RABA KISTNER

a kiwa company

GEOTECHNICAL ENGINEERING STUDY

FOR

PROPOSED OCELOT ELECTRICAL SUBSTATION
BROWNSVILLE, CAMERON COUNTY, TEXAS

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Project No. ABA24-011-00 (Revised)
October 28, 2024

Mr. Jesus Alfaro, SR/WA, R/W-NAC
Real Estate Manager
Brownsville Public Utilities Board (Brownsville PUB)
1425 Robinhood Drive
Brownsville, Texas 78521

RE: Geotechnical Engineering Study
Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Approximately 0.15 Mile East of Its Intersection with Simmons Boulevard
Brownsville, Cameron County, Texas

Dear Mr. Alfaro:

RABA KISTNER, Inc. (RKI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with **RKI** Proposal No. PBA24-014-00 (Revised), dated April 26, 2024. Please note that the original of our proposal was revised in order to include a modification in our scope of work, based on the electronic-mail attachment received by our office from you on Thursday, April 28, 2024. Written authorization to proceed with this study was received by our office via electronic-mail attachment on Monday, June 17, 2024, by means of the Professional Engineering Services Contract between Brownsville PUB (CLIENT) and **RKI**, dated May 29, 2024. The purpose of this study was to drill borings within the subject site, to perform laboratory testing on selected samples to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation and pavement recommendations and construction guidelines for the proposed electrical substation.

The following report contains our foundation and pavement recommendations and considerations based on our current understanding of the design tolerances, and structural and pavement loads. If any of these parameters change, then there may be alternatives for value engineering of the foundation and pavement systems, and **RKI** recommends that a meeting be held with the Brownsville PUB and the 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 Brownsville PUB during the construction of the project by conducting the construction materials engineering and testing services (quality assurance program).

Very truly yours,

RABA KISTNER, INC.

Saul Cruz, P.E.
Graduate Engineer

Katrin M. Leonard, P.E.
Vice President

STATE OF TEXAS
KATRIN M. LEONARD
95810
LICENSED
PROFESSIONAL ENGINEER

Oct. 28, 2024

Attachments

SC/KML

Copies Submitted: Above (1)

GEOTECHNICAL ENGINEERING STUDY

For

PROPOSED OCELOT ELECTRICAL SUBSTATION
ALONG THE SOUTH SIDE OF E. MORRISON ROAD
APPROXIMATELY 0.15 MILE EAST OF ITS INTERSECTION WITH SIMMONS BOULEVARD
BROWNSVILLE, CAMERON COUNTY, TEXAS

Prepared for

BROWNSVILLE PUBLIC UTILITIES BOARD
Brownsville, Texas

Prepared by

RABA KISTNER, INC.
McAllen, Texas

PROJECT NO. ABA24-011-00 (Revised)

October 28, 2024

RABAKISTNER

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Important Information About Your Geotechnical Engineering Report

INTRODUCTION

RABA KISTNER, Inc. (RKI) has completed the authorized subsurface exploration for the proposed Brownsville Public Utilities Board (Brownsville PUB) Ocelot electrical substation to be located along the south side of W. Morrison Road and approximately 0.15 mile east of its intersection with Simmons Boulevard in Brownsville, Cameron County, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for site preparation, and foundation design and construction considerations, as well as pavement design and construction guidelines.

PROJECT DESCRIPTION

We understand that the proposed project consists of the design and construction of an electrical ground grid system, including the following structures:

- Transformer with Secondary Containment:
  - Transformer's weight is estimated to be about 100,000 lbs, and to be approximately 12-ft long by 6-ft wide.
- Concrete Masonry Unit (CMU) or Precast Concrete Control Building:
  - Control building is planned to be single-story and approximately 36-ft long by 30-ft wide.
- Transmission Terminal / H-frame Structures
- Miscellaneous Switch, Bus Support Structures, and Electrical Equipment (including light standards)

The subject site is located along the south side of W. Morrison Road and approximately 0.15 mile east of its intersection with Simmons Boulevard in Brownsville, Cameron County, Texas. The site can be described as an undeveloped, recently cleared 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 visually estimated to be poor. The project site is bounded to the north by W. Morrison Road; to the east by an undeveloped tract of land; to the south by an existing unpaved, access road; and to the west by an existing asphalt-paved, trail.

Foundation loads for the proposed structures have not been provided at this time. The proposed structures are expected to create relatively light to moderate loads to be carried by the foundation systems, which are anticipated to consist of shallow and/or deep foundation systems. The pavement systems are anticipated to consist of a combination of both flexible (asphalt) and rigid (concrete) pavements.

For purposes of this geotechnical engineering report, the finished grade elevation (FGE) of the proposed structures were assumed to be about 12 inches (1 ft) above the ground surface elevation existing at the time of our study, since no site grading information was provided to us at the time of the preparation of this report.

PREVIOUS STUDY

RKI has previously performed a subsurface Reconnaissance Study within the subject site, which included a total of two borings located within the footprint area of the proposed electrical substation (**RKI** Project No. ABA22-013-00, dated June 20, 2022). The results of this study are on file in our office. Our previous data was utilized as supplementary information in the preparation of this report.

LIMITATIONS

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

The recommendations submitted in this report are based on the data obtained from six borings drilled at this site, our understanding of the project information provided to us by the CLIENT, 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 subject site. The nature and extent of variations across the subject 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 site. No environmental opinions are presented in this report. **RKI**'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 assumed in this report, 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 six borings drilled within the site, as shown in the following table.

Proposed Structure	Number of Borings	Depth, ft. *	Boring Identification
Ocelot Substation	2	50	B-1 and B-2
	2	70	B-3 and B-4
Pavement Areas	2	10	P-1 and P-2

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

The borings (designated as "B-" and "P-") were drilled on August 15 through August 20, 2024, at the locations shown on the Boring Location Map, Figure 1. The boring locations are approximate and were located in the field by an **RKI** representative based on the aerial map provided to us by the CLIENT via electronic-mail attachment on Wednesday, July 31, 2024. The borings were drilled to the depths shown in the previous table, below the ground surface elevations existing at the time of our study using a truck-mounted, rotary-drilling rig. The borings were drilled utilizing straight flight augers in combination with mud rotary drilling techniques and were backfilled with the auger cuttings following completion of the drilling operations. During the drilling operations, Split-Spoon (with Standard Penetration Test, SPT) and Shelby-Tube (ST) samples were collected.

The SPT and ST 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, unconfined compressive strength tests, dry unit weight determinations, a corrosivity test (including electrical resistivity, pH, chloride and sulfate content determinations), and percent passing a No. 200 sieve determinations.

With the exception of the corrosivity test and moisture-density relationship (Proctor) results, the results of the field and laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 7. A key to the classification of terms and symbols used on the logs is presented on Figure 8. The results of the laboratory and field testing are also tabulated on Figure 9 for ease of reference. Further, the result of the moisture-density relationship (Proctor) laboratory test of the subgrade soils is presented on Figure 11.

SPT results (N-values) are noted as "blows per ft" on the boring logs and on Figure 9, where "blows per ft" refers to the number of blows by a falling 140-lb (pound) hammer required for 1 ft of penetration into the subsurface materials.

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

GENERAL SITE CONDITIONS

GEOLOGY

Based on a cursory review of the Geologic Atlas of Texas (McAllen-Brownsville 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 Alluvium (floodplain) deposits consisting of clays, silts, sands, and gravel deposits of the Quaternary epoch (Holocene period).

According to the Soil Survey of Cameron 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 Rio Grande-Matamoros association consisting of nearly level to gently sloping, well-drained and moderately well-drained, silt loams and silty clays. The corresponding soil symbols appear to be CF, Cameron silty clay, and CH, Chargo silty clay.

FROST DEPTH

Based on the geographic location of the site, the subsurface conditions encountered in our borings, and the severity and duration of cold weather in our region, it is our judgment that the potential for frost may be considered to be negligible at this site.

SEISMIC COEFFICIENTS

Based upon a review of Section 1613 *Earthquake Loads* of the 2015 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 and our experience in the area, 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): $\mathbf{S_s = 0.036g}$.
- 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): $\mathbf{S_1 = 0.013g}$.
- Value of Site Coefficient: $\mathbf{F_a = 1.6}$.
- Value of Site Coefficient: $\mathbf{F_v = 2.4}$.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec., adjusted $\mathbf{S_{ms} = 0.057g}$.
- 1 sec., adjusted $\mathbf{S_{m1} = 0.032g}$.

The Design Spectral Response Acceleration Parameters are as follows:

- 0.2 sec.: **S_{DS} = 0.038g**.
- 1 sec.: **S_{D1} = 0.021g**.

STRATIGRAPHY

On the basis of the borings drilled for this site, the subsurface stratigraphy can be described as intermixed layers of moderately plastic to plastic, fine-grained soils with various amount of sand; and visually marginally plastic, course-grained soils. Each stratum has been designated by grouping materials that possess similar physical and engineering characteristics. The boring logs should be consulted for more specific stratigraphic information. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by **RKI** in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

The boring logs and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time may result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted.

GROUNDWATER

Groundwater was encountered in Borings B-1 through B-4 at depths ranging from about 5 ft to 7 ft below the ground surface elevation existing at the time of our study. In Borings P-1 and P-2, groundwater was not observed during or immediately upon completion of the drilling operations. The groundwater level in the borings may not have stabilized, particularly in less permeable cohesive soil, prior to backfilling. Hence, there is a potential for groundwater to exist beneath this site at shallower depths on a transient basis following periods of precipitation. Fluctuations in groundwater levels occur due to variations in rainfall, surface water run-off, recharge, or other factors not evident at the time of exploration. In addition, groundwater may potentially occur as a perched condition at the planned fill and soil interface, or within permeable soils or backfill. The construction process itself may also cause variations in the groundwater level.

FOUNDATION RECOMMENDATIONS AND CONSIDERATIONS

Site grading plans can result in changes in almost all aspects of foundation recommendations. We have prepared the foundation recommendations based on the assumption that the FGE of the proposed structures will be about 12 inches (1 ft) above the ground surface elevation existing at the time of our study and the stratigraphic conditions encountered in the borings at the time of our study. If site grading plans differ from the assumed finished grades, we must be retained to review the site grading plans prior to bidding the project for construction. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

Site features that will influence the geotechnical approach to the proposed project include:

- Potential to encounter relatively shallow groundwater seepage during excavation and site grading operations;
- In-situ electrical resistivity testing;
- Corrosivity characteristics of the upper subsurface soils;
- Potential for sensitive soils that are easily disturbed to construction traffic;
- Presence of expansive soil and potential for soil-related movements;
- Depth of planned fill for site improvements and potential for soil-related movements; and
- Potential for light to moderate foundation loads.

The following foundation systems are available to support the proposed additions:

- Shallow foundation systems with a fill-supported concrete floor slab or mat foundations; and
- Deep foundation systems, consisting of drilled, straight shaft piers. (Due to the depth to groundwater, underrreamed piers will be difficult to construction and are not recommended).

Please note that the foundation capacities presented herein are based on the Allowable Stress Design methodology.

GROUNDWATER SEEPAGE AND DEWATERING

As discussed herein, groundwater was encountered at depths ranging from about 5 ft to 7 ft below the ground surface elevation during our fieldwork. However, there is a possibility for groundwater to exist at shallower depths than those encountered in our borings (see section titled *Groundwater)*. Fluctuations in groundwater levels and groundwater seepage should be anticipated during construction. The contractor should be made aware and ready to handle/intercept potential water for anticipated excavations.

Where excavations extend into the underlying soil layers groundwater seepage should be anticipated. Raising the finished grade and performing the excavations during the drier season (such as summertime) will aid in reducing the potential for groundwater seepage, but will not eliminate the risk. For relatively shallow excavations, French drains or trench drains, which are discharged by gravity or sumps, may be required to intercept groundwater seepage so that the excavations are not submerged under water. For deep foundation excavations, this could include the use of slurry drilling and/or temporary casing (including overdrive techniques) to reduce groundwater seepage and sloughing of the soils.

The General Contractor should be prepared to control excess water encountered in the excavations due to perched water, seepage from natural or constructed interfaces (such as but not limited to fill and natural soil interface, utility backfills, other), and/or rainfall. Proper construction procedures and equipment will be critical for proper performance of the dewatered excavations. Additionally, protection of personnel entering the excavations and providing a dry, stable subgrade upon which to construct foundations will be crucial.

IN-SITU ELECTRICAL RESISTIVITY TESTING

In accordance with the approved work scope, in-situ electrical resistivity testing was performed by **RKI** on August 8, 2024, within the subject site. Testing of subsurface soils to a depth of approximately 20 feet below ground surface was conducted at one specified test station designated as “ERT-1” as depicted on the Boring Location Map, Figure 1. The following paragraphs provide a description of resistivity test methods and results.

Description of Resistivity Test Methods

Resistivity testing was conducted utilizing the Wenner Method, in general accordance with prescribed procedures set forth by the American Society for Testing and Materials (ASTM) in *Standard Guide for Using the Direct Current Resistivity Method for Subsurface Site Characterization, ASTM D6431-18 (ASTM 2018)*. As the specified methodology set forth in this reference source is consistent with *Standard Test Method for Field Measurement of Soil Resistivity Utilizing the Wenner Four-Electrode Method, ASTM G57-20 (ASTM, 2020)*, both reference sources were considered for the collection of the resistivity data as further described below.

In accordance with data collection requirements stipulated for the project, a series of resistivity measurements were obtained at station ERT-1. In order to evaluate potential anisotropy associated with ground resistivity measurements and to provide a check on the quality of data obtained, data was collected along orthogonal arrays at the test station oriented roughly north-to-south (N-S) and east-to-west (E-W). The resistivity arrays were located in areas that were not influenced by the presence of overhead transmission lines, underlying utilities/piping, significant ground disturbance, or other known influences that would negatively impact the results of the surveys.

Due to the fact that soil resistivity values can be affected by changes in moisture and temperature the weather conditions leading up to and during the survey were taken into account. The ambient temperature at the time the survey was conducted ranged from approximately 90° to 95° F. The weather conditions were clear and sunny. A review of published data available from the National Weather Service indicate that the West End Station (nearest weather station to the project area) did not receive rain within the 7 days preceding soil resistivity testing. Soil moisture conditions at the ground surface were observed to be slightly moist at the time testing was conducted.

Resistivity measurements were obtained according to the following procedure:

- Once the test station location was established, non-conducting tape measures were placed along the ground surface from the central point of the array and utilized to determine proper electrode placement. Before obtaining resistivity measurements, the instrument was set up and tested with a 19-ohm test resistor provided by the manufacturer, then tested again through the connecting wires to ensure proper conductivity to the electrodes.

- Four stainless steel electrodes were utilized for measurements and driven to depths of approximately 6 to 8 inches into the surface. Electrodes were configured along a straight line at the following spacings as set forth in the project specifications, which coincide roughly with the depth of measurement: 1.5, 5, 7.5, 10, 15, and 20 feet.

- Ground resistance measurements were obtained utilizing a portable field resistivity meter manufactured by L and R Instruments, Inc. (i.e., Super MiniRes Earth Resistivity and IP Instrument). The instrument utilizes rugged, solid-state components for all transmitter and receiver functions. A key characteristic of the instrument is the receiver architecture, which relies upon a "synchronous" detection method, allowing stable readings to be taken in relatively noisy environments. The Super MiniRes transmits at up to 10 milliamps.

- Utilizing the Super MiniRes, an electrical current (I) on the order of 10 milliamps peak amplitude was impressed between the two outer "current" electrodes and the potential (V) measured between the two inner "potential" electrodes. Soil resistance values (R) for each electrode configuration (A-spacing) were obtained by the internal calculation (V/I = R) provided directly by the Super MiniRes.

  In accordance with *ASTM G 57-20*, apparent soil resistivity (ρ) values were calculated as follows:

  $\rho = 2\pi aR$, which provides the resistivity of the soil at depth (a). As described above, the depth-of-measurement coincides with electrode spacing for the Wenner 4-Pin Method.

  Apparent resistivity values calculated utilizing depth-of-measurement values are reported in units of ohm-cm. These values were subsequently calculated according to the following formula: $\rho = 191.5(aR)$.

Discussion of Test Results

Resistance values (ohms) measured directly in the field and calculated apparent resistivity values (ohm-cm) for specified electrode-spacing configurations are presented on Figure 10. Based upon review of in-situ electrical resistivity test data and comparison with the drilling logs for geotechnical borings B-1 to B-4, P-1, and P-2, **RKI** offers the following observations:

- Based on our interpretation of resistivity data, we can offer the following apparent correlations between recorded measurements and subsurface soil conditions at the site:

    - According to the geotechnical boring logs, the surface soils at the site (Stratum I) consist predominately of brown fat/lean clay a depth of 20.0 feet. At the designated test stations, moderately low resistivity values were obtained from brown clay soils at a depth of 1.5 feet. These values generally range from 174.4 to 189.3 ohm-m and are indicative of relatively unconsolidated moist surface soil conditions.

- At depths of 5.0 and 7.5 feet, resistivity values generally range from 120.6 to 136.4 ohm-cm, indicative of the transition from the upper, relatively unconsolidated surface soils to the more moist and consolidated subsurface soils.

- At depths of 10.0 to 20.0 feet, resistivity values generally range from 99.6 to 118.7 ohm-cm, indicative of saturated clay soils. This is consistent with the geotechnical boring logs, which reported shallow groundwater at the project site ranging from 5.0 to 7.0 feet below the ground surface.

- Based on in-situ resistivity testing, it is recommended that the project team considers the range of soil resistivity values for the upper 20-foot depth interval of 100 to 190 ohm-cm. These values fall within the published range of values generally associated with wet to moist, clay soils.

- Generally speaking, resistivity data obtained along the orthogonal array for test station ERT-1 compares favorably and indicates relatively consistent values for all A-spacing measurements to a maximum depth of 20 feet below ground surface. As greater variations are expected with deep measurements, which is inherent to the testing process, potential anisotropy in soil conditions should be considered as part of the electrical design process for the proposed Ocelot Electrical Substation project.

CORROSIVITY POTENTIAL

The measurable soil properties that indicate the corrosion potential for steel in contact with soil are soil pH, chloride and sulfate ion concentration, and soil electrical resistivity. Corrosion of steel is most likely to occur in environments that have chloride ions, even in low concentrations, very low or very high pH, and/or low resistivity. The following information is being provided for preliminary planning purposes.

The following table presents general guidelines concerning the corrosion potential of a soil as a function of chloride and sulfate ion concentration, pH, and electrical resistivity. Each of the columns on this table should be used independently of the others when evaluating corrosion potential. For instance, it is not necessary to have an electrical resistivity of less than 1,000 ohm-cm and a pH of less than 4.0 to indicate a *Very High* potential for corrosion.

Soil Corrosion Potential			
Electrical Resistivity (Ohm-cm)(1)	**Chloride Ion Content (ppm)**	**pH(2)**	**Corrosion Potential**
< 1,000			Very High
1,000 - 3,000	> 500	<4 or >10	High
3,000 - 10,000	< 500		Moderate
> 10,000		>4 or < 10	Mild

(1)After Roberge, 2000
(2)After DOE-HDBK-1015/1-93

The potential corrosivity characteristics of the upper subsurface soils within the subject site were screened using a pH test, electrical resistivity test, sulfate and chloride content determination. These tests were conducted on composite soil samples obtained from the structures' footprint area from an approximate depth of about 2 ft below the existing ground surface elevation. Results are summarized in the following table:

Sample Location	Approximate Depth, (ft)*	Electrical Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (mg/kg)
B-2	0 to 2	640	8.9	140	323

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

The result of the laboratory tests conducted on the composite soil sample indicate a very high 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.

We recommend 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 soil, such as coating metal structural elements, pipings, and/or fittings.

DEGRADATION OF CONCRETE

The degradation of concrete is caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds which cause cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete. Sulfate concentrations in soil can be used to evaluate the need for protection of concrete based on the general guidelines shown in the following table.

Sulfate Attack Potential	
Sulfate Ion Concentration, ppm or mg/kg	**Aggressiveness[1]**
>20,000	Very Severe
2,000 to 20,000	Severe
1,000 to 2,000	Moderate
< 1,000	Negligible

[1]ACI 318-05/ACI 318R-05

Sulfate testing was completed on a sample taken from Boring B-2. The results showed a sulfate content of less than 1,000 ppm. The general guidelines from the above table indicate the soils have a "*Negligible*" potential for attacking concrete. Based on testing of the measured soil sulfate concentration for the soils at the site, it is our opinion there are no restrictions on the cementitious materials types.

SOFT SENSITIVE SOILS

Site grading may potentially expose soft, wet, sensitive fine-grained soils that require a special grading approach to establish a stable subgrade. When these sensitive soils are encountered, the geotechnical engineer should be contacted to observe the exposed subgrade. Proof-rolling and moisture conditioning of exposed subgrades may be waived if, in the opinion of the geotechnical engineer, it could result in disturbance to an otherwise stable subgrade. When these sensitive soils are encountered, we recommend excavating the subgrade areas using a trackhoe equipped with a toothless bucket working above the proposed subgrade. **Grading/construction equipment or foot traffic should be prohibited from trafficking on the potentially sensitive subgrade.** If the exposed subgrade is found to be overly soft, the subgrade can be improved by placing an 18-inch thick layer of compacted, crushed rock to provide a stable working platform.

For the crushed rock working platform, the initial lift of backfill should consist of an 18-inch thick loose lift of 2-inch-minus crushed rock. This procedure may be further enhanced by placing a geogrid on the subgrade before placing the crushed rock, provided that the geogrid will not interfere with foundation or utility construction. The crushed rock should be pushed into the excavation with the equipment working on top of the rock platform. After this initial layer is placed, the crushed rock should be thoroughly surface-compacted with a self-propelled vibratory roller. Subsequent lifts of finer-gradation crushed rock or select structural fill can be placed conventionally. As an alternative, consideration can be given to pushing “bull rock” into the soft subgrade until subgrade yielding stops; however, this approach may be restricted in areas that will not interfere with foundation or utility construction. Furthermore, deep soil mixing or other ground improvements may be used to improve the exposed subgrades.

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 2 inches were estimated for the stratigraphic conditions encountered in the borings. The PVR values were 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 (estimated active zone of 8 ft).

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....) or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

PVR REDUCTION RECOMMENDATIONS

As previously mentioned, for purposes of this geotechnical engineering report, the FGE of the proposed structures were assumed to be about 12 inches (1 ft) above the ground surface elevation existing at the time of our study, since no site grading information was provided to us at the time of the preparation of this report.

To reduce expansive, soil-related movements in at-grade construction beneath the structures' footprint areas to about 1 inch, we recommend to remove the upper 2 ft (24 inches) of the existing subgrade soils, and to replace them with properly-compacted, suitable, select fill materials within the proposed structures' footprint areas up to their FGE, which was assumed to be about 12 inches (1 ft) above the ground surface elevation existing at the time of our study (i.e. total of about 36 inches [3 ft] of select fill placement).

Further, if the foundation systems to support the proposed structures are planned to be founded at a lower or same elevation that the recommended overexcavation elevation, then the ground improvement to reduce expansive, soil-related movements in at-grade construction to about 1 inch may be omitted. Keep in mind that the estimated PVR values are computed based on the recommendations for the selection and placement of suitable, select fill materials which are addressed in the Foundation Construction Considerations section of the report.

Drainage Considerations

When overexcavation and select fill replacement is selected as a method to reduce the potential for expansive, soil-related movements at any site, considerations of surface and subsurface drainage may be crucial to construction and adequate foundation performance of the soil-supported structures. Filling excavations in relatively impervious clay soils with relatively pervious select fill material creates a "bathtub" beneath the structures, which can result in ponding or trapped water within the fill unless good surface and subsurface drainage is provided.

Water entering the fill surface during construction or entering the fill exposed beyond the building lines after construction may create problems with fill moisture control during compaction and increased access for moisture to the underlying expansive clays both during and after construction.

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 areas to divert surface runoff away from the excavation/fill areas 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 structures' perimeters;
- 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 structures) over the select fill material and any perimeter drain extending beyond the building 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 structures' perimeters;
- 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 slabs.

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.

SHALLOW FOUNDATION

The proposed structures may be founded on conventional spread and/or continuous footing foundations in conjunction with a fill-supported, interior concrete floor slab provided that the shallow foundation type can be designed to withstand the anticipated soil-related movements (see the *Expansive Soil-Related Movements* section of this report) and the estimated foundations loads without impairing either the structural or the operational performance of the proposed structures.

Allowable Soil-Bearing Capacity

Shallow foundations bearing on properly-compacted, suitable, select fill materials following the correct implementation of the ground improvement presented in the subsection titled *PVR Reduction Recommendations* presented in the *Expansive, Soil Related Movements* section of this report may be proportioned using the design parameters tabulated on the following table.

Shallow Foundation Design Parameters

Minimum depth below FGE:	24 in.
Minimum beam width:	12 in.
Maximum allowable soil-bearing pressure for continuous footings – grade beams:	1,500 psf
Maximum allowable soil-bearing pressure for spread footings – widened beams:	1,800 psf

Where psf = pounds per square feet

The maximum allowable soil-bearing pressures presented previously will provide a factor of safety of about 3 provided that the subgrade is prepared in accordance with the recommendations outlined in the *Site Preparation* subsection of the *Foundation Construction Considerations* section of this report, and that the ground improvement procedure included in the *PVR Reduction Recommendations* subsection of the

Expansive Soil-Related Movements section of this report is correctly implemented within the building footprint. Provided that the site improvement procedure recommended in this report is properly implemented, then it is anticipated that total settlements will be in the order of about 1 inch. Differential settlements typically are estimated to be about one-half the total estimated settlement for most subsurface conditions

Furthermore, the design parameters presented on the previous table are contingent upon the fill materials being selected and placed in accordance with the recommendations presented in the *Select Fill* subsection of the *Foundation Construction Considerations* section of this report. Should select fill selection and placement differ from the recommendations presented herein, **RKI** should be informed of the deviations in order to reevaluate our recommendations and design criteria.

Wire Reinforcement Institute (WRI) Criteria

Beam and slab-on-fill foundations are sometimes designed using criteria developed by the WRI. On the basis of the subsurface stratigraphy encountered, a general effective plasticity index for the proposed structures' foundations of 30 percent and a climatic rating (C_w) of 15 should be utilized for the design of the proposed structures' foundations.

MAT FOUNDATIONS

On the basis of the subsurface conditions encountered at the time of our field drilling activities, our field and laboratory testing, and our engineering analyses, the recommended maximum allowable soil-bearing pressure for the proposed structures is as shown on the following table:

Structure	Approximate Bearing Depth *	Maximum Allowable Soil-Bearing Pressure, psf
Electrical Grid Structures	2 ft	1,500

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

The maximum allowable soil-bearing pressure presented previously will provide a factor of safety of 3 with respect to the measured soil shear strength.

Considerations for Shallow Foundation Excavations

Shallow foundation excavations should be observed by the Geotechnical Engineer or their representative prior to placement of reinforcing steel and concrete. This is necessary to document that the bearing soils at the bottom of the excavations are similar to those encountered in the borings and that excessive soft materials and water are not present in the excavations. If soft or yielding soils are encountered in the foundation excavations, they should be removed and replaced with a compacted non-expansive fill material or lean concrete up to the design foundation bearing elevation.

Disturbance from foot traffic and from the accumulation of excess water can result in losses in bearing capacity and increased settlement. If inclement weather is anticipated at the time construction, consideration should be given to protecting the bottoms of beam trenches by placing a thin mud mat (layer of flowable fill or lean concrete) at the bottom of trenches immediately following excavation. This will reduce disturbance from foot traffic and will impede the infiltration of surface water. All necessary precautions should be implemented to protect open excavations from the accumulation of surface water runoff and rain.

AREA FLATWORK

It should be noted that ground-supported flatwork such as walkways, driveways, courtyards, sidewalks, etc., will be subject to the same magnitude of potential soil-related movements as discussed previously (see *Expansive, Soil-Related Movements* subsection of the *Foundation Analyses* section of this report) for this site. Thus, where these types of elements abut rigid building foundations or isolated structures, differential movements should be anticipated. As a minimum, we recommend that flexible joints be provided where such elements abut the main structures to allow for differential movement at these locations. Where the potential for differential movement is objectionable, it may be beneficial to consider methods of reducing anticipated movements to match the adjacent structures' performance.

DRILLED, STRAIGHT-SHAFT PIERS

Drilled, straight-shaft piers may also be considered to support the proposed structures. We recommend extending the piers to a minimum depth of 12 ft below the ground surface elevation existing at the time of our study or below final grade, 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 Range (ft) *	Maximum Allowable End-Bearing Pressure (ksf)
12 to 29	1.50
30 to 49	4.50
50 to 60	6.25

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

Approximate Depth Range (ft) *	Allowable Side Shear Resistance (ksf)
0 to 10	Neglect
10 to 15	0.25
15 to 30	0.35
30 to 60	0.45

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

The side shear resistance values presented above 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. Based on the 70-ft maximum depth of exploration, pier depths should not exceed a depth of 60 ft below the ground surface elevations existing at the time of our study.

Due to the presence of groundwater, the use of slurry drilling techniques and/or temporary casing should be anticipated for the construction of the drilled piers. Consequently, slightly deeper piers may be required to accommodate for the casing procedures.

Expansive Soil Uplift on Pier Shafts

The pier shafts will be subjected to potential uplift forces if the surrounding expansive soils within the active zone are subjected to alternate drying and wetting conditions. The maximum potential uplift force acting on the shafts may be estimated by:

$$F_u = 22\ D$$

Where: F_u = uplift force in kips; and
D = diameter of the shaft in feet.

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.

Allowable Uplift Resistance

Resistance to uplift forces exerted on the drilled piers will be provided by the sustained compressive axial force (dead load) plus the allowable uplift resistance provided by the soil. The resistance provided by the soil depends on the bearing capacity of the soils located above the pier shaft and below the active zone. The allowable uplift resistance values provided by the soils at this site are tabulated on the following table. These values were evaluated using a factor of safety of 2.

Approximate Depth Range (ft) *	Allowable Uplift Resistance (ksf)
0 to 10	Neglect
10 to 15	0.15
15 to 30	0.20
30 to 60	0.30

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

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the uplift resistive force and 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 percent of the cross-sectional area of the shaft, whichever is greater. Splices in vertical reinforcement should be staggered.

PIER SPACING

Where possible, we recommend that the drilled, straight-shaft piers be spaced at a distance of at least one 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, **RKI** 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, and the engineering properties of the pier. The pier 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 pier 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 in the following table. Thus, if a program other than “L-Pile Plus” is used, **RKI** 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 the following table:

Soil Type	Approximate Depth Range (ft) *	c, tsf	ϕ (°)	ε_{50}	k_s, (pci)	k_c, (pci)	γ, (pcf)
Clay Soils (Above the Groundwater Table)	0 to 5	0.3	-	0.010	100	--	100
Clay Soils (Below the Groundwater Table)	5 to 15	0.5	-	0.007	100	--	47
Clay Soils (Below the Groundwater Table)	15 to 30	0.8	-	0.005	500	200	52
Clay Soils (Below the Groundwater Table)	30 to 60	0.9	-	0.005	500	200	52

* Below the ground surface elevation 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 in the previous tables for subgrade modulus and the strain at 50% 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 in the previous tables **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.

CONSIDERATIONS FOR DRILLED PIERS

Drilled pier excavations must be examined by an **RKI** 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 observed in Borings B-1 through B-6 at depths of about 5 ft to 7 ft below the ground surface elevations existing at the time of our study. Groundwater seepage and/or side sloughing will be encountered at the time of construction, depending on climatic conditions prevalent at the time of construction. 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 will be required.

GRADE BEAMS

For the structures being considered, we recommend that the grade beams interconnecting the piers be ground-supported on properly-compacted, suitable select fill materials, but designed to span the piers.

FLOOR SLABS

For the structures being considered, the floor slabs may be ground supported on properly-compacted, suitable, select fill materials, provided that the anticipated movements discussed under the *Expansive Soil-Related Movements* section of this report will not impair the performance of the floor, frame, or roof systems.

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 structures' foundations and to facilitate rapid drainage away from the structures' foundations. Failure to provide positive drainage away from the structures can result in localized differential vertical movements in soil the supported foundation and floor slabs.

Current ordinances, in compliance with the Americans with Disabilities Act (ADA), may dictate maximum slopes for walks and drives around and into new building. These slope requirements can result in drainage problems for the ground-supported building. We recommend that, on all sides of the proposed structures foundation, the maximum permissible slope be provided away from the proposed structures. Also to help control drainage in the vicinity of the structures, we recommend that roof/gutter downspouts and landscaping irrigation systems not be located adjacent to the structures' foundations. Where a select fill overbuild is provided outside of the floor slab/foundation footprints, the surface should be sealed with an impermeable layer (pavement or clay cap) to reduce infiltration of both irrigation and surface waters.

Materials used as fill material for the construction of the clay cap should consist of clay soils. All material used in the clay cap must be free of roots, vegetation, and other organic or degradable materials. The following soils, as classified according to the USCS, are preferred for use as clay cap: CL and CH. In addition to the USCS classification, clay cap soils shall have a minimum liquid limit of 30 percent, with plasticity indices ranging from 20 to 35 percent, and a minimum amount passing a No. 200 sieve of 85 percent.

Careful consideration should also be given to the location of water bearing utilities, as well as to provisions for drainage in the event of leaks in water bearing utilities. All leaks should be immediately repaired.

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

SITE PREPARATION

The structures' areas 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. Further, we recommend that the ground improvement procedure presented in the *PVR Reduction Recommendations* section of this report be implemented in order to reduce expansive, soil-related movements in at-grade construction to about 1 inch.

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 their representative to document subgrade conditions and preparation. Weak or soft areas identified during proofrolling activities should be treated with hydrated lime or Portland cement or removed and replaced with suitable, compacted select fill in accordance with the recommendations presented under the *Select Fill* subsection of this section of the report. If the treatment option is selected, the weak or soft areas may be mixed with hydrated lime or Portland cement down to a minimum depth of 8 inches in order to aid in drying the soils and develop a firm working surface. Proofrolling operations and any excavation/backfill activities should be observed by **RKI** 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 8 in. and recompacting to a minimum of 98 percent of the maximum dry density as determined from the American Society for Testing and Materials (ASTM) D698, Compaction Test. The moisture content of the subgrades should be maintained within the optimum moisture content to three 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, CL, 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.

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 98 percent of the maximum dry density as determined by ASTM D698. The moisture content of the fill 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 the final lift of fill is permanently covered.

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

GENERAL FILL

Areas requiring fill that do not have requirements for reducing the expansive, soil-related movements, such as green spaces and general areas, can utilize on-site soils. These materials should have maximum particle sizes of 4 inches and placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 95 percent of maximum density as determined by ASTM D698. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction.

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, is 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

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 earthwork and utility contractors interested in bidding on the work perform their own tests in the form of test pits and/or test piers determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

WET WEATHER CONDITIONS

Earthwork contractors should be made aware of the moisture sensitivity of the near surface soils and potential compaction difficulties. If construction is undertaken during wet weather conditions, the surficial soils may become saturated, soft, and unworkable. Drainage trenches within the soils to be excavated, reworked and/or recompacted may be required. Additionally, subgrade stabilization techniques, such as chemical (cement, flyash or hydrated lime) treatment, may be required to provide a more weather-resistant working surface during pad construction. Therefore, we recommend that consideration be given to construction during the dryer months. Alternatively, the contractor should protect all exposed areas once topsoil has been stripped, as well as provide positive drainage during earthwork operations.

UTILITIES

Utilities which project through slab-on-grade, slab-on-fill, “floating” floor slabs, 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.

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. 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:

- 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 backfilling procedures should be tested and documented.
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N or CONTECH C-Drain Geocomposite) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.
- Locating the water-bearing utilities, roof drainage outlets and irrigation spray heads outside of the select fill and perimeter drain boundaries.

PAVEMENT RECOMMENDATIONS

Recommendations for both flexible and rigid pavements for a 20-year design period are presented in this report. The CLIENT may select either pavement type depending on the performance criteria established for the proposed project. In general, flexible pavement systems have a lower initial construction cost as compared to rigid pavements. However, maintenance requirements over the life of the pavement are typically much greater for flexible pavements. This typically requires regularly scheduled observation and repair, as well as overlays and/or other pavement rehabilitation at approximately one-half to two-thirds of the design life. Rigid pavements are generally more "forgiving", and therefore tend to be more durable and require less maintenance after construction.

For either pavement type, drainage conditions will have a significant impact on long-term performance, particularly where permeable base materials are utilized in the pavement section. Drainage considerations are discussed in more detail in a subsequent section of this report.

SWELL/HEAVE POTENTIAL

It should be understood that pavement sections in expansive soil environments can develop longitudinal cracking along unprotected pavement edges. In the semi-arid climate of the project site, this condition typically occurs along the unprotected edges of pavements where the adjoining grounds are not developed.

The longitudinal cracking generally occurs between 2 to 4 feet inside of and parallel to the unprotected edges of the pavement. The occurrence of these cracks can be more prevalent in the absence of lateral restraint and embankments. Differential drying and shrinkage of the highly expansive soil subgrade between the exposed pavement edge and that beneath the pavement section commonly causes the cracking. This problem can best be addressed by providing either a horizontal or vertical moisture barrier at the unprotected pavement edge.

A horizontal barrier is commonly in the form of a paved shoulder extending 8 feet or greater beyond the edge of the pavement. Other methods of shoulder treatment, such as using geofabrics beyond the edge of the roadway, are sometimes used in an effort to help reduce longitudinal cracking. Although this alternative does not eliminate the longitudinal cracking phenomenon, the location of the cracking is transferred to the shoulder rather than within the traffic lane.

Vertical barriers installed along the unprotected edges of pavements are also effective in preventing non-uniform drying and shrinkage of the subgrade soils. These barriers are typically in the form of a vertical moisture barrier/membrane extending a minimum of 6 feet below the top of the subgrade at the pavement edge. Both types of barriers must be sealed at the edge of the pavement to prevent a crack that would facilitate the drying of the subgrade soils.

A more economical alternative, which will not limit the shrinkage of the underlying subgrade soils but may help reduce the occurrence of longitudinal cracking, is the use of a geogrid base reinforcement in the pavement section. Geogrid gives the pavement section a tensile strength component that is not otherwise inherent in a typical flexible base pavement section. Another consideration is to treat the subgrade soil with lime.

SUBGRADE CONDITIONS

A single generalized subgrade condition has been assumed for this site. The predominant subgrade soils used in developing the pavement sections for this project are the plastic, clay soils. On the basis of our past experience with similar subsurface conditions in this area, a design California Bearing Ratio (CBR) value of 2 was assigned to evaluate the pavement components. This design CBR value assumes that the subgrade soils will be prepared in accordance with the recommendations stated in the *Subgrade Preparation* subsection of the *Pavement Construction Guidelines* section of this report.

LIME TREATMENT OF SUBGRADE

In general, the subgrade soils at this site are plastic in nature and can be difficult to work with, particularly during periods of inclement weather. The performance of the subgrade soils may be improved by treating the upper 8 inches with hydrated lime. A sufficient quantity of hydrated lime should be mixed with the subgrade soils to decrease the plasticity index of the soil-lime mixture to 18 percent or less and to increase the pH of the soil-lime mixture to at least 12.4. For estimating purposes, we recommend that a minimum of 3 percent lime by weight be considered for lime treatment. For construction purposes, we recommend that the percent of hydrated lime treatment be determined by appropriate laboratory testing at the time of construction.

Based on a recently reported adverse reaction to lime addition in certain sulfate-containing soils, it is strongly recommended to perform additional laboratory testing to determine the concentration of soluble sulfates in the subgrade soils. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure.

DESIGN INFORMATION

The following recommendations for the pavement sections are based on our past experience with similar subgrade soils; assumed traffic loadings; an assumed CBR value for the subgrade soils; and design procedures by the American Association of State Highway and Transportation Officials (AASHTO). The pavement design and analyses performed are based directly on the 1993 and 1997 editions of the "Guide for the Design of Pavement Structure" by AASHTO.

The pavement systems for the proposed electrical substation can be divided into two general areas, each with different loading conditions and performance criteria. These areas are:

- Automobile drives and parking lots (light vehicular traffic); and
- Truck driveways and drive-in lanes (heavy vehicular traffic).

For a 20-year design period, Equivalent Single Axle Loads (ESAL's) were estimated for an assumed traffic loading of 1 tractor-trailer truck per day for the light duty traffic areas. This corresponds to about 17,500 ESAL's. For the heavy duty traffic areas, ESAL's were assumed to be a traffic loading of 5 tractor-trailer trucks per day. This corresponds to about 87,000 ESAL's. It is recommended that the project Civil Engineer review the above mentioned assumed level of traffic and design period to ensure that they are appropriate for the intended use of the proposed electrical substation.

FLEXIBLE PAVEMENTS

The following flexible pavement sections are available for this site, and other sections may be considered upon request:

Pavement Area	Option	LTS (in.)	FBM (in.)	CLS (in.)	HMAC (in.)
Light Vehicular Traffic Areas	I	8	8	--	2
Heavy Vehicular Traffic Areas	I	12	10	--	2
	II	12	--	14	--

Where: LTS = Lime-Treated Subgrade
FBM = Flexible Base Material
CLS = Crushed Limestone Base Material
HMAC = Hot-Mix Asphaltic Concrete Surface Course

RIGID PAVEMENTS

The following rigid pavement section is available for this site:

Pavement Area	Lime-Treated Subgrade (in.)	Reinforced Concrete (in.)
Light Vehicular Traffic Areas	8	5-1/2
Heavy Vehicular Traffic Areas	12	6

It is recommended that the concrete pavements be reinforced with reinforcing steel bars. As a minimum, the reinforcing bars should be No. 3 reinforcing bars spaced at about 15 in. on center in both directions (depending upon slab dimensions). The concrete reinforcing should be placed approximately 1/3 the slab thickness below the surface, but not less than 2 in. The reinforcing steel should not extend across construction or expansion joints.

Joints in concrete pavements aid in the construction and control the location and magnitude of cracks. Where practical, lay out the construction, expansion, control, and sawed joints to form square panels, but not to exceed American Concrete Institute (ACI) 302.69 Code recommendations. The ratio of slab length-to-width should not exceed 1.25. Recommended joint spacings are 15 ft longitudinal and 15 ft transverse.

All control joints should be formed or sawed to a depth of at least 1/4 the thickness of the concrete slab. Sawing of control joints should begin as soon as the concrete will not ravel, generally the day after placement. Control joints may be hand formed or formed by using a premolded fill. We recommend that all longitudinal and transverse construction joints be dowelled to promote load transfer.

If possible, the pavement should develop a minimum slope of 0.015 ft/ft to provide surface drainage. Reinforced concrete pavement should cure a minimum of 7 days before allowing any traffic.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

Areas to support pavements should be stripped of all vegetation and/or organic topsoil down to a minimum depth of 8 inches and extend a minimum of 2 ft beyond the pavement perimeters. Upon completion of site stripping activities, the exposed subgrade should be thoroughly proofrolled in accordance with the *Site Preparation* subsection recommendations provided in the *Foundation Construction Considerations* section of this report. Likewise, upon completion of the proofrolling activities and just prior to select fill placement, the exposed subgrade should be scarified and recompacted as recommended in such subsection.

PAVEMENT DRAINAGE CONSIDERATIONS

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

1) Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.
2) Final site grading should eliminate isolated depressions adjacent to curbs which may allow surface water to pond and infiltrate into the underlying soils. Curbs should completely penetrate flexible base materials and should be installed to sufficient depth to reduce infiltration of water beneath the curbs.
3) Pavement surfaces should be maintained to help reduce surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

ON-SITE SOILS

The pavement recommendations presented in this report were prepared assuming that on-site soils will be used for site grading in the proposed pavement areas. If used, we recommend that on-site soils be placed in loose lifts not exceeding 8 in. in thickness and compacted to a minimum of 98 percent of the maximum dry density as determined from ASTM D698. 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. We recommend that on-site sand fill materials be free of roots, vegetation, and/or other organic or degradable material. We also recommend that the maximum particle size not exceed 4 in. or one half the lift thickness, whichever is smaller.

SELECT FILL

If implemented, select fill materials utilized for achieving finished subgrade elevations in pavement areas should be in accordance with the *Select Fill* subsection recommendations provided in the *Foundation Construction Considerations* section of this report.

LIME TREATMENT OF SUBGRADE

Lime treatment of the subgrade soils should be in accordance with the TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 260, Lime Treatment (Road Mixed). Lime-treated subgrade soils should be compacted to a minimum of 95 percent of the maximum dry density at a moisture content within the range of optimum moisture content to three percentage points above the optimum moisture content as determined by ASTM D1557.

FLEXIBLE BASE COURSE

The flexible base course should consist of material conforming to TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, Item 247, Flexible Base, Type A through Type E, Grades 1, 2, 3, and 5.

The flexible base course should be placed in lifts with a maximum compacted thickness of 8 in. and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557. The moisture content of the base course materials should be maintained within the range of three percentage points below the optimum moisture content to three percentage points above the optimum moisture content until permanently covered.

CRUSHED LIMESTONE BASE COURSE

The crushed limestone base course should consist of material conforming to TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, Item 247, Flexible Base, Type A, Grade 1.

The crushed limestone base course should be placed in lifts with a maximum compacted thickness of 8 in. and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557. The moisture content of the crushed limestone base course materials should be maintained within the range of three percentage points below the optimum moisture content to three percentage points above the optimum moisture content until permanently covered.

ASPHALTIC CONCRETE SURFACE COURSE

The asphaltic concrete surface course should conform to TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, Item 341, Dense-Graded Hot-Mix

Asphalt, Type D. The asphaltic concrete should be compacted to a minimum of 92 percent of the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

PORTLAND CEMENT CONCRETE

The Portland cement concrete pavement should be air entrained to result in a 4 percent plus/minus 1 percent air, should have a maximum slump of 5 inches, and should have a minimum 28-day compressive strength of 3,500 psi. A liquid membrane-forming curing compound should be applied as soon as practical after broom finishing the concrete surface. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.

MISCELLANEOUS PAVEMENT RELATED CONSIDERATIONS

Longitudinal Cracking

It should be understood that asphalt pavement sections in expansive soil environments, such as those encountered at this site, can develop longitudinal cracking along unprotected pavement edges. These cracks can develop within a very short period of time (as short as three to four weeks after construction). In the semi-arid climate of south Texas this condition typically occurs along the unprotected edges of pavements where moisture fluctuation is allowed to occur over the lifetime of the pavements.

Pavements that do not have a vertical and/or horizontal protective barrier to reduce moisture fluctuation of the highly expansive clay subgrade between the exposed pavement edge and that beneath the pavement section tend to develop longitudinal cracks 1 to 4 ft from the edge of the pavement. Once these cracks develop, further degradation and weakening of the underlying granular base may occur due to water seepage through the cracks. The occurrence of these cracks can be more prevalent in the absence of lateral restraint and embankments. This problem can best be addressed by providing either a horizontal or vertical moisture barrier at the unprotected pavement edge.

At a minimum, we recommend that the curbs are constructed such that the depth of the curb extends through the entire depth of the granular base material and into the subgrade to act as a protective barrier against the infiltration of water into the granular base.

In most cases, a longitudinal crack does not immediately compromise the structural integrity of the pavement system. However, if left unattended, infiltration of surface water runoff into the crack will result in isolated saturation of the underlying base. This will result in pumping of the flexible base, which could lead to rutting, cracking, and pot-holes. For this reason, we recommend that the owner of the facility immediately seal the cracks and develop a periodic sealing program.

Pavement Maintenance

Regular pavement maintenance is critical in maintaining pavement performance over a period of several years. All cracks that develop in asphalt pavements should be regularly sealed. Areas of moderate to severe fatigue cracking (also known as alligator cracking) should be sawcut and removed. The underlying base should be checked for contamination or loss of support and any insufficiencies fixed or removed and the entire area patched.

All cracks that develop in concrete pavements should be routed and sealed regularly. Joints in concrete pavements should be maintained to reduce the influx of incompressible materials that restrain joint movement and cause spalling and/or cracking. Other typical facility maintenance techniques should be followed as required.

Construction Traffic

Construction traffic on prepared subgrade or granular base should be restricted as much as possible until the protective asphalt surface pavement is applied. Significant damage to the underlying layers resulting in weakening may occur if heavily loaded vehicles are allowed to use these areas prior to the complete construction of the pavement section. Heavy traffic loads should not be allowed on light duty traffic areas either before or after completion of the pavement section.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS ENGINEERING AND TESTING 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, **RABA KISTNER, Inc.**, is retained to perform the construction materials engineering and testing services during the construction of the project. This is because:

- **RKI** has an intimate understanding of the geotechnical engineering report's findings and recommendations. **RKI** understands how the report should be interpreted and can provide such interpretations on site, on the CLIENT's behalf.
- **RKI** knows what subsurface conditions are anticipated at this site.
- **RKI** is familiar with the goals of the CLIENT and the project's design professionals, having worked with them in the development of the project geotechnical workscope. This enables **RKI** to suggest remedial measures (when needed) which help meet others' requirements.

- **RKI** 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.
- **RKI** 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

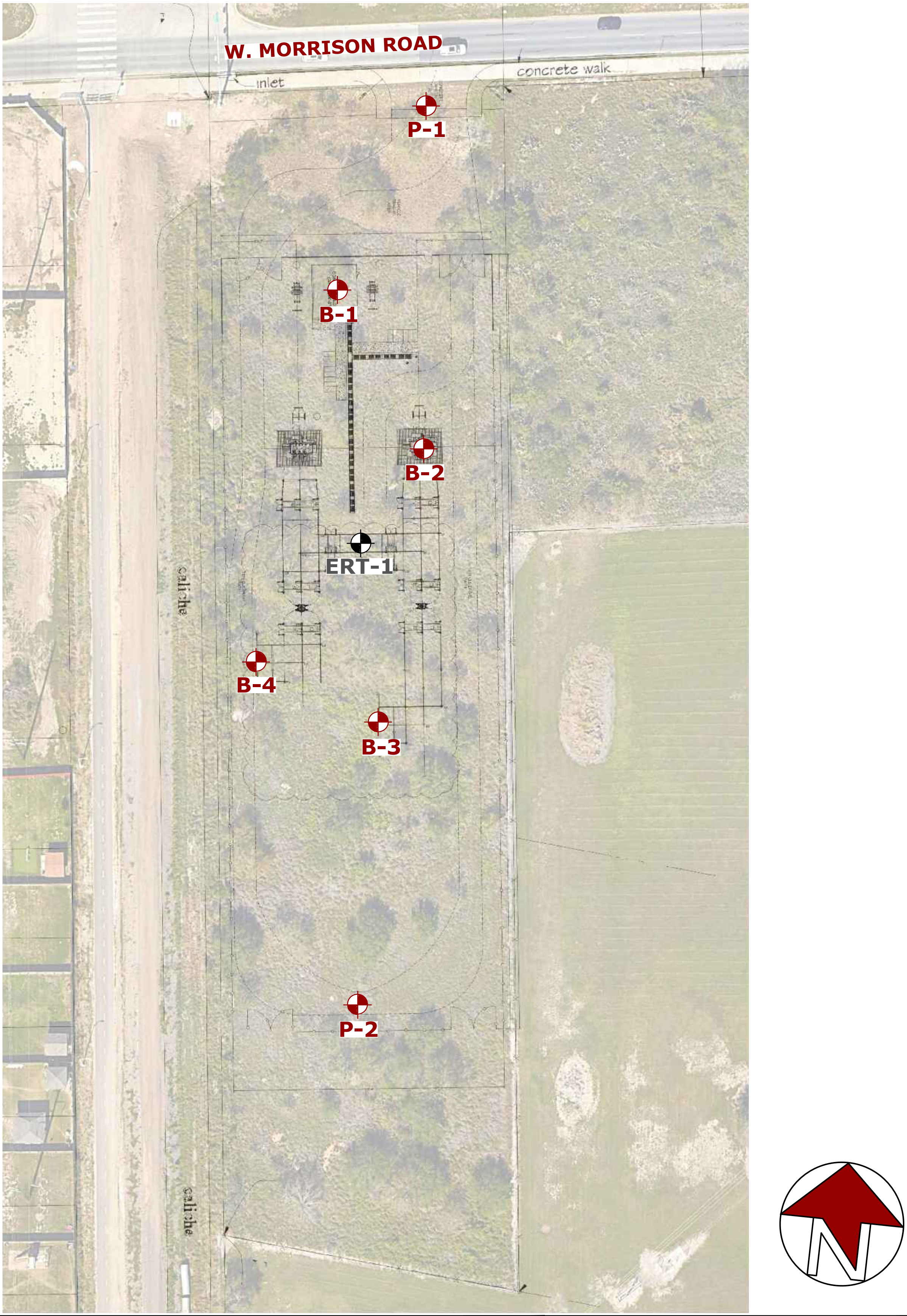
Appropriate budgets need to be developed for the required construction materials engineering and testing services. At the appropriate time before construction, we advise that **RKI** and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as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

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected General Contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. **RKI** 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.

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

ATTACHMENTS

O:\Active Projects\McAllen\2024\ABA24\ABA24-011-00 Prop. Ocelot Electrical Substation\Drawings\Figure 1 - Revised.dwg



RABA KISTNER
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(956) 682-5487 FAX
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TBPE Firm F-3257

BORING LOCATION MAP
PROPOSED OCELOT ELECTRICAL SUBSTATION

ALONG THE SOUTH SIDE OF W. MORRISON ROAD
BROWNSVILLE, CAMERON COUNTY, TEXAS

REVISIONS:

No.	DATE	DESCRIPTION
1	10-23-24	B-3 & B-4 Locations

PROJECT No.:	ABA24-011-00 (Revised)
ISSUE DATE:	10-23-2024
DRAWN BY:	DV
CHECKED BY:	AD
REVIEWED BY:	SC
FIGURE:	1

LOG OF BORING NO. B-1

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

DEPTH, FT	DESCRIPTION OF MATERIAL SURFACE ELEVATION: Existing Grade, ft	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	**FAT CLAY (CH)** firm to stiff, dark brown to brown, with calcareous nodules	7		36	
		6			98
5			104		
	During the drilling operations, groundwater was encountered at a depth of about 6.5 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 7 ft.				
	LEAN CLAY (CL) stiff, brown	8		19	
10					
15		15			89
20				18	
25	**LEAN CLAY (CL)** firm to very stiff, light brown	7			

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

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

DEPTH DRILLED:	50.0 ft	**DEPTH TO WATER:**	6.5 ft	**PROJ. No.:**	ABA24-011-00
DATE DRILLED:	8/15/2024	**DATE MEASURED:**	8/15/2024	**FIGURE:**	2a

LOG OF BORING NO. B-1

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

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

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURFACE ELEVATION: Existing Grade, ft	BLOWS PER FT	UNIT DRY WEIGHT, pcf	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)	PLASTICITY INDEX	% -200
			LEAN CLAY (CL) firm to very stiff, light brown *(continued)*	10				
35				14				
40				11				
45				16				
				19				
50			Boring terminated at a depth of about 50 ft.					
55								

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 8/15/2024
DEPTH TO WATER: 6.5 ft
DATE MEASURED: 8/15/2024
PROJ. No.: ABA24-011-00
FIGURE: 2b

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

LOG OF BORING NO. B-2

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

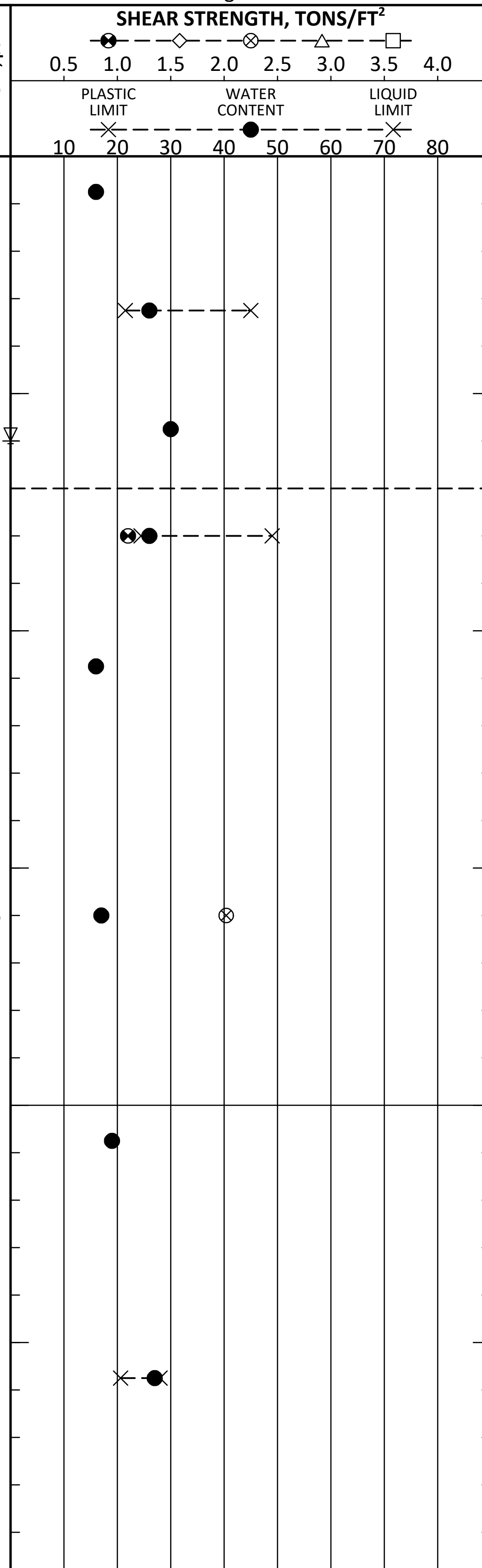
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

SURFACE ELEVATION: Existing Grade, ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	**LEAN CLAY with SAND (CL)** firm to stiff, dark brown to brown, with calcareous nodules	6			79
		6		24	
5	During the drilling operations, groundwater was encountered at a depth of about 7 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 6 ft.	9			
7	**LEAN CLAY (CL)** very stiff to stiff, brown			25	
10		12			
15			116		84
20	**SILTY CLAY with SAND (CL-ML)** stiff, brown	9			
25		9		7	



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

DEPTH DRILLED: 50.0 ft	**DEPTH TO WATER:** 6 ft	**PROJ. No.:** ABA24-011-00
DATE DRILLED: 8/15/2024	**DATE MEASURED:** 8/15/2024	**FIGURE:** 3a

LOG OF BORING NO. B-2

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

SHEAR STRENGTH, TONS/FT²: 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
	CLAYEY SAND (SC) medium dense, brown to light brown	13			
35		13			
40	**LEAN CLAY (CL)** stiff to very stiff, brown to light brown	15			
45		18			
		17			
50	Boring terminated at a depth of about 50 ft.				
55					

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

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 8/15/2024
DEPTH TO WATER: 6 ft
DATE MEASURED: 8/15/2024
PROJ. No.: ABA24-011-00
FIGURE: 3b

LOG OF BORING NO. B-3

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron 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	**SANDY LEAN CLAY (CL)** firm , brown, with roots extending down to a depth of about 2 feet	7		24	
		6			52
5	**FAT CLAY with SAND (CH)** firm to soft to stiff, brown. During the drilling operations, groundwater was encountered at a depth of about 7 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 5 ft.	5		31	
			97		
10		3			
15					
20	**SILTY CLAY (CL-ML)** firm to very soft, brown	6		7	
25		2			87

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

DEPTH DRILLED: 70.0 ft
DATE DRILLED: 8/17/2024
DEPTH TO WATER: 5 ft
DATE MEASURED: 8/17/2024
PROJ. No.: ABA24-011-00
FIGURE: 4a

LOG OF BORING NO. B-3

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron 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
	LEAN CLAY with SAND (CL) very stiff to firm, brown	17			
35		18			
40		7			96
45	**LEAN CLAY (CL)** very stiff to firm, brown to light brown	16			
50		15			
55		7			

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

DEPTH DRILLED: 70.0 ft
DATE DRILLED: 8/17/2024
DEPTH TO WATER: 5 ft
DATE MEASURED: 8/17/2024
PROJ. No.: ABA24-011-00
FIGURE: 4b

LOG OF BORING NO. B-3

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	SHEAR STRENGTH, TONS/FT² (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)	PLASTICITY INDEX	% -200
			SURFACE ELEVATION: Existing Grade, ft					
			CLAYEY SAND (SC) medium dense, brown	27				
			FAT CLAY with SAND (CH) firm to soft to stiff, brown					
65				24				
				25				
70			Boring terminated at a depth of about 70 ft.					
75								
80								
85								

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

DEPTH DRILLED:	70.0 ft	**DEPTH TO WATER:**	5 ft	**PROJ. No.:**	ABA24-011-00
DATE DRILLED:	8/17/2024	**DATE MEASURED:**	8/17/2024	**FIGURE:**	4c

LOG OF BORING NO. B-4

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

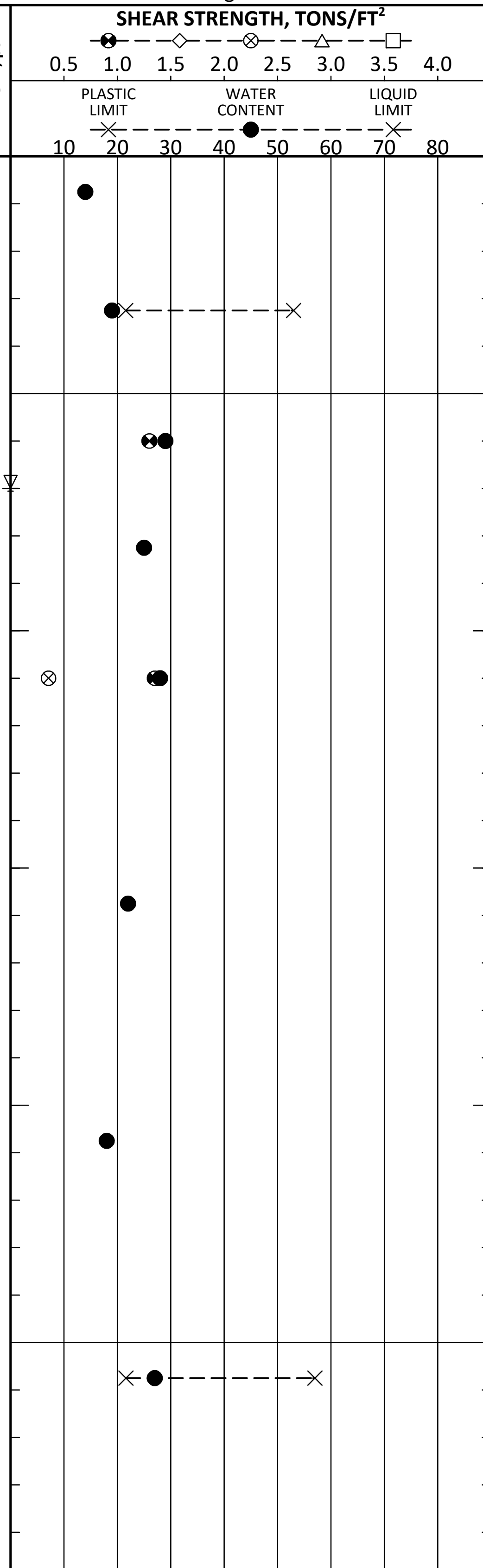
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger & Mud Rotary **LOCATION:** See Figure 1

SURFACE ELEVATION: Existing Grade, ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	SANDY FAT CLAY (CH) soft to stiff, brown, with roots extending down to a depth of about 2 feet	4			69
		9		31	
5	**SILT (ML)** stiff to soft to stiff, brown to light brown				94
	During the drilling operations, groundwater was encountered at a depth of about 13 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 7 ft.	9		NP	
10			97		
15		13			
20		15			
25	**CLAYEY SAND (SC)** very loose to medium dense, brown Driller's Note: Converted to mud rotary drilling method at a depth of about 25 ft	3		35	

SHEAR STRENGTH, TONS/FT^2



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

DEPTH DRILLED: 70.0 ft **DEPTH TO WATER:** 7 ft **PROJ. No.:** ABA24-011-00
DATE DRILLED: 8/17/2024 **DATE MEASURED:** 8/17/2024 **FIGURE:** 5a

LOG OF BORING NO. B-4

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger & Mud Rotary **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
	CLAYEY SAND (SC) very loose to medium dense, brown *(continued)*	21			
35		19			
40	**LEAN CLAY (CL)** firm to very stiff, brown to light brown	6			
45		20			100
50		19			
55	**LEAN CLAY (CL)** stiff to very stiff, brown	10			

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

DEPTH DRILLED:	70.0 ft	**DEPTH TO WATER:**	7 ft	**PROJ. No.:**	ABA24-011-00
DATE DRILLED:	8/17/2024	**DATE MEASURED:**	8/17/2024	**FIGURE:**	5b

LOG OF BORING NO. B-4

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger & Mud Rotary **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	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	WATER CONTENT (%)	PLASTICITY INDEX	% -200
			LEAN CLAY (CL) stiff to very stiff, brown *(continued)*	27				
65								
				27				
				26				
70			Boring terminated at a depth of about 70 ft.					
75								
80								
85								

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

DEPTH DRILLED: 70.0 ft	**DEPTH TO WATER:** 7 ft	**PROJ. No.:** ABA24-011-00	
DATE DRILLED: 8/17/2024	**DATE MEASURED:** 8/17/2024	**FIGURE:** 5c	

LOG OF BORING NO. P-1

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

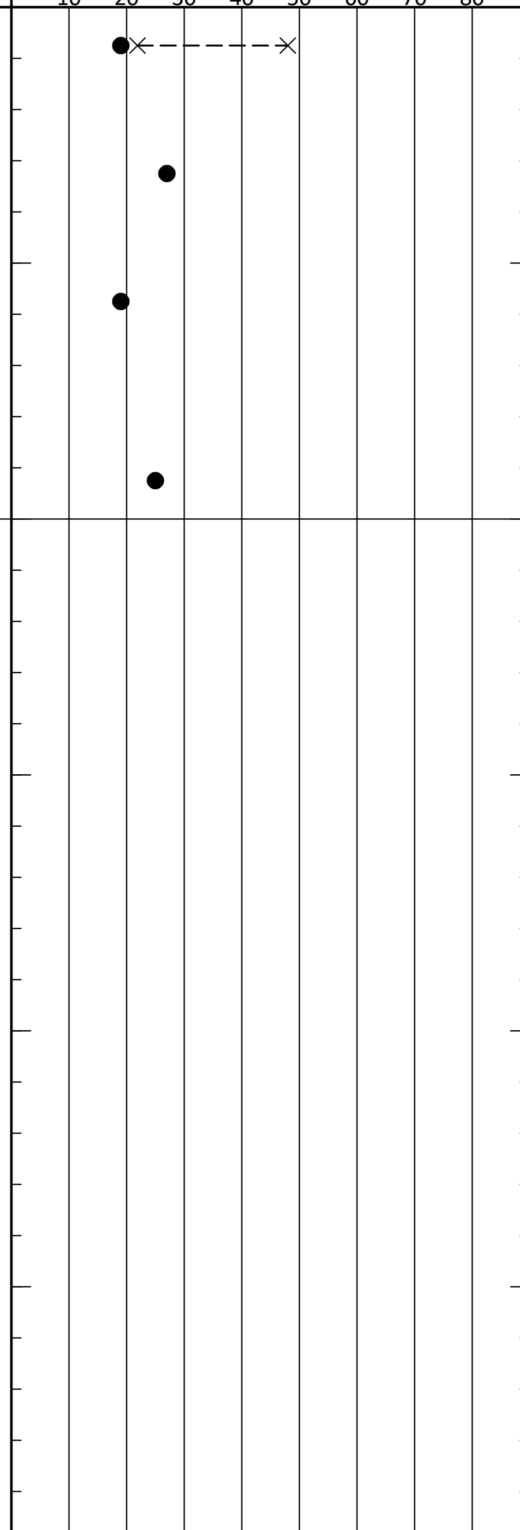
DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

SURFACE ELEVATION: Existing Grade, ft

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	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0–10	**LEAN CLAY (CL)** firm, brown, with calcareous nodules	5		26	
		8			96
		8			
		8			
10	Boring terminated at a depth of about 10 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:	10.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ABA24-011-00
DATE DRILLED:	8/20/2024	**DATE MEASURED:**	8/20/2024	**FIGURE:**	6

LOG OF BORING NO. P-2

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURFACE ELEVATION: Existing Grade, ft	BLOWS PER FT	UNIT DRY WEIGHT, pcf	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)	PLASTICITY INDEX	% -200
			FAT CLAY with SAND (CH) firm, brown, with roots extending down to a depth of about 2 feet	6				84
				7			30	
5				7				
				7				
10			Boring terminated at a depth of about 10 ft. NOTES: Upon completion of the drilling operations, the boring was observed dry.					
15								
20								
25								

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

DEPTH DRILLED: 10.0 ft
DATE DRILLED: 8/20/2024
DEPTH TO WATER: DRY
DATE MEASURED: 8/20/2024
PROJ. No.: ABA24-011-00
FIGURE: 7

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

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

ROCK TERMS

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

OTHER

ASPHALT
BASE
CONCRETE/CEMENT
BRICKS / PAVERS
WASTE
NO INFORMATION

WELL CONSTRUCTION AND PLUGGING MATERIALS

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

SAMPLE TYPES

AIR ROTARY
MUD ROTARY
SHELBY TUBE
GRAB SAMPLE
NO RECOVERY
SPLIT BARREL
CORE
NX CORE
SPLIT SPOON
GEOPROBE SAMPLER
PITCHER
TEXAS CONE PENETROMETER
ROTOSONIC -DAMAGED
ROTOSONIC -INTACT
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. ABA24-011-00

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: Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

FILE NAME: ABA24-011-00.GPJ 9/10/2024

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	7	17	58	22	36	CH				
	2.5 to 4.0	6	28						98		
	5.0 to 7.0		20					104		1.01	UC
	7.5 to 9.0	8	26	42	23	19	CL				
	10.0 to 12.0		23							1.30	PP
	15.0 to 16.5	15	18						89		
	20.0 to 22.0		21	37	19	18	CL			1.10	PP
	25.0 to 26.5	7	30								
	30.0 to 31.5	10	28								
	35.0 to 36.5	14	24								
	40.0 to 41.5	11	25								
	45.0 to 46.5	16	29								
	48.5 to 50.0	19	25								
B-2	0.0 to 1.5	6	16						79		
	2.5 to 4.0	6	26	45	21	24	CL				
	5.0 to 6.5	9	30								
	7.0 to 9.0		26	49	24	25	CL			1.10	PP
	10.0 to 11.5	12	16								
	15.0 to 17.0		17					116	84	2.02	UC
	20.0 to 21.5	9	19								
	25.0 to 26.5	9	27	28	21	7	CL-ML				
	30.0 to 31.5	13	23								
	35.0 to 36.5	13	26								
	40.0 to 41.5	15	27								
	45.0 to 46.5	18	24								
	48.5 to 50.0	17	23								
B-3	0.0 to 1.5	7	14	43	19	24	CL				
	2.5 to 4.0	6	13						52		
	5.0 to 6.5	5	28	58	27	31	CL				
	7.0 to 9.0		28					97		0.57	UC
	10.0 to 11.5	3	30								
	15.0 to 17.0		16							1.40	PP
	20.0 to 21.5	6	25	27	20	7	CL-ML				
	25.0 to 26.5	2	26						87		
	30.0 to 31.5	17	22								
	35.0 to 36.5	18	22								
	40.0 to 41.5	7	24						96		
	45.0 to 46.5	16	28								
	50.0 to 51.5	15	27								

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 PROJECT NO. ABA24-011-00

RABAKISTNER

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

FILE NAME: ABA24-011-00.GPJ 9/10/2024

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-3	55.0 to 56.5	7	28								
	60.0 to 61.5	27	25								
	65.0 to 66.5	24	26								
	68.5 to 70.0	25	26								
B-4	0.0 to 1.5	4	14						69		
	2.5 to 4.0	9	19	53	22	31	CH				
	5.0 to 7.0		29						94	1.30	PP
	7.5 to 9.0	9	25	CNBD	CNBD	NP	ML				
	10.0 to 12.0		28					97		0.36	UC
	15.0 to 16.5	13	22								
	20.0 to 21.5	15	18								
	25.0 to 26.5	3	27	57	22	35	SC				
	30.0 to 31.5	21	25								
	35.0 to 36.5	19	27								
	40.0 to 41.5	6	28								
	45.0 to 46.5	20	27						100		
	50.0 to 51.5	19	26								
	55.0 to 56.5	10	29								
	60.0 to 61.5	27	26								
	65.0 to 66.5	27	25								
	68.5 to 70.0	26	25								
P-1	0.0 to 1.5	5	19	48	22	26	CL				
	2.5 to 4.0	8	27						96		
	5.0 to 6.5	8	19								
	8.5 to 10.0	8	25								
P-2	0.0 to 1.5	6	18						84		
	2.5 to 4.0	7	17	51	21	30	CH				
	5.0 to 6.5	7	28								
	8.5 to 10.0	7	24								

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 PROJECT NO. ABA24-011-00

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FIGURE 9b

RESISTIVITY SOUNDING DATA SHEET

Wenner Array, Method ASTM G-57

Proposed Ocelot Electrical Substation Project

South of Morrison Road

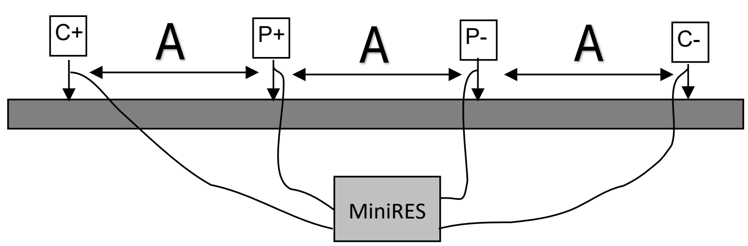
Brownsville, Cameron County, Texas

RKI Project Number:	ABA24-011-00		
Date:	8/8/2024	**Time:**	9:00am
Meter:	Super MiniRes	**Units:**	Ohms
Weather Conditions:	Clear, hot	**Observer(s):**	Anthony Krupa

SOUNDING No.: **ERT-1**

Location Description: ***South-central, proposed building pad area***

Electrode Spacing (Feet)			Factor	Meter Reading (Ohms)		Apparent Resistivity (Ohm-Feet)		Apparent Resistivity (Ohm-Centimeters)	
A	**A/2**	**3A/2**	**(2 * PI * A)**	**N-S**	**E-W**	**N-S**	**E-W**	**N-S**	**E-W**
1.5	***0.8***	***2.3***	9.42	0.659	0.607	6.2	5.7	189.3	174.4
5	***2.5***	***7.5***	31.40	0.131	0.126	4.1	4.0	125.4	120.6
7.5	***3.8***	***11.3***	47.10	0.095	0.087	4.5	4.1	136.4	125.0
10	***5.0***	***15.0***	62.80	0.058	0.055	3.6	3.5	111.1	105.3
15	***7.5***	***22.5***	94.20	0.041	0.039	3.9	3.7	117.8	112.0
20	***10.0***	***30.0***	125.60	0.031	0.026	3.9	3.3	118.7	99.6



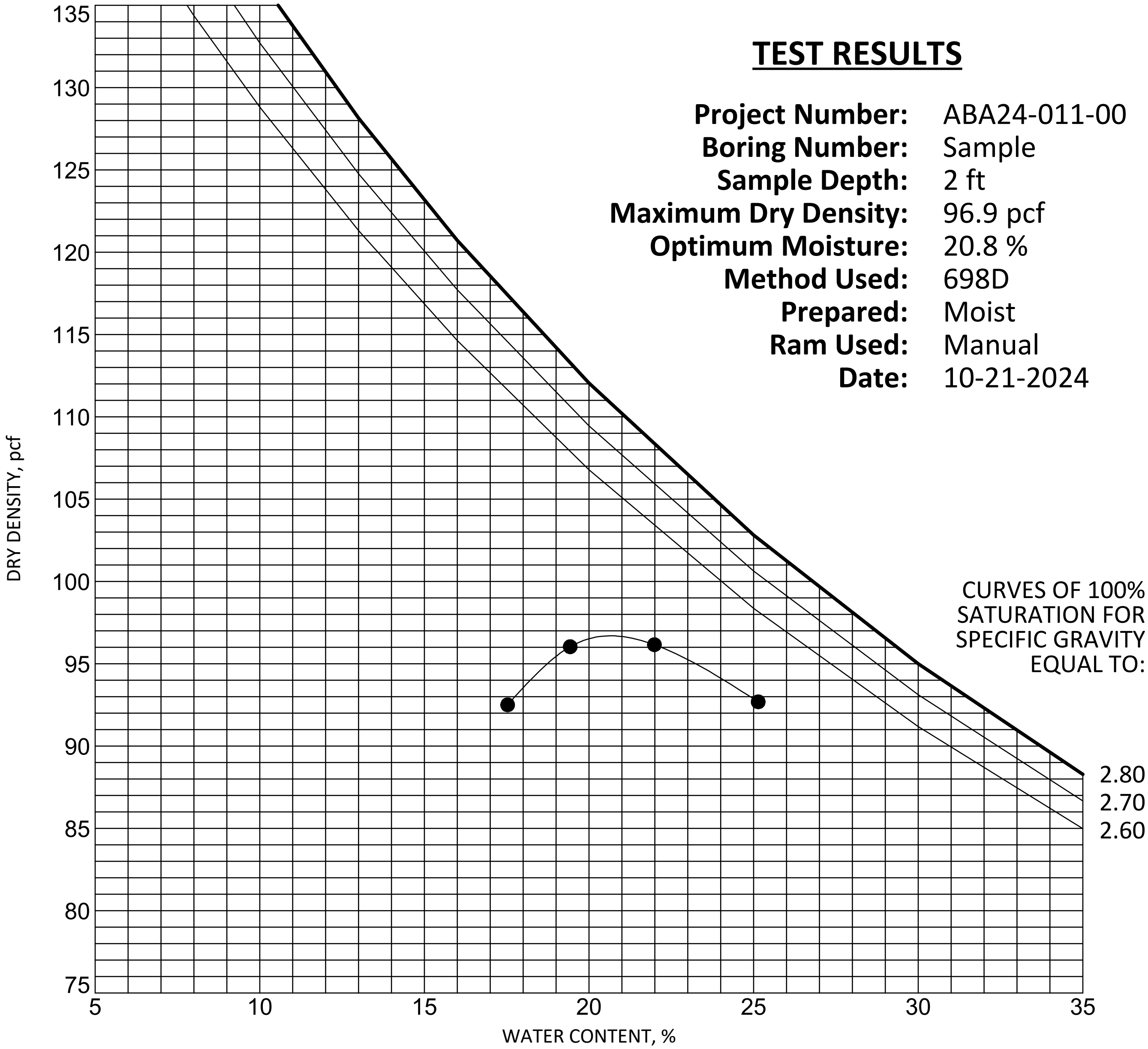
Notes:

GPS LOCATIONS:
N 25.96471
W 97.52142

UTM Zone 14
NAD 83
(horizontal position error ~±3-5 meters)

* Resistor test = 19.005
* Wire test = 19.005

Soil Description: Brown Clay, moist, firm



TEST RESULTS

Project Number: ABA24-011-00
Boring Number: Sample
Sample Depth: 2 ft
Maximum Dry Density: 96.9 pcf
Optimum Moisture: 20.8 %
Method Used: 698D
Prepared: Moist
Ram Used: Manual
Date: 10-21-2024

Liquid Limit: 52
Plastic Limit: 23
Plasticity Index: 29

Passing No. 200 (%): 78

Borrow Source Description: Near Boring B-2
Classification (ASTM D 2488): FAT CLAY with SAND (CH)
Classification (AASHTO M 145-95): A-7-6

FIGURE 11

RABA KISTNER
a kiwa company
TBPE Firm Registration No. F-3257

800 E. Hackberry
McAllen, Texas 78501
(956) 682-5332
(956) 682-5487 fax
www.rkci.com

MOISTURE-DENSITY RELATIONSHIP

Proposed Ocelot Electrical Substation
Along the South Side of W. Morrison Road
Brownsville, Cameron County, Texas

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.

GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

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