1	Boring Location Map
Figures 2 through 4	Logs of Borings
Figure 5	Key to Terms and Symbols
Figure 6	Results of Soil Analyses

ATTACHMENTS

O:\Active Projects\McAllen\2017\AMA17\AMA17-036-00 Prop. Mirando City Water Storage Tank-Webb\Drawings\Figure 1.dwg



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BORING LOCATION MAP PROPOSED MIRANDO CITY WATER STORAGE TANK

NEAR THE SWC OF MAIN STREET
AND J W EDGAR STREET
MIRANDO CITY, WEBB COUNTY, TEXAS

REVISIONS:

No.	DATE	DESCRIPTION

PROJECT No.:	AMA17-036-00
ISSUE DATE:	09-12-17
DRAWN BY:	DV
CHECKED BY:	SC
REVIEWED BY:	KML
FIGURE:	1

LOG OF BORING NO. B-1

Mirando City - Elevated Water Storage Tank
Along the East Side of Main Street
Mirando City, Webb County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger **LOCATION:** See Figure 1

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

DEPTH, FT	DESCRIPTION OF MATERIAL (SURFACE ELEVATION: Existing Grade, ft)	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	**SILTY SAND (SM)** medium dense to dense, brown to light brown, with gravel	19		8	
		11			12
5		34		NP	
	SANDSTONE very dense, whitish-brown	50/6"			
10		50/ 11"			24
	LEAN CLAY (CL) hard to very stiff, light olive-brown, with calcareous nodules				
15		50/9"		27	
20		22			
25	**FAT CLAY (CH)** hard, light grayish-brown, with black ferrous stains	34			

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	50.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	AMA17-036-00
DATE DRILLED:	8/24/2017	**DATE MEASURED:**	8/24/2017	**FIGURE:**	2a

LOG OF BORING NO. B-1

Mirando City - Elevated Water Storage Tank
Along the East Side of Main Street
Mirando City, Webb County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

DEPTH, FT	DESCRIPTION OF MATERIAL SURFACE ELEVATION: Existing Grade, ft	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	FAT CLAY (CH) hard, light grayish-brown, with black ferrous stains *(continued)*	49		32	
35		50/ 11"			
40		39			
45	- with gypsum crystals below a depth of about 45 ft	43			
		50			
50	Boring terminated at a depth of about 50 ft. NOTES: Upon completion of the drilling operations, the boring was observed dry.				
55					

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 8/24/2017
DEPTH TO WATER: DRY
DATE MEASURED: 8/24/2017
PROJ. No.: AMA17-036-00
FIGURE: 2b

LOG OF BORING NO. B-2

Mirando City - Elevated Water Storage Tank
Along the East Side of Main Street
Mirando City, Webb County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

SURFACE ELEVATION: Existing Grade, ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	**SILTY SAND (SM)** medium dense, brown to light brown	14			33
		17		1	
5		30			43
	SANDSTONE very dense to dense to very dense, whitish-brown	50/6"		5	
10		40			
15		REF/ 1"			
	FAT CLAY (CH) very stiff, light olive-brown, with black ferrous stains				
20		27			
25		28			

Boring terminated at a depth of about 25 ft.

NOTES:
Upon completion of the drilling operations, the boring was observed dry.

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	25.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	AMA17-036-00
DATE DRILLED:	8/24/2017	**DATE MEASURED:**	8/24/2017	**FIGURE:**	3

LOG OF BORING NO. B-3

Mirando City - Elevated Water Storage Tank
Along the East Side of Main Street
Mirando City, Webb County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

SURFACE ELEVATION: Existing Grade, ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	**CLAYEY SILTY SAND (SC-SM)** medium dense to dense, brown, with gravel	22		7	
		30			17
5		35		7	
7	**SANDSTONE** dense, light brown	44			
10		40			21
15	**FAT CLAY (CH)** hard, light olive-brown, with calcareous nodules	REF/ 2"			
20		33			
		39			
25	Boring terminated at a depth of about 25 ft.				

NOTES:
Upon completion of the drilling operations, the boring was observed dry.

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	25.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	AMA17-036-00
DATE DRILLED:	8/24/2017	**DATE MEASURED:**	8/24/2017	**FIGURE:**	4

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

- CALCAREOUS
- CALICHE
- CLAY
- CLAYEY
- GRAVEL
- GRAVELLY
- PEAT
- SAND
- SANDY
- SILT
- SILTY
- FILL

ROCK TERMS

- CHALK
- CLAYSTONE
- CLAY-SHALE
- CONGLOMERATE
- DOLOMITE
- IGNEOUS
- LIMESTONE
- MARL
- METAMORPHIC
- SANDSTONE
- SHALE
- SILTSTONE

OTHER

- ASPHALT
- BASE
- CONCRETE/CEMENT
- BRICKS / PAVERS
- WASTE
- NO INFORMATION

WELL CONSTRUCTION AND PLUGGING MATERIALS

- BLANK PIPE
- SCREEN
- BENTONITE
- CEMENT GROUT
- BENTONITE & CUTTINGS
- CONCRETE/CEMENT
- CUTTINGS
- GRAVEL
- SAND
- VOLCLAY

SAMPLE TYPES

- AIR ROTARY
- GRAB SAMPLE
- CORE
- GEOPROBE SAMPLER
- ROTOSONIC -DAMAGED
- MUD ROTARY
- NO RECOVERY
- NX CORE
- PITCHER
- ROTOSONIC -INTACT
- SHELBY TUBE
- SPLIT BARREL
- SPLIT SPOON
- TEXAS CONE PENETROMETER
- DISTURBED

STRENGTH TEST TYPES

- POCKET PENETROMETER
- TORVANE
- UNCONFINED COMPRESSION
- TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
- TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. AMA17-036-00

RABA KISTNER

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

RELATIVE DENSITY

Penetration Resistance Blows per ft	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

COHESIVE STRENGTH

Resistance Blows per ft	Consistency	Cohesion TSF
0 - 2	Very Soft	0 - 0.125
2 - 4	Soft	0.125 - 0.25
4 - 8	Firm	0.25 - 0.5
8 - 15	Stiff	0.5 - 1.0
15 - 30	Very Stiff	1.0 - 2.0
> 30	Hard	> 2.0

PLASTICITY

Plasticity Index	Degree of Plasticity
0 - 5	None
5 - 10	Low
10 - 20	Moderate
20 - 40	Plastic
> 40	Highly Plastic

ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
EI = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Mirando City - Elevated Water Storage Tank
Along the East Side of Main Street
Mirando City, Webb County, Texas

FILE NAME: AMA17-036-00.GPJ 9/12/2017

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-1	0.0 to 1.5	19	3	31	23	8	SM				
	2.5 to 4.0	11	3						12		
	5.0 to 6.5	34	1	CNBD	CNBD	NP	SM				
	7.5 to 8.5	50/6"	7								
	10.0 to 11.4	50/ 11"	3						24		
	15.0 to 16.3	50/9"	19	49	22	27	CL				
	20.0 to 21.5	22	24								
	25.0 to 26.5	34	28								
	30.0 to 31.5	49	19	53	21	32	CH				
	35.0 to 36.4	50/ 11"	22								
	40.0 to 41.5	39	23								
	45.0 to 46.5	43	23								
	48.5 to 50.0	50	23								
B-2	0.0 to 1.5	14	3						33		
	2.5 to 4.0	17	2	18	17	1	SM				
	5.0 to 6.5	30	2						43		
	7.5 to 8.5	50/6"	2	21	16	5	SC-SM				
	10.0 to 11.5	40	2								
	15.0 to 15.1	REF/ 1"	9								
	20.0 to 21.5	27	21								
	23.5 to 25.0	28	27								
B-3	0.0 to 1.5	22	4	24	17	7	SC-SM				
	2.5 to 4.0	30	6						17		
	5.0 to 6.5	35	4	24	17	7	SC-SM				
	7.5 to 9.0	44	3								
	10.0 to 11.5	40	6						21		
	15.0 to 15.2	REF/ 2"	21								
	20.0 to 21.5	33	29								
	23.5 to 25.0	39	30								

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial CNBD = Cound Not Be Determined NP = Non-Plastic

Important Information about This

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 constructor — 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 this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this 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.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many 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;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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 geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this 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 geotechnical-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 confirmation-dependent recommendations included in your report. *Confirmation-dependent 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 confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront 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. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical 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 Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors 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 constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors 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 constructors fail to 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental 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 environmental 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, many 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 GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

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