Geotechnical Engineering Report

Rio Grande Distribution Substation

Brownsville, Texas

October 11, 2017

Terracon Project No. 88175125

Prepared for:

Brownsville Public Utilities Board
Brownsville, Texas

Prepared by:

Terracon Consultants, Inc.
Pharr, Texas

October 11, 2017

Brownsville Public Utilities Board
1495 Robinhood Drive
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Attn: Diane Solitaire
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Regarding: Geotechnical Engineering Report
Rio Grande Distribution Substation
Brownsville, Texas
Terracon Project No. 88175125

Dear Ms. Solitaire:

Terracon Consultants, Inc. (Terracon) is pleased to submit this Geotechnical Engineering Report for the project referenced above to be constructed in Brownsville, Texas. We trust that this report is responsive to your project needs. Please contact us if you have any questions or if we can be of further assistance.

We appreciate the opportunity to work with you on this project and look forward to providing additional geotechnical engineering services, as applicable, in addition to providing the construction materials testing services during construction.

Sincerely,
Terracon Consultants, Inc.
(Texas Firm Registration No.: F-3272)

Stephany Chacón, E.I.T.
Staff Engineer

Alfonso A. Soto, P.E., D.GE
Principal

Enclosures
Copies Submitted: Addressee: (1) Electronic

TABLE OF CONTENTS

Table of Contents Cont'd

EXECUTIVE SUMMARY

A geotechnical investigation has been performed for the proposed Rio Grande Distribution Substation to be located on the northeast quadrant (NEQ) of Power Plant Drive and Rio Vista Avenue in Brownsville, Cameron County, Texas. Three borings, designated B-1 through B-3 were performed to depths of about 50 feet below existing grade (grade at the time of our field activities) within the proposed substation areas. Groundwater was observed between 8 and 16 feet below existing grade during our drilling activities. Based on the information obtained from our subsurface exploration, the site can be developed for the proposed project. The following geotechnical considerations were identified:

- Stripping should include surface vegetation, loose topsoil, or other unsuitable materials in the substation area.
- Proofrolling should be performed to detect weak areas. Weak areas should be removed and replaced with select fill or soils exhibiting similar characteristics as the adjacent in-situ soils.
- A shallow foundation system consisting of a slab-on-grade or mat would be appropriate to support the structural loads of light-weight structures provided the subgrade is prepared as discussed in this report. Spread footings can be used for structures that are not sensitive to movement.
- Grade beams for a slab-on-grade foundation system should be sized for a net total load allowable bearing pressure of 2,500 psf.
- A mat foundation system should be sized for a maximum contact pressure of 2,500 psf and a subgrade modulus of 75 pcf.
- A deep foundation system consisting of straight-sided, drilled piers bearing at a minimum depth of 10 feet and a maximum of 45 feet may be considered if greater capacity than shallow foundations can provide is needed.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **GENERAL COMMENTS** should be read regarding the limitations of this report.

GEOTECHNICAL ENGINEERING REPORT
RIO GRANDE DISTRIBUTION SUBSTATION
BROWNSVILLE, TEXAS

Project No. 88175125
October 11, 2017

1.0 INTRODUCTION

Terracon Consultants, Inc. (Terracon) is pleased to submit our Geotechnical Engineering Report for the proposed Rio Grande Distribution Substation in Brownsville, Texas. This project was authorized by Mr. James McCann, P.E., with BPUB through issuance of a signed Notice to Proceed dated September 6, 2017. Our scope of services for this project was outlined in Terracon Proposal No. P88175125, dated August 15, 2017.

The purpose of this report is to describe the subsurface conditions observed during our field subsurface exploration program for this study, analyze and evaluate the field and laboratory test data collected during the study, and provide recommendations with respect to:

- Site and subgrade preparation; and
- Foundation design and construction.

2.0 PROJECT INFORMATION

2.1 Project Description

Item	Description
Site layout	See Appendix A, Figure A-2, Boring Location Plan
Structures	The project will include typical substation and transmission structures.
Foundation construction	Anticipated shallow foundations (mat, slab-on-grade, footings) and deep (drilled pier foundations for transmission tower structures).
Finished floor elevation	Min. 2 feet above existing grade (grade at the time of our field activities).
Maximum loads	Variable.

2.2 Site Location and Description

Item	Description
Location	This project will be located on the northeast quadrant (NEQ) of Power Plant Drive and Rio Vista Ave. in Brownsville, Texas. Latitude: 25.914226° Longitude: -97.519363° W.
Existing improvements	None. Vacant land.
Current ground cover	The site is generally covered by native grasses and soils.
Existing topography	Relatively flat and level.

3.0 SUBSURFACE CONDITIONS

3.1 Geology

3.1.1 Site Geology

Based on the Geologic Atlas of Texas, McAllen – Brownsville prepared by The University of Texas, the site is located on the Alluvium Formation of the Holocene (Recent) Period of the Quaternary Age. Floodplain deposits, lower course of Rio Grande, are divided into areas dominantly mud, Qam, and areas dominantly silt and sand, Qas. All other areas are alluvium undivided, Qal, except for some areas where tidal flat areas are mapped. The soils are mostly composed of clay, silt, sand, gravel and organic matter. The silt and sand are described as calcareous and dark gray to dark brown in color. The sand is mostly quartz and the gravel along Rio Grande include sedimentary rocks from the Cretaceous and Tertiary and a wide variety of igneous and sedimentary rocks from Trans-Pecos Texas, Mexico, and New Mexico including agate. The gravel in side streams of the Rio Grande is mostly Tertiary rocks and chert derived from Uvalde Gravel which caps divide.

3.2 Typical Profile

Based on the results of the borings, subsurface conditions on the project site can be generalized as follows:

Description	Depth (ft)	Plasticity Index (%)	In-situ Moisture Content (%)	Moisture content vs. Plastic limit[1] (%)		SPT N-Value[2] (bpf)	Fines[3] (%)
				Dry	Wet		
Sandy Lean Clay (CL)	0 - 6	19 - 24	13 - 26	3 - 4	9	4 - 17	89

Description	Depth (ft)	Plasticity Index (%)	In-situ Moisture Content (%)	Moisture content vs. Plastic limit[1] (%)		SPT N-Value[2] (bpf)	Fines[3] (%)
				Dry	Wet		
Fat Clay (CH)[4]	4 - 50	21 - 45	19 - 29	0 - 3	0 - 10	6 - 22	98 - 99

1. The difference between a soil sample's in-situ moisture content and its corresponding plastic limit.
2. bpf = blows per foot.
3. Percent passing the No. 200 sieve.
4. With Lean Clay (CL) seams

Conditions encountered at each boring location are indicated on the individual boring logs. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in-situ, the transition between materials may be gradual. Details for each of the borings can be found on the boring logs in Appendix A of this report.

3.3 Groundwater

The borings were advanced using dry drilling techniques to their termination depths in an effort to evaluate groundwater conditions at the time of the field investigation. The borings were backfilled with cuttings after completion of the drilling operations for safety purposes. Groundwater was observed in the borings as summarized below.

Location	Depth to groundwater (feet)		
	During drilling	15 minutes after initial groundwater reading	After boring termination
B-1	11	9	16
B-2	9½	8	10
B-3	12	9	10

These groundwater observations are considered short-term, since the borings were open for a short time period. On a long-term basis, groundwater may be present at shallower depths. Additionally, groundwater will fluctuate with sea-level and seasonal weather changes and should be evaluated at the time of construction.

4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

The following recommendations are based upon the data obtained in our field and laboratory programs, project information provided to us and on our experience with similar subsurface and site conditions.

4.1 Geotechnical Considerations

Grade changes for the proposed site were not provided to us at the time of this report. However, based on the existing topography, we anticipate that final grade will be at least 1½ feet above existing grade.

Expansive soils are present at the site. This report provides recommendations to help mitigate the effects of soil shrinkage and expansion. However, even if these procedures are followed, some movement and at least minor cracking in the structures should be anticipated. The severity of cracking and other cosmetic damage such as uneven floor slabs will probably increase if any modification of the site results in excessive wetting or drying of the expansive soils. Eliminating the risk of movement and cosmetic distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction. We would be pleased to discuss other construction alternatives that could further reduce the potential for movement with you upon request. Recommendations to minimize excessive movements are discussed in the **"4.2 Earthwork"** and **"4.6.1 Design Recommendations"** sections of this report.

4.1.1 Swell Test Results

Several swell tests were performed on soil samples from the borings drilled at the site. After surcharge pressures were applied the samples were inundated with water for about 72 to 96 hours while measurements of vertical displacement were taken. The magnitude of swell is recorded as a function of the change in thickness during the test in relation to the initial thickness of the sample.

Based on our laboratory results, the samples tested generally exhibit a moderate to high free swell potential as indicated by percent free swells of 1.1 percent to 5.4 percent within the upper 8 feet. When equivalent overburden pressure was applied, the results ranged between 0.6 and 3.1 percent swell. The summary of test results is presented in Appendix B, Exhibits B-2 and B-3.

4.2 Earthwork

Construction areas should be stripped of vegetation, topsoil, and other unsuitable material. Additional excavation as recommended in the **"4.6.1 Design Recommendations"** section of this report should be performed within the structures and equipment areas. Once final subgrade elevations have been achieved, the exposed subgrade should be carefully proofrolled with a 15-ton pneumatic roller or a fully loaded dump truck to detect weak zones in the subgrade. Special care should be exercised when proofrolling the fill soils to detect soft/weak areas. Weak areas detected during proofrolling, as well as zones of fill containing organic matter and/or debris should be removed and replaced with select fill in the proposed building area. Weak areas observed in proposed service areas may be replaced with clean on-site soils or select fill.

Proper site drainage should be maintained during construction so that ponding of surface runoff does not occur and causes construction delays and/or inhibit site access.

Subsequent to proofrolling, and just prior to placement of fill, the exposed subgrade within the construction area should be evaluated for moisture and density. If the moisture, density, and/or the requirements do not meet the criteria described in the table below, the subgrade should be scarified to a minimum depth of 8 inches, moisture adjusted and compacted to at least 95 percent of the Standard Effort (ASTM D 698) maximum dry density. Select fill and on-site soils should meet the following criteria.

Fill Type [1]	USCS Classification	Acceptable Location for Placement
Select Fill	CL and/or SC (7≤PI≤20)	Must be used to construct the pads under the structures and all grade adjustments.
Aggregate base course[2]	SC, GC, Caliche, Crushed Limestone Base	Top 6 inches of pads.
On-Site Soils	CL	The on-site soils may suitable for use as fill material as long as they are free from organics and have a PI between 7 and 20.

1. Prior to any filling operations, samples of the proposed borrow and on-site materials should be obtained for laboratory moisture-density testing. The tests will provide a basis for evaluation of fill compaction by in-place density testing. A qualified soil technician should perform sufficient in-place density tests during the filling operations to evaluate that proper levels of compaction, including dry unit weight and moisture content, are being attained.
2. The clayey gravel and caliche materials should meet the gradation requirements of Item 247, Type B, Grades 1 through 3 as specified in the 2004 TxDOT Standard Specifications Manual and a plasticity index between 7 and 20. Crushed limestone or crushed concrete material should meet the requirements of 2004 TxDOT Item 247, Type A or D, Grade 1. The select fill materials should be free of organic material and debris, and should not contain stones larger than 2 inches in the maximum dimension.

If imported, blended or mixed soils are intended for use to construct the building pad, Terracon should be contacted to provide additional recommendations. Blended or mixed soils do not occur naturally. These soils are a blend of sand and clay and will require mechanical mixing at the site. If these soils are not mixed thoroughly to break down the clay clods and blend-in the sand to produce a uniform soil matrix, the fill material may be detrimental to the slab performance. If blended soils are used, we recommend that additional samples of the blended soils, as well as the clay clods, be obtained prior to and during earthwork operations to evaluate if the blended soils can be used in lieu of select fill. The actual type and amount of mechanical mixing at the site will depend on the amount of clay and sand, and properties of the clay.

4.2.1 Compaction Requirements

ITEM	DESCRIPTION
Fill Lift Thickness	The fill soils should be placed on prepared surfaces in lifts not to exceed 8 inches loose measure, with compacted thickness not to exceed 6 inches.
Compaction Requirements (On-site Soils)	The on-site soils, including subgrade, should be compacted to at least 95 percent of the Standard Effort (ASTM D 698) maximum dry density between optimum and 4 percent of the optimum moisture content.
Compaction Requirements (Select Fill Soils)	The select fill soils should be compacted to at least 95 percent of the Standard Effort (ASTM D 698) maximum dry density within 2 percent of the optimum moisture content.

4.2.2 Wet Weather/Soft Subgrade Considerations

Construction operations may encounter difficulties due to the wet or soft surface soils becoming a general hindrance to equipment due to rutting and pumping of the soil surface, especially during and soon after periods of wet weather. If the subgrade cannot be adequately compacted to minimum densities as described above, one of the following measures will be required: 1) removal and replacement with select fill, 2) cement or fly ash treatment of the soil to dry out and increase the stability of the subgrade, or 3) drying by natural means if the schedule allows.

In our experience with similar soils in this area, chemical treatment is the most efficient and effective method to increase the supporting value of wet and weak subgrade. Terracon should be contacted for additional recommendations if chemical treatment of the soils is needed.

Prior to placing any fill, all vegetation, topsoil, and any otherwise unsuitable materials should be removed from the construction areas. Wet or dry material should either be removed or moisture conditioned and re-compacted. After stripping and grubbing, the subgrade should be proof-rolled where possible to aid in locating loose or soft areas. Proof-rolling can be performed with a 15-ton roller or fully loaded dump truck. Soft, dry and low-density soil should be removed or compacted in place prior to placing fill.

4.2.3 Grading and Drainage

It is important that positive drainage be established during construction such that water will not pond around the structures during or following the construction period.

All grades must be adjusted to provide positive drainage away from the structures. Where paving or flatwork abuts the structures, care should be taken that the joint is properly sealed and maintained. Roof drains should discharge away from the control building.

4.2.4 Corrosion Considerations

Laboratory soil pH, chlorides, resistivity and sulfates tests were conducted at an analytical lab on a selected soil sample recovered from the boring to assess the corrosivity risk of the soils at this site. The results of the laboratory tests are provided in Appendix B of this report.

4.2.5 Thermal Resistivity Testing

A laboratory thermal resistivity (TR) was performed on a remolded specimen of representative soils from the native soils. The bulk sample was recovered from the upper 4 feet of soil in the vicinity of boring B-1. The results of TR. The results of TR testing are included in Exhibits B-11 and B-12 of Appendix B.

4.3 Foundation Systems

The proposed structures, depending on the load requirements and expected performance, can be supported on a variety of foundations which include drilled piers, mats, and slabs. Near-surface foundations, such as the equipment pads and the slab foundations may be sensitive to movement. If a structure can withstand the anticipated movement presented in this report, then spread footings may be utilized.

A slab foundation would be appropriate for the proposed building-type structures. However, spread footings should not be utilized for buildings or structures sensitive to movement because of the increased risk of differential movement. Thickened and widened sections of the slab could be constructed for areas of concentrated loads.

A deep foundation system, such as drilled piers, would be appropriate to support the structural loads of dead end structures, and those structures requiring more capacity than a shallow foundation can provide. Recommendations for these types of foundation systems are provided in the following sections, along with other geotechnical considerations for this project.

4.3.1 Design Recommendations – Drilled Pier Foundation System

Axial compressive loads for the structures may be supported on straight-sided (non-underreamed) piers. Due to the presence of subsurface water at relatively shallow depth, underreamed drilled piers should not be considered for this site.

For the purposes of evaluating the subsoils for use in foundation analyses, we have developed soil parameters for axial capacity analysis for foundation design which are provided in Exhibits B-4 through B-6 of Appendix B.

Straight-sided drilled pier foundations may be designed using the following equations to evaluate the pier foundation sizes for both compressive and tensile (uplift) axial loading:

Ultimate skin friction capacity[1,5]:	$Q_s = \pi d\ (f_s)(h)$
Ultimate end bearing capacity[1,5]:	$Q_b = 0.25\pi d^2(q_{eb})$
Ultimate pier capacity in compression[5]:	$Q_c = Q_s + Q_b$
Ultimate pier capacity in tension (uplift)[1,5]:	$Q_t = \pi d\ (f_s)(h)(R)+W$
Ultimate skin friction[2,4]:	$f_s = \alpha c_u$
Ultimate skin friction[3,4]:	$f_s = \sigma' K \tan\delta$
Ultimate end bearing pressure[2,4]:	$q_{eb} = c_u N_c$
Ultimate end bearing pressure[3,4]:	$q_{eb} = \sigma' N_q$
Estimated uplift pressure (kips)[10, 11]:	$Up = (\pi \bullet d \bullet z_a \bullet \alpha \bullet Sp) / 1000$
Ultimate uplift resistance (kips)[1,6]	$Q_r = 2.5 \bullet d(ft.) \bullet h(ft.) + W(kips) + P_{DL}(kips)$
Minimum percentage of steel[7]	As required by structural engineer
Maximum embedment depth	45 feet below existing grade
Approximate total settlement[8]	About 1 inch
Estimated differential settlement[9]	Approximately ½ of total settlement

1. Definitions: d = pier diameter; h = pier segment length; R = uplift reduction factor (equal to 0.7 for sands, 0.9 for clays); W = effective weight of the pier foundation; P_{DL} = dead load acting on the drilled pier
2. Ultimate value for cohesive soils only.
3. Ultimate value for cohesionless soils only.
4. Soil parameters provided in Exhibits B-4 through B-6 in Appendix B: α = skin friction adhesion factor (strength reduction factor).

   (equal to zero with the top 5 feet and one shaft diameter of the base of the pier); c_u = undrained shear strength of the soil; N_c = end bearing capacity factor for clay soil; σ' = effective overburden pressure; K = horizontal stress coefficient; δ = soil to pier friction angle (equal to soil angle of internal friction (ϕ) for concrete piers); N_q = end bearing capacity factor for granular soils
5. A factor of safety of 3 should be applied to ultimate end bearing, 2 to side shear (skin friction), and 2 to uplift (tension).
6. A factor of safety of at least 2 should be applied to the computed ultimate uplift force.
7. The piers should contain sufficient vertical reinforcing steel throughout the entire shaft length to resist uplift (tensile) forces due to post-construction heave of the clay soils. The amount of reinforcing steel required can be computed by assuming that the dead load of the structure surcharges the pier and that the above estimated tensile force acts vertically on the shaft.
8. Provided proper construction practices are followed. A clear distance between the piers of three times the pier diameter should be provided to develop the recommended bearing pressures and to control settlements. If this clearance cannot be maintained in every case,

   Terracon should be contacted so that we may determine the reduced capacities. Settlements provided for single, isolated piers/piles only.
9. Will result from variances in subsurface conditions, loading conditions and construction procedures, such a cleanliness of the bearing area or flowing water in the shaft. Settlements provided for for single, isolated piers only.
10. The upper 4 feet of the borings are not considered to be highly expansive.
11. U_p = uplift load, kips; d = shaft diameter, feet; z_a = length of shaft over which active clay soils are expected to create uplift, feet; α = factor applied to vertical absorption pressure to account for shaft/soil friction; Sp = absorption pressure, psf.

4.3.1.1 Lateral Loading

Lateral loads will most likely be induced in drilled piers, but we do not anticipate that this will be the controlling design load case. A number of methods, including hand solutions and computer programs, are available for calculating the lateral behavior of piles or piers. The majority of these methods rely on key soil parameters such as soil elastic modulus (E), soil subgrade modulus (k_s), strain at 50 percent of the principal stress difference (ε_{50}), undrained shear strength (c), angle of internal friction (ϕ), and load-deflection (*p-y*) criteria. The *p-y* criteria, which are commonly used to model soil reaction, were developed from instrumented load tests and are generally considered to provide the best model of soil behavior under short-term lateral loading. The majority of *p-y* curve models use a form of parabola to describe the initial portion of a *p-y* curve; at small deflections the slope of these *p-y* curves approach infinity.

This is an important consideration, since most laterally loaded pile or pier analysis programs use *p-y* curves to develop equivalent soil springs in their computations. These soil springs are defined as the soil reaction, *p*, divided by the soil deflection, *y*, or the *secant modulus* of the *p-y* curve.

For most foundations subjected to lateral loads, the pier foundation is generally designed with a limit of 1 inch of deflection at the top of the pier and 1 degree of rotation as measured from the vertical axis of the pier. The analysis is generally conducted using the working loads and the limit state values. The applied loads are then doubled to evaluate the deflection and rotation at the top of the pier to determine if the foundation will topple over under extreme overload. This overload condition may indicate that the foundation would deflect or rotate such that the pole will tilt but not experience failure.

The choice of the final design embedment depth is also evaluated by determining the critical depth of the foundation. The critical depth is actually a depth range and can be defined as that range in depth at which a small decrease in the pier embedment depth will result in a large increase in the groundline deflection of the pier. The analysis is conducted by incrementally decreasing the embedment depth and plotting the resultant groundline deflection versus embedment depth. The embedment depth is then chosen below the critical depth range. In some cases, groundline deflections significantly less than 1 inch are needed to satisfy the critical depth issue.

Appropriate soil parameters for design of laterally loaded pier foundations under undrained conditions and criteria for the FAD Tools program are provided in Exhibits B-7 through B-9 of Appendix B. The criteria shown on these tables are based on our interpretation of the subsurface conditions. Maximum design loads were not provided when this report was issued.

4.3.1.2 Drilled Pier Foundation Construction Considerations

Drilled excavations to a minimum depth of 10 feet and a maximum of 45 feet below existing grade (grade at the time of our field activities) may be used for installation of the drilled piers for the proposed structures at this site. As previously discussed, relatively shallow subsurface water was encountered in the borings.

Based on our current groundwater observations (refer to the **"3.3 Groundwater"** section), groundwater was observed between 8 and 16 feet during drilling activities. Depending on weather conditions, groundwater levels may vary from the levels observed during our field program. Water must not be allowed to accumulate in the bottom of the pier excavations. Therefore, the contractor should be prepared to remove water from the drilled piers if necessary.

Sloughing is likely to occur below the subsurface water level during construction. Therefore, we recommend that casing or slurry drilling techniques be used to control sloughing of the subsurface soils during pier construction. Casing should only be used in drilled piers terminating in the Clay soils. Slurry drilling techniques are appropriate for piers terminating in all soil types encountered in the borings. The casing and slurry methods are discussed in the following paragraphs.

Casing Method- Casing will provide stability of the excavation walls but may not completely eliminate subsurface water influx potential or stability of the pier excavation bottom unless the casing penetrates below any pervious soils. Casing that terminates in pervious soils (sand) may generate "boils" due to the head differential between the inside and outside of the casing and require that the casing be extended until the excess seepage or boils are eliminated. The actual casing depth should be chosen by the drilling subcontractor. If this operation is not successful or to the satisfaction of the engineer, the pier excavation should be flooded with fresh water to offset the differential water pressure caused by the unbalanced water levels inside and outside of the casing. When the pier excavation depth is achieved and the bearing area has been cleaned, steel and concrete should then be placed immediately in the excavation. If more than 6 inches of water is present in the excavation, water should be removed by pumping or the concrete should be tremied completely to the bottom of the excavation with a closed-end tremie.

Removal of casing should be performed with extreme care and under proper supervision to reduce mixing of the surrounding soil and water with the fresh concrete. Rapid withdrawal of casing or the auger may develop suction that could cause the soil to intrude into the excavation. An insufficient head of concrete in the casing during its withdrawal could also allow the soils to intrude into the wet concrete. Both of these conditions may induce "necking", a section of reduced diameter, in the pier.

Slurry Method- As an alternate to the use of casing to install the pier foundations, water or a weighted drilling fluid may be considered. Slurry displacement drilling can only prevent sloughing and water influx but cannot control sloughing once it has occurred. Therefore, slurry displacement drilling techniques must begin at the ground surface, not after sloughing materials are encountered.

Typical drilling fluids include those which contain polymers or bentonite. If a polymer is used with "hard" mixing water, a water softening agent may be required to achieve intimate mixing and the appropriate viscosity. The polymer manufacturer should be consulted concerning proper use of the polymer. If bentonite slurry is used, the bentonite should be mixed with water several hours before placing in the pier excavation. Prior mixing gives the bentonite sufficient time to hydrate properly. The drilling fluid should only be of sufficient viscosity to control sloughing of the excavation walls and subsurface water flow into the excavation. Care should be exercised while extracting the auger so that suction does not develop and cause disturbance or create "necking" in the excavation walls as described above. Casing should not be employed in conjunction with the slurry drilling technique due to possible trapping of loose soils and slurry between the concrete and natural soil.

The use of weighted drilling fluid when installing drilled pier foundations requires extra effort to ensure an adequate bearing surface is obtained. A clean-out bucket should be used just prior to pier completion in order to remove any cuttings and loose soils which may have accumulated in the bottom of the excavation. Steel and concrete should be placed in the excavation immediately after pier completion. A closed-end tremie should be used to place the concrete completely to the bottom of the excavation in a controlled manner to effectively displace the slurry during concrete placement.

When the pier excavation depth is achieved and the bearing area has been cleaned, steel and concrete should then be placed immediately in the excavation. The concrete should be placed completely to the bottom of the excavation with a closed-end tremie in the pier excavation if more than 3 inches of water is ponded on the bearing surface or the slurry drilling technique is used. A short tremie may be used if the excavation has less than 3 inches of ponded water or if the water is pumped out prior to concrete placement. The fluid concrete should not be allowed to strike the pier reinforcement, temporary casing (if required) or excavation sidewalls during concrete placement.

All aspects of concrete design and placement should comply with the American Concrete Institute (ACI) 318 Code Building Code Requirements for Structural Concrete, ACI 336.1 Standard Specification for the Construction of Drilled Piers, and ACI 336.3R entitled Suggested Design and Construction Procedures for Pier Foundations. Concrete should be designed to achieve the specified 28-day strength when placed at a 7-inch slump with a ±1 inch tolerance. If a high range water reducer is used to achieve this slump, the span of slump retention for the

specific admixture under consideration should be thoroughly investigated. Compatibility with other concrete admixtures should also be considered. A technical representative of the admixture supplier should be consulted on these matters.

Successful installation of drilled piers is a coordinated effort involving the general contractor, design consultants, subcontractors and suppliers. Each must be properly equipped and prepared to provide their services in a timely fashion. Several key items of major concern are:

- Proper drilling rig with proper equipment (including casing and augers)
- Reinforcing steel cages tied to meet project specifications;
- Proper scheduling and ordering of concrete for the piers; and
- Monitoring of installation by design professionals.

Pier construction should be carefully monitored to assure compliance of construction activities with the appropriate specifications. Particular attention to the referenced publication is warranted for pier installation. A number of items of concern for pier installation include those listed below.

- Pier locations
- Vertical alignment
- Competent bearing
- Steel placement
- Concrete properties and placement
- Casing removal (if required)
- Slurry viscosity (if required)
- Proper casing seal for subsurface water control

If the contractor has to deviate from the recommended foundations, Terracon should be notified immediately so additional engineering recommendations can be provided for an appropriate foundation type.

4.3.2 Design Recommendations - Mat Foundation

A mat foundation may be utilized for some of the structures at the site. The Potential Vertical Rise (PVR) of the soils encountered at the site is about 1 to 1½ inches in present condition. Based on the stiffness of a mat foundation, we anticipate that a PVR of about 1 inch, the typical design movement for these types of structures, can be tolerated.

The mat can be designed as a uniform thick concrete member extending above final grade or can be designed as a less thick member with the main mat portion buried and skids extending above final grade to support the structure.

The mat should be analyzed using a soil-structure interaction program to identify areas of high contact stresses, excessive movements and large moments. The subgrade and select fill soils should be prepared as outlined in the **"4.2 Earthwork"** section of this report, which contains material and placement requirements for select fill, as well as other subgrade preparation recommendations.

Item	Description
Select Fill Pad	As needed (Min. 2 feet)
Minimum Mat Embedment Depth[1]	8 inches below final grade
Maximum Contact Pressures	2,500 psf
Modulus of Subgrade Reaction	75 pounds per cubic inch (pci)
Soil Unit Weight[2]	100 pounds per cubic foot (pcf)
Estimated PVR[3]	About 1 inch
Approximate total settlement [4]	About 1 inch
Estimated differential settlement[4]	Approximately ½ of total settlement

1. To bear within the select fill.
2. A buried mat would provide more resistance to uplift than the uniform thick mat since the weight of the soil overlying the mat would also be included in the uplift resistance computations. In addition to the weight of foundation and structure, any soil directly overlying the foundation may also be considered. The soil unit weight provided above may be assumed for the on-site soils placed above the footing, provided the fill is properly compacted.
3. The buried mat foundation system should be designed to tolerate the anticipated soil movement and provide satisfactory support to the proposed structures.
4. This estimated post-construction settlement is assuming proper construction practices are followed.

4.3.2.1 Mat Foundation Construction Considerations

Excavations for a mat foundation should be performed with equipment capable of providing a relatively clean bearing area. The bottom 6 inches of the excavations should be completed with a smooth-mouthed bucket or by hand labor.

The excavations should be neatly excavated and properly formed. If neat excavation is not possible then the foundation should be overexcavated and formed. All loose materials should be removed from the overexcavated areas and filled with lean concrete, compacted cement stabilized sand (two sacks cement to one cubic yard of sand) or flowable fill (ACI-229R).

Steel should be placed and the foundation poured the same day of excavation. If not, a seal slab consisting of lean concrete should be poured on the same day the contractor achieves the final bearing level in order to protect the exposed foundation soils. The bearing surface should be excavated with a slight slope to create an internal sump for runoff water collection and removal. If surface runoff water in excess of 2 inches accumulates at the bottom of the excavation, it should be pumped out prior to concrete placement. Under no circumstances should water be allowed to adversely affect the quality of the bearing surface.

The bearing surface of the foundation should be evaluated after excavation is completed and immediately prior to placing concrete.

4.3.3 Design Recommendations – Slab-on-grade Foundation System

The foundation design parameters presented below are based on our evaluation using published theoretical and empirical design methods.

These were developed based on our understanding of the proposed project, our interpretation of the information and data collected as a part of this study, our area experience and the results of our evaluation. The structural engineer should select the appropriate slab design method and code for the amount of anticipated slab movement indicated.

The slab-on-grade foundation may be designed using the following parameters provided the subgrade is prepared as outlined in the **"4.2 Earthwork"** section of this report:

Item	Description
Select Fill Pad	Minimum 2 feet
Bearing Pressure	Net total load – 2,500 psf
Climatic Rating, C_w	15
Design Plasticity Index	26
Soil Support Index	0.89
Estimated PVR[1]	About 1 inch or less
Approximate total settlement [2]	About 1 inch
Estimated differential settlement[2]	Approximately ½ of total settlement
Minimum perimeter grade beam embedment depth[3]	18 inches below final grade

1. The slab-on-grade foundation system should be designed to tolerate the anticipated soil movement and provide satisfactory support to the proposed building. The foundation should have adequate exterior and interior grade beams to provide sufficient rigidity to the foundation system such that the slab deflections that result are considered tolerable to the supported building.
2. This estimated post-construction settlement is assuming proper construction practices are followed.
3. To bear within the select fill. The grade beams may be thickened and widened where necessary to support concentrated loads.

4.3.3.1 Slab-on-grade Foundation Construction Considerations

Excavations for grade beams should be performed with equipment capable of providing a relatively clean bearing area. The bottom 6 inches of the excavations should be completed with a smooth-mouthed bucket or by hand labor. The excavations should be neatly excavated and properly formed. Debris in the bottom of the excavation should be removed prior to steel placement. Water should not be allowed to accumulate at the bottom of the excavation.

To reduce the potential for groundwater seepage into the excavations and to minimize disturbance to the bearing area, we recommend that concrete and steel be placed as soon as possible after the excavations are completed. Excavations should not be left open for more

than 36 hours. The bearing surface of the grade beams should be evaluated after excavation is completed and immediately prior to placing concrete.

4.3.4 Design Recommendations – Spread Footing Foundation System

A spread footing foundation system may also be used to support some of the planned structures that are not sensitive to movement. In addition to the normally consolidated clay soils, the near surface clay soils have a moderate to high potential for shrink/swell movements. The PVR of the soils in the general area of the substation is expected to be about 1 to 1½ inches. Some industrial structures may not be affected by movements of this magnitude and shallow spread footings might be a cost effective foundation option.

The spread footing foundation system may be designed using the following parameters provided the subgrade is prepared as outlined in the **"4.2 Earthwork"** section of this report:

Item	Description
Minimum embedment below finished grade[1]	2½ feet
Bearing Pressures	Net total load – 2,500 psf
Approximate total settlement [2]	About 1 inch
Estimated differential settlement[3]	Approximately ½ of total settlement
Allowable passive pressure[4]	650 psf (if considered)
Coefficient of sliding friction[5]	0.25
Uplift Resistance[6]	Foundation Weight (150 pcf) & Soil Weight (100 pcf)

1. To bear within the native soils.
2. This estimated post-construction settlement of the shallow footings is without considering the effect of stress distribution from adjacent foundations and assuming proper construction practices are followed. A clear distance between the footings of one footing size should not produce overlapping stress distributions and would essentially behave as independent foundations.
3. Differential settlements may result from variances in subsurface conditions, loading conditions and construction procedures. The settlement response of the footings will be more dependant upon the quality of construction than upon the response of the subgrade to the foundation loads. We estimate that the differential settlements should be approximately one-half of the total settlement.
4. The passive pressure along the exterior footings should be neglected unless pavement is provided up to the edge of the building. For interior footings, the allowable passive pressure may be used for the entire depth of the footing.
5. Lateral loads transmitted to the footings will be resisted by a combination of soil-concrete friction on the base of the footings and passive pressure on the side of the footings. We recommend that the allowable frictional resistance be limited to 250 psf.
6. The ultimate uplift capacity of shallow footings should be reduced by an appropriate factor of safety to compute allowable uplift capacity.

4.3.4.1 Spread Footing Foundation Construction Considerations

Excavations for shallow footings should be performed with equipment capable of providing a relatively clean bearing area. The bottom 6 inches of the excavations should be completed with a smooth-mouthed bucket or by hand labor. The excavations should be neatly excavated and properly formed. Debris in the bottom of the excavation should be removed prior to steel placement. Water should not be allowed to accumulate at the bottom of the excavation. To reduce the potential for groundwater seepage into the excavations and to minimize disturbance to the bearing area, we recommend that concrete and steel be placed as soon as possible after the excavations are completed. Excavations should not be left open for more than 36 hours. The bearing surface of the footings should be evaluated after excavation is completed and immediately prior to placing concrete.

4.4 Foundation Construction Monitoring

The performance of the foundation system for the proposed structures will be highly dependent upon the quality of construction. Thus, we recommend that fill pad compaction and foundation installation be monitored full time by an experienced Terracon soil technician under the direction of our Geotechnical Engineer. During foundation installation, the base should be monitored to evaluate the condition of the subgrade. We would be pleased to develop a plan for compaction and foundation installation monitoring to be incorporated in the overall quality control program.

4.5 Seismic Considerations

Description	Value
2012/15 International Building Code Site Classification (IBC) [1]	D [2]
Site Latitude (Degrees)	25.912470° N
Site Longitude (Degrees)	-97.519160° W
Mapped Spectral Acceleration for Short Periods (0.2-Second): (S_S) [3]	0.034 g
Mapped Spectral Acceleration for a 1-Second Period: (S_1) [3]	0.013 g

1	The site class definition was determined using SPT N-values in conjunction with section 1613.3.2 in the 2012/15 IBC and Table 20.3-1 in the 2010 ASCE-7.
2	Section 20.1 in the 2010 ASCE-7 requires a site soil profile determination extending to a depth of 100 feet for seismic site classification. The current scope does not include the required 100 foot soil profile determination. Borings extended to a maximum depth of 50 feet, and this seismic site class definition considers that similar soil continues below the maximum depth of the subsurface exploration. Additional exploration to deeper depths would be needed to confirm the conditions below the current depth of exploration.
3	The Spectral Acceleration values were determined using publicly available information provided on the United States Geological Survey (USGS) website. The spectral acceleration values can be used to determine the site coefficients using Tables 1613.3.3 (1) and 1613.3.3 (2) in the 2012/15 IBC.

4.6 Floor Slab

Planned finished grades at the site were not available at the time of this report. We anticipate that the finished floor elevation for the proposed structures is planned to be at least 2 feet above existing grade. If cuts and/or significant fills are planned, Terracon should be notified to review and/or modify our recommendations given in this subsection.

4.6.1 Design Recommendations – Floor Slab

The subsurface soils at this site generally exhibit moderate to high expansion potential. Based on the information developed from our field and laboratory programs and on method TEX-124-E in the Texas Department of Transportation (TxDOT) Manual of Testing Procedures, we estimate that the subgrade soils at this site exhibit a Potential Vertical Rise (PVR) of about 1 to 1½ inches in present condition.

The actual movements could be greater if poor drainage, ponded water, and/or other sources of moisture are allowed to infiltrate beneath the structure after construction.

A select fill pad of at least 24 inches over a minimum of 6 inches of moisture conditioned subgrade should be constructed directly below the floor slab and should also extend a minimum of 3 feet beyond the edge of the proposed control house building. The final exterior grade adjacent to the building should be sloped to promote positive drainage away from the structure.

The subgrade and select fill soils should be prepared as outlined in the **"4.2 Earthwork"** section of this report, which contains material and placement requirements for select fill, as well as other subgrade preparation recommendations. The floor slab should be designed using the following recommendations.

ITEM	DESCRIPTION
Excavation	Min. 6 inches
Floor slab support	Min. 24-inch select fill building pad over a minimum of 6 inches of moisture conditioned subgrade (plus additional, if needed, to achieve Finish Floor Elevation)
Modulus of subgrade reaction	100 pounds per cubic inch (pci)
Estimated Potential Vertical Rise (PVR)	About 1 inch or less

5.0 GENERAL COMMENTS

Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications.

Terracon also should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of weather.

The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made.

Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX A

FIELD EXPLORATION

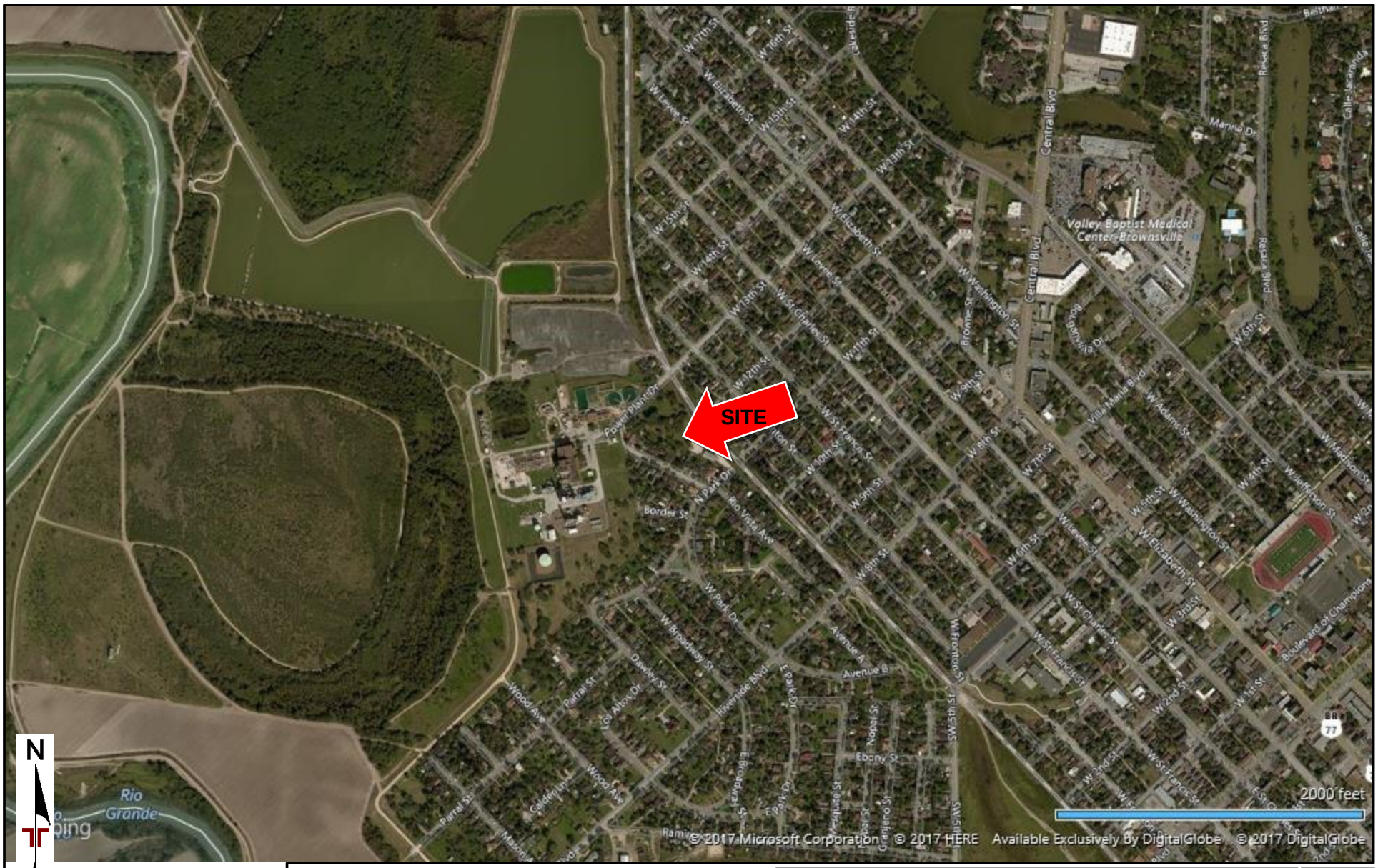


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager:	SC
Drawn by:	SC
Checked by:	AAS
Approved by:	AAS

Project No.	88175125
Scale:	NTS
File Name:	88175125
Date:	9/28/17

Terracon
1506 Mid Cities Dr
Pharr, TX 78577-2128

SITE LOCATION

Rio Grande Distribution Substation
Power Plant Drive and Rio Vista Ave
Brownsville, TX

Exhibit

A-1

AERIAL PHOTOGRAPHY PROVIDED BY MICROSOFT BING MAPS

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: SC	Project No. 88175125
Drawn by: SC	Scale: AS SHOWN
Checked by: AAS	File Name: 88175125
Approved by: AAS	Date: 9/28/17

Terracon
1506 Mid Cities Dr
Pharr, TX 78577-2128

EXPLORATION PLAN

Rio Grande Distribution Substation
Power Plant Drive and Rio Vista Ave
Brownsville, TX

Exhibit
A-2

Field Exploration Description

Subsurface conditions were evaluated by drilling three test borings to depths of 50 feet within the proposed substation areas. The borings were drilled using truck-mounted drilling equipment at the approximate location shown on the Boring Location Plan, Exhibit A-2 of Appendix A. The borings were located by measuring without the use of surveying equipment. The boring depths were measured from the existing ground surface at the time of our field activities. At the completion of our field activities, the borings were backfilled with soil cuttings.

The Logs of Borings, presenting the subsurface soil descriptions, type of sampling used, and additional field data, are presented on Exhibits A-4 through A-6 of Appendix A. The General Notes, which defines the terms used on the log, are presented on Exhibit D-1. The Unified Soil Classification System is presented on Exhibit D-2 of Appendix D.

Cohesive soil samples were generally recovered using open-tube samplers. Pocket penetrometer tests were performed on samples of cohesive soils to serve as a general measure of consistency.

Granular soils and soils for which good quality open-tube samples could not be recovered were sampled by means of the Standard Penetration Test (SPT). This test consists of measuring the number of blows (N) required for a 140-pound hammer free falling 30 inches to drive a standard split-spoon sampler 12 inches into the subsurface material after being seated six inches. This blow count or SPT N-value is used to evaluate the stratum.

Samples were removed from samplers in the field, visually classified, and appropriately sealed in sample containers to preserve their in-situ moisture contents. Samples were returned to our laboratory in Pharr, Texas.

BORING LOG NO. B-1

PROJECT: Rio Grande Distribution Substation

CLIENT: Brownsville Public Utilities Board
Brownville, TX

SITE: Power Plant Drive and Rio Vista Ave
Brownsville, TX

LOCATION See Exhibit A-2

Latitude: 25.914270° Longitude: -97.519160°

DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS: LL-PL-PI	PERCENT FINES
	SANDY LEAN CLAY (CL), brown, stiff to very stiff				4-5-7 N=12				13			
					3-3-5 N=8				26		39-17-22	
		5			5-7-10 N=17				18			
6.0	**FAT CLAY (CH)**, light brown to tan, stiff to very stiff				4-4-5 N=9				21		57-18-39	
			▽		6-6-10 N=16				21			98
		10	▽									
		15	▼		6-4-7 N=11				25		64-23-41	
		20			4-5-6 N=11				20			
		25			5-5-6 N=11				22			99
	- with Lean Clay (CL) seams at 28½ feet	30			3-4-5 N=9				24		46-17-29	
		35			5-5-8 N=13				25			
		40			5-8-11 N=19				21			98
		45			6-8-11 N=19				19		56-19-37	
50.0	- with Silty Sand (SM) seams at 48½ feet				7-8-11 N=19				24			43
	Boring Terminated at 50 Feet	50										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Dry augered to 50 feet

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

- ▽ *While drilling*
- ▽ *After 15 minutes*
- ▼ *At completion of drilling*
- *Cave-in depth*

Terracon
1506 Mid Cities Dr
Pharr, TX

Boring Started: 09-14-2017
Boring Completed: 09-14-2017
Drill Rig: CME 55
Driller: SWD
Project No.: 88175125
Exhibit: A-4

BORING LOG NO. B-2

PROJECT: Rio Grande Distribution Substation

CLIENT: Brownsville Public Utilities Board
Brownville, TX

SITE: Power Plant Drive and Rio Vista Ave
Brownsville, TX

LOCATION See Exhibit A-2

Latitude: 25.914080° Longitude: -97.519230°

DEPTH	Description	Depth (Ft.)	Field Test Results	Strength Test: Test Type	Strength Test: Compressive Strength (tsf)	Strength Test: Strain (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
	SANDY LEAN CLAY (CL), brown, medium stiff to hard		4-4-3 N=7				14		37-18-19	
			4.5+ (HP)				15			
4.0	**FAT CLAY (CH)**, light brown to tan, stiff to very stiff	5	3.5 (HP)		3.10	9	21	103	64-19-45	
			3.5 (HP)				19			98
		10	2.75 (HP)		2.73	9	20	105		
		15	2.5 (HP)				24		51-20-31	
		20	3.75 (HP)		2.71	9	18	108		
		25	3.0 (HP)				29		60-20-40	
		30	1.25 (HP)				28			91
		35	3.0 (HP)		2.50	5.8	20	105		
		40	4.5+ (HP)				28		52-18-34	
		45	5-6-8 N=14				18			91
	- with Lean Clay (CL) seams at 48½ feet									
50.0	***Boring Terminated at 50 Feet***	50	5-6-8 N=14				22		41-15-26	

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Dry augered to 50 feet

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

- *While drilling*
- *After 15 minutes*
- *At completion of drilling*
- *Cave-in depth*

Terracon
1506 Mid Cities Dr
Pharr, TX

Boring Started: 09-14-2017
Boring Completed: 09-14-2017
Drill Rig: CME 55
Driller: SWD
Project No.: 88175125
Exhibit: A-5

BORING LOG NO. B-3

PROJECT: Rio Grande Distribution Substation

CLIENT: Brownsville Public Utilities Board
Brownville, TX

SITE: Power Plant Drive and Rio Vista Ave
Brownsville, TX

LOCATION See Exhibit A-2

Latitude: 25.914320° Longitude: -97.519290°

DEPTH	Description	DEPTH (Ft.)	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS: LL-PL-PI	PERCENT FINES
	LEAN CLAY (CL), brown, medium stiff		3-4-3 N=7				16		43-19-24	
4.0	**FAT CLAY (CH)**, brown to light brown, medium stiff to very stiff		2-2-2 N=4				26			89
		5	2-3-3 N=6				22		53-21-32	
			4-5-7 N=12				22			
		10	4-7-10 N=17				22		65-24-41	
	- grades to light brown and tan below 13½ feet	15	4-4-6 N=10				28			99
		20	4-5-6 N=11				20			
	- with Lean Clay (CL) seams at 23½ feet	25	3-4-4 N=8				22		41-17-24	
		30	4-4-5 N=9				23			97
		35	5-7-9 N=16				21		58-24-34	
		40	6-9-11 N=20				22			
	- with Lean Clay (CL) seams at 43½ feet	45	8-10-12 N=22				20		37-16-21	
50.0		50	7-9-11 N=20				18			
	Boring Terminated at 50 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Dry augered to 50 feet

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

- *While drilling*
- *After 15 minutes*
- *At completion of drilling*
- *Cave-in depth*

Terracon
1506 Mid Cities Dr
Pharr, TX

Boring Started: 09-14-2017
Boring Completed: 09-14-2017
Drill Rig: CME 55
Driller: SWD
Project No.: 88175125
Exhibit: A-6

APPENDIX B

LABORATORY TESTING

Laboratory Testing

Soil samples were tested in the laboratory to measure their dry unit weight and natural water content. Unconfined compression tests were performed on selected samples and a calibrated hand penetrometer was used to estimate the approximate unconfined compressive strength of some cohesive samples. Selected samples were also classified using the results of Atterberg limit testing. The calibrated hand penetrometer has been correlated with unconfined compression tests and provides a better estimate of soil consistency than visual examination alone. The test results are provided on the boring logs included in Appendix A.

Descriptive classifications of the soils indicated on the boring logs are in accordance with the enclosed General Notes and the Unified Soil Classification System. Also shown are estimated Unified Soil Classification Symbols. A brief description of this classification system is attached to this report. All classification was by visual manual procedures.

Rio Grande Distribution Substation

Power Plant Drive and Rio Vista Avenue

Brownsville, Texas

Project No.: 88175125

SWELL TEST SUMMARY

			INITIAL CONDITIONS		FINAL CONDITIONS			
Boring No.	Depth (feet)	Overburden Pressure (psf)	Moisture Content (%)	γ_d (pcf)	Moisture Content (%)	γ_d (pcf)	Moisture Gain (%)	Percent Swell
B-2	3.0	100	19.0	103.2	26.8	77.2	7.8	5.40
B-2	7.0	100	21.7	102.8	24.7	81.5	3.1	1.10

Rio Grande Distribution Substation

Power Plant Drive and Rio Vista Avenue

Brownsville, Texas

Project No.: 88175125

SWELL TEST SUMMARY

			INITIAL CONDITIONS		FINAL CONDITIONS			
Boring No.	Depth (feet)	Overburden Pressure (psf)	Moisture Content (%)	γ_d (pcf)	Moisture Content (%)	γ_d (pcf)	Moisture Gain (%)	Percent Swell
B-2	3.0	460	18.8	103.2	25.1	80.0	6.3	3.10
B-2	7.0	940	21.3	100.9	23.5	81.2	2.2	0.60

B-1
AXIAL CAPACITY ANALYSES
DESIGN SOIL PARAMETERS FOR
UNDRAINED CONDITIONS
Rio Grande Distribution Substation
Brownsville, Texas

Soil Layer	Depth to Bottom of Soil Layer (feet)	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Adhesion Factor (-)	Friction Angle (degrees)	Horizontal Stress Coefficient (-)	Bearing Capacity Factors N_c[4] (-)	N_q (-)	N_g (-)
1	4	115	1,000	----	0	----	6	1	0
2	10	120	2,000	0.49	0	----	6	1	0
3	35	56	1,500	0.58	0	----	9	1	0
4	50	60	2,500	0.44	0	----	9	1	0

NOTES:

1. Design depth to subsurface water is about 10 feet.
2. For uplift conditions, the computed skin friction should be multiplied by 0.7 for sands and 0.9 for clay.
3. The unit allowable end bearing should not exceed 100 kips per square foot.
4. The N_c value of 9 for non-granular soils is for D/B ratios greater than 4. Otherwise, use N_c = 6.

B-2

AXIAL CAPACITY ANALYSES

DESIGN SOIL PARAMETERS FOR
UNDRAINED CONDITIONS

Rio Grande Distribution Substation

Brownsville, Texas

Soil Layer	Depth to Bottom of Soil Layer (feet)	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Adhesion Factor (-)	Friction Angle (degrees)	Horizontal Stress Coefficient (-)	Bearing Capacity Factors N_c[4] (-)	N_q (-)	N_g (-)
1	4	115	1,000	----	0	----	6	1	0
2	10	125	3,000	0.40	0	----	6	1	0
3	40	63	2,750	0.42	0	----	9	1	0
4	50	58	2,000	0.49	0	----	9	1	0

NOTES:

1. Design depth to subsurface water is about 10 feet.
2. For uplift conditions, the computed skin friction should be multiplied by 0.7 for sands and 0.9 for clay.
3. The unit allowable end bearing should not exceed 100 kips per square foot.
4. The N_c value of 9 for non-granular soils is for D/B ratios greater than 4. Otherwise, use N_c = 6.

B-3
AXIAL CAPACITY ANALYSES
DESIGN SOIL PARAMETERS FOR
UNDRAINED CONDITIONS
Rio Grande Distribution Substation
Brownsville, Texas

Soil Layer	Depth to Bottom of Soil Layer (feet)	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Adhesion Factor (-)	Friction Angle (degrees)	Horizontal Stress Coefficient (-)	Bearing Capacity Factors N_c[4] (-)	N_q (-)	N_g (-)
1	6	110	750	----	0	----	6	1	0
2	10	118	1,500	0.58	0	----	6	1	0
3	30	53	1,250	0.65	0	----	9	1	0
4	50	60	2,500	0.44	0	----	9	1	0

NOTES:

1. Design depth to subsurface water is about 10 feet.
2. For uplift conditions, the computed skin friction should be multiplied by 0.7 for sands and 0.9 for clay.
3. The unit allowable end bearing should not exceed 100 kips per square foot.
4. The N_c value of 9 for non-granular soils is for D/B ratios greater than 4. Otherwise, use N_c = 6.

B-1
LATERAL CAPACITY ANALYSES
DESIGN SOIL PARAMETERS FOR
UNDRAINED CONDITIONS
Rio Grande Distribution Substation
Brownsville, Texas

Soil Layer	LPILE Soil Type	Depth to Soil Layer Top (feet)	Depth to Soil Layer Bottom (feet)	LPILE Soil Modulus k (pci)	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Internal Friction Angle (degrees)	LPILE Soil Strain Factor e_{50}	FAD Tools Strength Reduction Factor (--)	FAD Tools Deformation Modulus (ksi)
1	Stiff Clay without Free Water	0	4	428	115	1,000	0	0.010	----	0.55
2	Stiff Clay without Free Water	4	10	622	120	2,000	0	0.007	0.49	1.20
3	Stiff Clay without Free Water	10	35	525	56	1,500	0	0.008	0.58	0.85
4	Stiff Clay without Free Water	35	50	718	60	2,500	0	0.006	0.44	1.50

NOTES:

1. Design depth to subsurface water is about 10 feet.
2. FAD Tools deformation moduli values based on *FAD 5.1 User Guide, December 2015*

B-2
LATERAL CAPACITY ANALYSES
DESIGN SOIL PARAMETERS FOR
UNDRAINED CONDITIONS
Rio Grande Distribution Substation
Brownsville, Texas

Soil Layer	LPILE Soil Type	Depth to Soil Layer Top (feet)	Depth to Soil Layer Bottom (feet)	LPILE Soil Modulus k (pci)	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Internal Friction Angle (degrees)	LPILE Soil Strain Factor e_{50}	FAD Tools Strength Reduction Factor (--)	FAD Tools Deformation Modulus (ksi)
1	Stiff Clay without Free Water	0	4	428	115	1,000	0	0.010	----	0.55
2	Stiff Clay without Free Water	4	10	815	125	3,000	0	0.006	0.40	1.75
3	Stiff Clay without Free Water	10	40	767	63	2,750	0	0.006	0.42	1.60
4	Stiff Clay without Free Water	40	50	622	58	2,000	0	0.007	0.49	1.20

NOTES:

1. Design depth to subsurface water is about 10 feet.
2. FAD Tools deformation moduli values based on *FAD 5.1 User Guide, December 2015*

B-3
LATERAL CAPACITY ANALYSES
DESIGN SOIL PARAMETERS FOR
UNDRAINED CONDITIONS
Rio Grande Distribution Substation
Brownsville, Texas

Soil Layer	LPILE Soil Type	Depth to Soil Layer Top (feet)	Depth to Soil Layer Bottom (feet)	LPILE Soil Modulus k (pci)	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Internal Friction Angle (degrees)	LPILE Soil Strain Factor e_{50}	FAD Tools Strength Reduction Factor (--)	FAD Tools Deformation Modulus (ksi)
1	Stiff Clay without Free Water	0	4	380	110	750	0	0.012	----	0.40
2	Stiff Clay without Free Water	4	10	525	118	1,500	0	0.008	0.58	0.85
3	Stiff Clay without Free Water	10	30	477	53	1,250	0	0.009	0.65	0.70
4	Stiff Clay without Free Water	30	50	718	60	2,500	0	0.006	0.44	1.50

NOTES:

1. Design depth to subsurface water is about 10 feet.
2. FAD Tools deformation moduli values based on *FAD 5.1 User Guide, December 2015*

Client Sample Results

Client: Terracon Consulting Eng & Scientists
Project/Site: Brownsville, TX

TestAmerica Job ID: 560-69924-1

Client Sample ID: B-1 4-6

Date Collected: 09/11/17 08:00
Date Received: 09/20/17 09:45

Lab Sample ID: 560-69924-1

Matrix: Solid

Method: 9056A - Anions, Ion Chromatography - Soluble

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Chloride	2900		200	27	mg/Kg			09/26/17 01:44	50
Sulfate	1700		250	48	mg/Kg			09/26/17 01:44	50

General Chemistry

Analyte	Result	Qualifier	RL	RL	Unit	D	Prepared	Analyzed	Dil Fac
pH	8.0	H	0.01	0.01	SU			09/26/17 14:25	1

General Chemistry - Soluble

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Resistivity	1.5		0.99	0.64	ohm-m			09/26/17 23:30	1

4370 Contractors Common
Livermore, CA 94551
Tel: 925-999-9232
Fax: 925-999-8837
info@geothermusa.com

September 28, 2017

Terracon Consultants, Inc.
1506 Mid Cities Drive
Pharr, Texas 78577
Attn: Stephany Chacón, E.I.T.

Re: Thermal Analysis of Native Soil
Rio Grande Distribution Substation – Brownsville, TX (Project No. 88175125)

The following is the report of thermal dryout characterization tests conducted on one (1) native soil sample from the referenced project sent to our laboratory.

Thermal Resistivity Tests: For thermal dryout characterization the sample was tested at its 'optimum' moisture content and 95% of the maximum dry density provided by **Terracon**. A series of thermal resistivity measurements were made in stages with moisture content ranging from 'wet' to the totally dry condition. The tests were conducted in accordance with the IEEE standard 442. The results are tabulated below and the thermal dry out curve is presented in **Figure 1.**

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

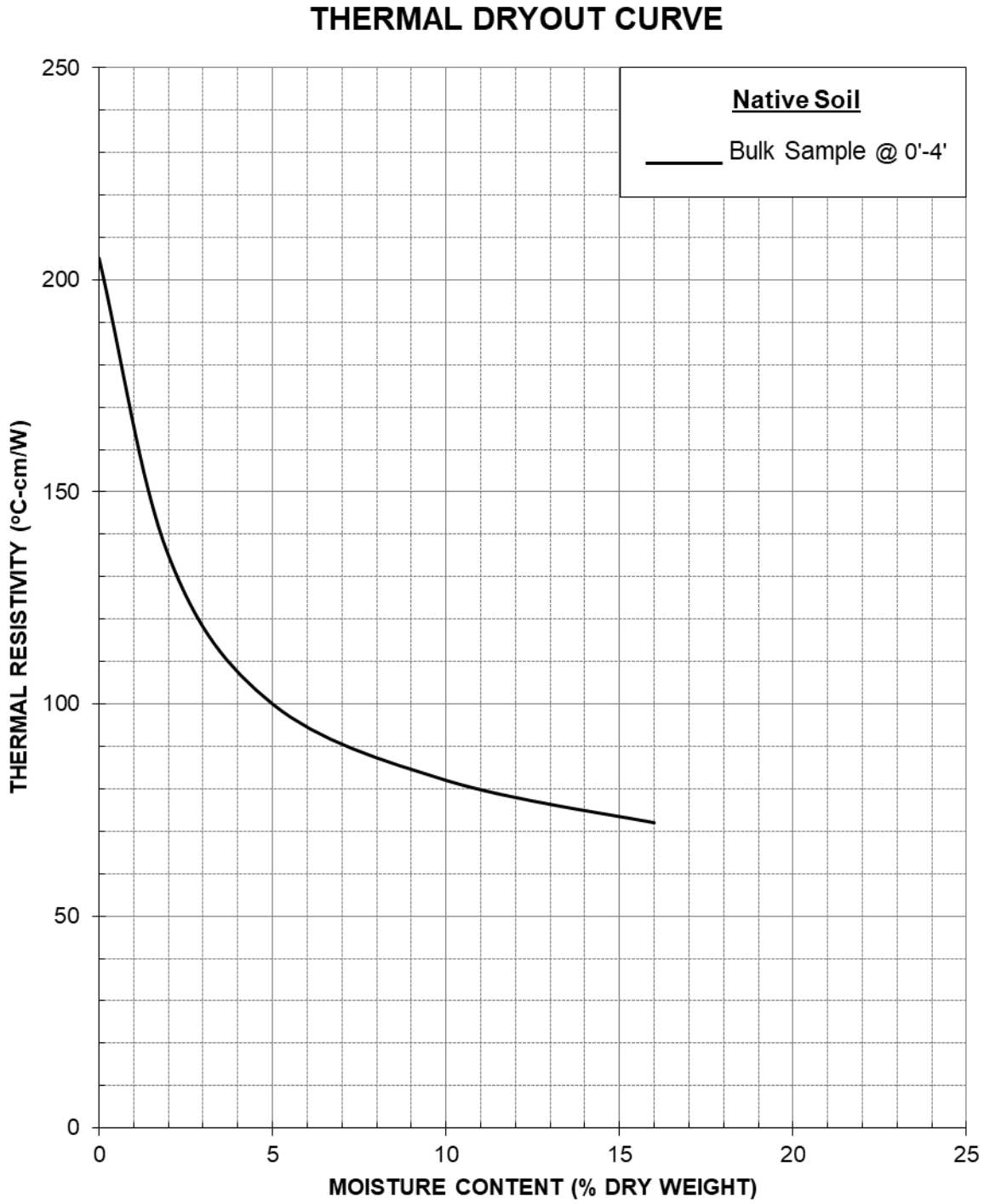
Sample ID	Description (Terracon)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
		Wet	Dry		
Bulk Sample	Dark Brown Lean Clay with Sand	72	205	19	93

Comments: The thermal characteristic depicted in the dryout curve applies for the sample at the test dry density.

Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA

Nimesh Patel



Terracon Consultants Inc

Thermal Analysis of Native Soil

Rio Grande Distribution Substation – Brownsville, TX (Project No. 88175125)

September 2017 **Figure 1**

Exhibit B-12

APPENDIX C

ELECTRICAL RESISTIVITY TESTS

Electrical Earth Resistivity #1

Project Name: Rio Grande Distribution Substation

Project No: 88175125 Date: 9/14/2017

Boring No.: B-1 Latitude: 25.91426 °

Test No. and Direction: 1 - North - South Longitude: -97.51924 °

Personnel: George Flores, Jr.

Method used for measurements: Wenner Array (4 Pin) ASTM G 57

Instrument Used:

Manufacturer: AEMC
Model No.: 6470-B
Serial No.: 230351 HDDV

Test No.	A Spacing (feet)	Meter Reading	Scale	Resistance Apparent	Multiplier	Earth Resistivity OHM-CM
1	5	0.72	1.00	0.72	958	686
2	10	0.31	1.00	0.31	1,915	592
3	20	0.14	1.00	0.14	3,830	521
4	40	0.06	1.00	0.06	7,660	467
5	60	0.04	1.00	0.04	11,491	425
6	80	0.04	1.00	0.04	15,321	628
7	100	0.04	1.00	0.04	19,151	766

Electrical Earth Resistivity #2

Project Name: Rio Grande Distribution Substation

Project No: 88175125 Date: 9/14/2017

Boring No.: B-1 Latitude: 25.91426 °

Test No. and Direction: 2 - East - West Longitude: -97.51924 °

Personnel: George Flores, Jr.

Method used for measurements: Wenner Array (4 Pin) ASTM G 57

Instrument Used:

Manufacturer: Megger
Model No.: DET 4TC
Serial No.: 080108/1319

Test No.	A Spacing (feet)	Meter Reading	Scale	Resistance Apparent	Multiplier	Earth Resistivity OHM-CM
1	5	0.85	1.00	0.85	958	812
2	10	0.26	1.00	0.26	1,915	488
3	20	0.13	1.00	0.13	3,830	506
4	40	0.04	1.00	0.04	7,660	283
5	60	0.02	1.00	0.02	11,491	195
6	80	0.02	1.00	0.02	15,321	368
7	100	0.04	1.00	0.04	19,151	728

APPENDIX D

SUPPORTING DOCUMENTS

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING

Auger	Split Spoon
Shelby Tube	Macro Core
Ring Sampler	Rock Core
Grab Sample	No Recovery

WATER LEVEL

- Water Initially Encountered
- Water Level After a Specified Period of Time
- Water Level After a Specified Period of Time

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

FIELD TESTS

(HP)	Hand Penetrometer
(T)	Torvane
(b/f)	Standard Penetration Test (blows per foot)
(PID)	Photo-Ionization Detector
(OVA)	Organic Vapor Analyzer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance Includes gravels, sands and silts.			CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			
Descriptive Term (Density)	Standard Penetration or N-Value Blows/Ft.	Ring Sampler Blows/Ft.	Descriptive Term (Consistency)	Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-Value Blows/Ft.	Ring Sampler Blows/Ft.
Very Loose	0 - 3	0 - 6	Very Soft	less than 500	0 - 1	< 3
Loose	4 - 9	7 - 18	Soft	500 to 1,000	2 - 4	3 - 4
Medium Dense	10 - 29	19 - 58	Medium-Stiff	1,000 to 2,000	4 - 8	5 - 9
Dense	30 - 50	59 - 98	Stiff	2,000 to 4,000	8 - 15	10 - 18
Very Dense	> 50	≥ 99	Very Stiff	4,000 to 8,000	15 - 30	19 - 42
			Hard	> 8,000	> 30	> 42

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

Exhibit D-1

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]					Soil Classification	
					Group Symbol	Group Name [B]
Coarse Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]		GW	Well-graded gravel [F]
			$Cu < 4$ and/or $1 > Cc > 3$ [E]		GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH		GM	Silty gravel [F,G,H]
			Fines classify as CL or CH		GC	Clayey gravel [F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]		SW	Well-graded sand [I]
			$Cu < 6$ and/or $1 > Cc > 3$ [E]		SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH		SM	Silty sand [G,H,I]
			Fines Classify as CL or CH		SC	Clayey sand [G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A" line [J]		CL	Lean clay [K,L,M]
			PI < 4 or plots below "A" line [J]		ML	Silt [K,L,M]
		Organic:		< 0.75	OL	Organic clay [K,L,M,N]
						Organic silt [K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line		CH	Fat clay [K,L,M]
			PI plots below "A" line		MH	Elastic Silt [K,L,M]
		Organic:	Liquid limit - oven dried	< 0.75	OH	Organic clay [K,L,M,P]
			Liquid limit - not dried			Organic silt [K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor				PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.

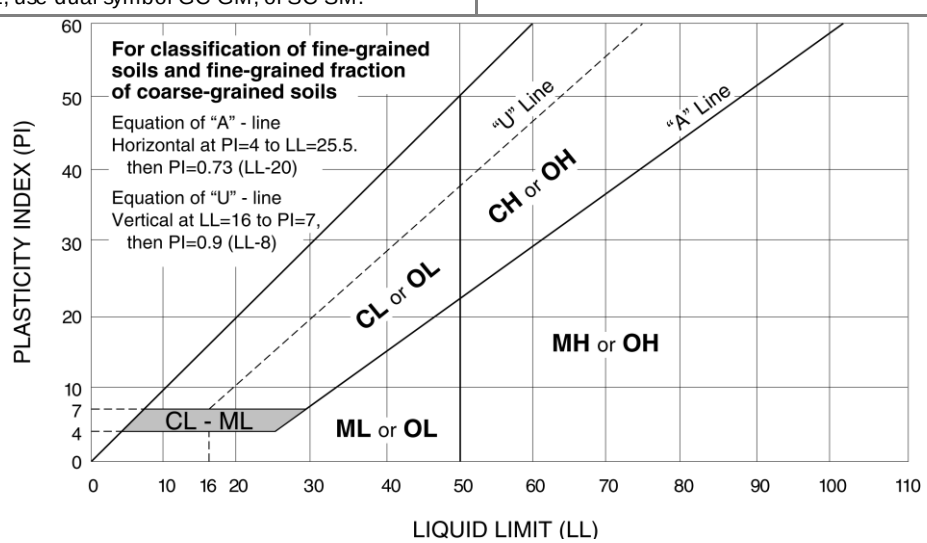


Exhibit D-2