NOTES:
Bearings are rotated to grid north, NAD83, Texas State Plane Central Zone.

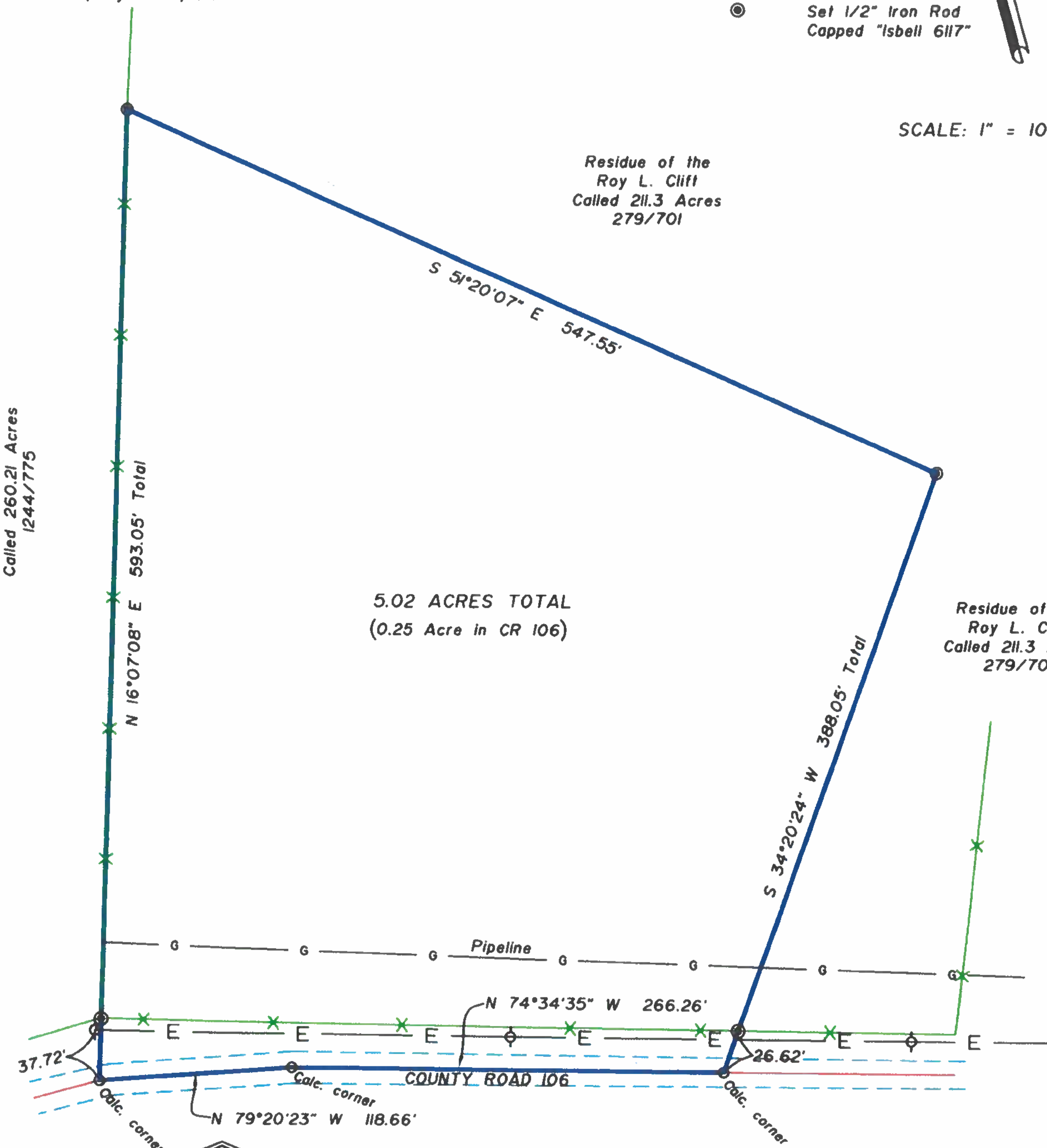
This survey was performed without the benefit of a title commitment. There may be easements affecting this property that are not shown.

Field notes accompany this plat.

LEGEND:

- Power Pole
- Wire Fence
- Electric Line
- Set 1/2" Iron Rod Capped "Isbell 6117"

SCALE: 1" = 100'



STATE OF TEXAS
REGISTERED
SHANE ALAN ISBELL
6117
PROFESSIONAL
LAND SURVEYOR

Registered Professional Land Surveyor No. 6117

SURVEY PLAT OF
5.02 ACRES
F. DEL VALLE SURVEY, A - 29
LEON COUNTY, TEXAS
JULY 24, 2025

PREPARED BY:
ISBELL LAND SURVEYING
1366 CR 320/CENTERVILLE, TX/Ph. 979-255-9177
FIRM REGISTRATION NO. 10192000
JOB NO. 25044

FIELD NOTES
5.02 ACRES
F. DEL VALLE SURVEY, A – 29
LEON COUNTY, TEXAS
JULY 24, 2025

All that certain lot, tract or parcel of land being 5.02 acres in the F. Del Valle Survey, Abstract No. 29, Leon County, Texas, and being a part of the Called 211.3 acre tract of land described in the deed from W. H. Worrell and wife, Etta Wanda Worrell to Roy L. Clift in Volume 279, Page 701 of the Deed Records of Leon County, Texas, said 5.02 acre tract of land being more particularly described by metes and bounds as follows:

BEGINNING at a calculated point in the center of County Road 106 for the west corner, being the calculated west corner of the Called 211.3 acre tract and the calculated south corner of the Called 260.21 acre tract in Volume 1244, Page 775;

THENCE N 16 ° 07 ' 08 " E, at a distance of 37.72 feet passing a ½" Iron Rod Capped "Isbell 6117" set for reference and continuing along the occupied line between the Called 211.3 acre tract and Called 260.21 acre tract for a total distance of 593.05 feet to a ½" Iron Rod Capped "Isbell 6117" set for the north corner;

THENCE S 51 ° 20 ' 07 " E, 547.55 feet to a ½" Iron Rod Capped "Isbell 6117" set for the east corner;

THENCE S 34 ° 20 ' 24 " W, 388.05 feet to a calculated point in the center of County Road 106 for the south corner, being in the calculated south line of the Called 211.3 acre tract from which a ½" Iron Rod Capped "Isbell 6117" set for reference bears N 34 ° 20 ' 24 " E a distance of 26.62 feet;

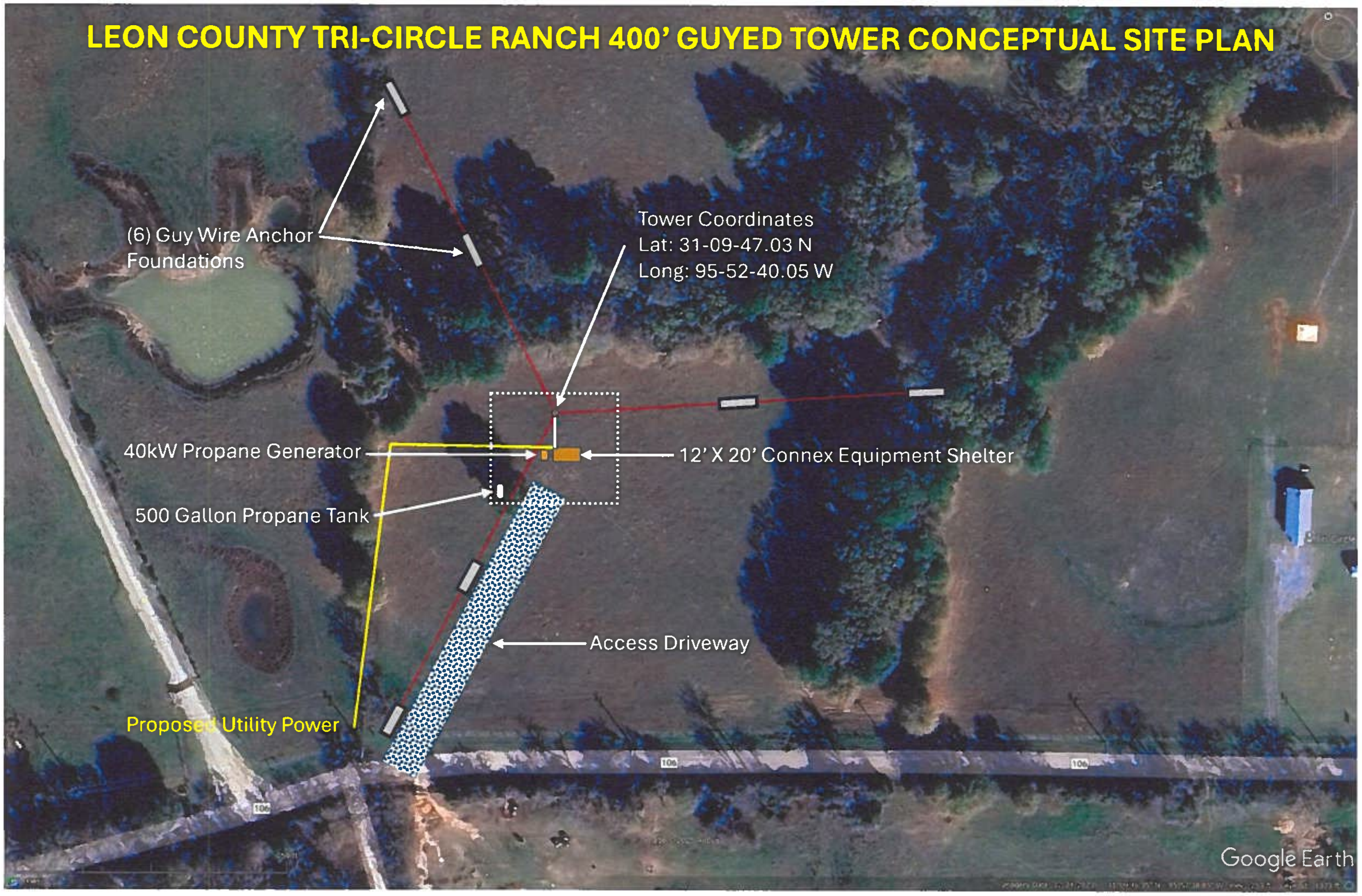
THENCE N 74 ° 34 ' 35 " W, 266.26 feet along the centerline of County Road 106 to a calculated point for corner;

THENCE N 79 ° 20 ' 23 " W, 118.66 feet along the centerline of County Road 106 and the calculated south line of the Called 211.3 acre tract TO THE POINT OF BEGINNING AND CONTAINING AN AREA OF 5.02 ACRES OF LAND, MORE OR LESS, according to a survey performed by Shane A. Isbell, Registered Professional Land Surveyor No. 6117. Bearings are rotated to grid north, NAD83, Texas State Plane Central Zone. For other information, see accompanying plat.

Shane A. Isbell

RPLS No. 6117
Isbell Land Surveying
Firm No. 10192000
Job No. 25044

STATE OF TEXAS
REGISTERED
SHANE ALAN ISBELL
6117
PROFESSIONAL
LAND SURVEYOR


LEON COUNTY TRI-CIRCLE RANCH 400' GUYED TOWER CONCEPTUAL SITE PLAN
(6) Guy Wire Anchor Foundations
Tower Coordinates
Lat: 31-09-47.03 N
Long: 95-52-40.05 W
40kW Propane Generator
12' X 20' Connex Equipment Shelter
500 Gallon Propane Tank
Access Driveway
Proposed Utility Power
Google Earth

July 29, 2025

TJ Foley
Leon County Commissioner's Office
PO Box 898
Centerville, Texas 75833

SUBJECT: Leon County – Tri-City Ranch Tower
Leona, Texas
Geotechnical Investigation
ETTL Job No. G 6564-25

Mr. Foley:

Submitted herein is the report summarizing the results of a geotechnical investigation conducted at the site of the above-referenced project.

If you have any questions regarding this report or if we can be of further assistance during construction, please do not hesitate to contact us. We are available to perform any construction materials testing and inspection services that you may require. Thank you for the opportunity to be of service.

Sincerely,

ETTL Engineers & Consultants Inc.
Texas Registered Engineering Firm #F3208

James Werbiski	Owen Sanderson
James Werbiski, E.I.T.	Owen B. Sanderson, P.E.
Project Manager	Senior Engineer

STATE OF TEXAS
OWEN B. SANDERSON
120504
LICENSED
PROFESSIONAL ENGINEER

07/29/2025

Distribution: (PDF) Mission Critical Partners, LLC

Geotechnical Investigation

Leon County
Tri-City Ranch Tower

Leona, Texas

Submitted To:

TJ Foley
Leon County
Centerville, Texas

Prepared by;

ETTL Engineers & Consultants Inc.
Tyler, Texas

July 2025

TABLE OF CONTENTS

1.0 INTRODUCTION

This study was performed at the request and authorization of TJ Foley, Leon County Commissioners' Office, Centerville, Texas, per our proposal dated May 12th, 2025. The field operations were conducted on July 7th, 2025.

The purpose of this investigation was to define and evaluate the general subsurface conditions of the proposed tower site located at the Tri-City Ranch in Leona, Texas. A site map depicting the project location is included in **APPENDIX A**, as is a Plan of Borings depicting the boring locations selected to cover the proposed site.

Specifically, the study was planned to determine the following:

- Subsurface stratigraphy within the limits of exploratory borings.
- Classification, strength, expansive properties, and compressibility characteristics of the foundation soils.
- Subgrade preparation and fill placement recommendations.
- Suitable foundation types and recommended allowable loading.
- Construction-related issues that may be anticipated by the investigation.

The investigation was carried out in three phases: 1) field exploration, sampling, and testing; 2) laboratory testing; and 3) engineering evaluation of data, the details of which are set forth in the following sections.

A variety of tests were conducted on selected soil samples to provide the data used as the basis for the conclusions and recommendations of this study. The conclusions and recommendations that follow are based on limited information regarding site grading. The boring locations were located based on provided GPS coordinates from the client's representative. ETTL did not confirm by survey that the locations indicated on the Plan of Borings or the elevations stated herein accurately reflect the locations on the ground.

2.0 PROJECT DESCRIPTION

This project consists of constructing a guyed tower with the following proposed design parameters:

- Steel Tower (400 ft tall) with 6 guyed wire anchors
  - 332kips (factored axial dead load)
  - 1 kip-ft Torque
- 6 concrete block anchors
  - 14x4x4
  - Embedment depth ~ 9.0 feet BEG
- Mat footing base
  - 11x11x1.5

    - Embedment depth ~ 6.0 BEG
- Base Pedestal
    - 3-feet diameter
    - 5.5 feet tall
    - 4.5 feet embedment

Preliminary grading plans were not provided when this report was being drafted. Minimal cut/fill is anticipated to bring the site to grade.

3.0 SITE DESCRIPTION

Based on Google Earth Data and on-site observations, the site is currently grass-covered with minimal trees in the vicinity of the tower structure. Many mature trees are in proximity to the east anchor. A creek was observed and runs west to east near the proximity of the planned inside-north anchor. The general topography of the site is relatively flat at the points where the anchors and tower are located.

4.0 FIELD OPERATIONS

The subsurface conditions at the site were defined by four (4) sample core borings. The field boring logs were prepared as drilling and sampling progressed. The plan of borings and final logs are included in **APPENDIX A**. Descriptive terms and symbols used on the logs are in accordance with the Unified Soil Classification System (ASTM D 2487). A reference key is provided on the final page of this report.

A truck-mounted drill rig utilizing solid stem auger drilling procedures was used to advance the borings. Soils were sampled by means of a 1 3/8-inch I.D. by 24-inch-long split-spoon sampler driven into the bottom of the borehole in accordance with ASTM D 1586 procedures. In conjunction with this sampling technique, the Standard Penetration Test was conducted by recording the N-value, which is the number of blows required by a 140-pound weight falling 30 inches to drive a split-spoon sampler 1 foot into the ground. For very dense strata, the number of blows is limited to a maximum of 50 blows within a 6-inch increment. Where possible, the sampler is "seated" six inches before the N-value is determined. The N-value obtained from the Standard Penetration Test provides an approximate measure of the relative density that correlates with the shear strength of the soil. The blow count obtained was multiplied by 1.0 to conservatively convert the N values from the manual hammer to the standard N_{60} value for use in correlations to predict engineering properties ($N_{60} \leq 100$). The disturbed samples were removed from the sampler, logged, packaged, and transported to the laboratory for further identification and classification.

Soils were also sampled by means of a 3-inch O.D. by 12-inch-long thick-walled Shelby Tube sampler. Using the drilling rig's hydraulic pressure, the sampler was pushed smoothly into the bottom of the borehole. The consistency of these samples was measured by a calibrated pocket penetrometer. These values, recorded in tons per square foot, are shown on the boring logs. Such samples were extruded in the field, logged, sealed to maintain in situ conditions, and packaged for transport to the laboratory.

All boreholes were backfilled with cuttings after collecting final groundwater readings. Samples obtained during our field studies and not consumed by laboratory testing procedures will be retained in our Tyler office for 30 days. To arrange storage beyond this point in time, please contact the Tyler office.

4.1 Ground Water Observations

Seepage was observed during flight auger drilling at depths ranging from 8 feet to 18 feet. Upon completion of the drilling activities, the open boreholes were measured for groundwater. See table below for groundwater measurements.

TABLE 4.1 - Boring Identification			
Boring	Depth (ft.)	Structure/Location	[1] G.W. Depth (ft.)
B-1	30	Tower	18.0
B-2	30	North Anchor	18.0
B-3	30	South Anchor	18.0
B-4	30	East Anchor	8.0

Table Notes:

1) G.W. = highest groundwater observation during the drilling activities measured from the depth of the existing ground.

Data regarding the groundwater level was obtained by observations in open boreholes. At best this provides only an approximation of the phreatic surface at the time of drilling. *The phreatic surface that should be considered for the design of this project may vary significantly from that which was observed in the borings due to the following factors*:

- The characteristics of the soil profile may have prevented the water level in the bore hole from rising to the phreatic level during the time period of observation.
- A given borehole may not intercept groundwater-bearing zones (i.e., the groundwater is perched or travels in seams or fissures that are not continuous over the entire site)
- Groundwater may only be perched in pockets above local aquicludes, but the distribution of borings is not generally adequate to confirm this with a high level of certainty.
- Groundwater level varies seasonally and with rainfall.
- Rotary wash drilling methods introduce fluid into the borehole which often makes it impossible to distinguish between groundwater and drilling fluid.

If the designer believes that the level of groundwater could significantly impact the project, then ETTL should be contacted to develop a plan for piezometer installation and monitoring to accurately assess groundwater levels at the site.

5.0 LABORATORY TESTING

Upon return to the laboratory, a geotechnical engineer visually examined all samples, and several specimens were selected for representative identification of the substrata. By determining the Atterberg liquid and plastic limits (ASTM D 4318) and the percentage of fines passing the No. 200 sieve (ASTM D 1140), field classification of the various strata was verified. Natural moisture content tests were also conducted (ASTM D 2216).

Laboratory tests were conducted on samples recovered from the borings to evaluate the physical and engineering properties of the different strata and were performed in general accordance with applicable ASTM procedures. The number and type of tests performed for this study are listed in the table below. Details regarding these tests are included on the logs (**APPENDIX A**) and in the Laboratory Test Reports located in **APPENDIX B**.

TABLE 5.0 – Soil Laboratory Testing Procedures		
Laboratory Test	**Test Method**	**Number of Tests**
Dry Sieve Analysis (% Passing No. 4)	ASTM D 6913	12
Dry Sieve Analysis (% Passing No. 40)	ASTM D 6913	12
Washed Sieve Analysis (% Passing No. 200)	ASTM D 1140	12
Atterberg Limits (Liquid & Plastic Limits)	ASTM D 4318	12
Moisture Content by Dry Weight	ASTM D 2216	12
UU Triaxial Compression	ASTM D2850	3
One-Dimensional Swell	ASTM D4546	2

The above laboratory tests were performed in general accordance with applicable ASTM, U.S. Army Corps of Engineers procedures, and/or generally accepted practice. It should be noted that reference to ASTM or other standard procedures does not imply that all cross-referenced procedures in ASTM or other standards have been used, or that all ASTM or other procedures used have been followed exactly. Only those ASTM or other standard procedures and/or portions of procedures, which, in the professional judgment of the geotechnical engineer of record for this report, are applicable, appropriate, and necessary for this particular project, have been used or followed.

6.0 FOUNDATION SOIL STRATIGRAPHY AND PROPERTIES

6.1 Site Geology

According to the Bureau of Economic Geology at the University of Texas at Austin, Geologic Atlas of Texas, Palestine Sheet, the proposed site is located in the Cook Mountain Formation (Ecm).

Generally, the formation consists of shale, marl, and sand, with minor amounts of limestone, glauconite, and gypsum. A more detailed description would show, from bottom to top, 65 feet of blue to gray marl (clay), lignitic shale, limestone lentils, and numerous ferruginous concretions; 100 feet of thinly bedded fossiliferous chocolate colored shale/sandy shale, and ferruginous concretions; 65 to 100 feet of thinly bedded fine to medium sand interbedded with shale and silty shale, which designated as the Spiller Sand Member of the Crockett Formation and later assigned to the Cook Mountain Formation; and 100 feet of shale with some ferruginous concretions that have formed around selenite crystals. Approximately 45 feet above the base of the marl is a persistent and mapped unit consisting of beds of spherical ferruginous concretions, up to 6 inches in diameter, and beds of shale and bentonite. The thickness of the Cook Mountain Formation ranges from about 340 feet near its area of outcrop to 405 feet in the vicinity of Lovelady, and averages about 375 feet.

For more information, please refer to the National Geologic Map Database and the Geologic Atlas of Texas:

http://ngmdb.usgs.gov/ngmdb/ngmdb_home.html
https://www.twdb.texas.gov/groundwater/aquifer/GAT/

6.2 Site Stratigraphy

The soils beneath the proposed structure footprint generally consist of strata as described below (depths are approximate):

TABLE 6.2 Site Stratigraphy

Boring I.D.	Layer	Soils encountered	Depth (ft)
Tower	1	Loose sands	0 - 3
	2	Very stiff to hard expansive clays	5 - 30
North Anchor	1	medium stiff Expansive clays	0 - 3
	2	Hard expansive clays	3 – 13
	3	Stiff to very stiff expansive clays	13 - 30
South Anchor	1	medium stiff Expansive clays	0 – 3
	2	Stiff to very stiff expansive clays	3 – 30
East Anchor	1	loose sands	0 – 3
	2	Medium dense expansive clayey sands and expansive clays	3 – 13
	3	Stiff to very stiff expansive clays	13 – 30

Notes:

1) Sand content may vary significantly from what was observed

2) Layers described as "sand" are considered to pose minimal to negligible risk with respect to vertical heave

The classifications are based on weathering, depositional environment, mineralogy, color change, lithology, and structure. Detailed on the boring logs in **APPENDIX A** are the specific soil types, the condition of the soil, and the depths of the various soil strata encountered. The logs show defined boundaries between various soil types, but in reality, the transition between types is generally gradual.

6.3 Soil Properties

Due to the non-homogeneous nature of the soil and the necessarily limited data, the issue of assigning quantitative design parameters for the various characteristics of a soil mass is a matter of interpretation. In assessing shear strength along a failure surface that passes through a large mass, it is reasonable to expect that strength variations will be encountered along any potential surface. Where data are sufficient, we believe that it is overly conservative to take the lowest test data values as representative of the characteristics of a soil mass. On the other hand, using average values could be unconservative. How we recommend selecting appropriate values to use is explained below.

TABLE 6.3.1 - Soil Properties

Description of Layer	Depth	Drained (Sand)/ Undrained (Clay) Conditions			Soil Type (For L-Pile Analysis) [4]	Soil Class
		Cohesion c (psf) [2/5]	Moist Unit Wt. (pcf)[1]	Angle of Internal Friction [3]		
Compacted Select Fill	Where occurs	0	120	30	Sand (Reese)	SM, SC
Loose Sand (N<10)	Where occurs	0	110	28	Stiff Clay w/o Free Water	SM,
Medium Dense Sand (10<N<30)	Where occurs	1200	115	30	Stiff Clay w/o Free Water	SC
Dense Sand (N>30)	Where occurs	1500	120	32	Stiff Clay w/o Free Water	SC
Medium Stiff to Stiff Sandy Clays	Where occurs	1500	113	-	Stiff Clay w/o Free Water	CL-CH
Stiff to Very Stiff Sandy Clays	Where occurs	1800	120	-	Stiff Clay w/o Free Water	CL-CH

Notes:

1) Buoyant unit weight when applicable, see boring logs for groundwater depth.

2) Peak Unconsolidated/Undrained shear strength (psf) at in-situ moisture content, measured by U.U. triaxial test or estimated from SPT field data.
3) Estimated drained friction angle (Phi' = degrees), measured by CU triaxial and CD direct shear, or correlated values.
4) Use default L-Pile values for K and e50, as applicable, where values are not otherwise indicated.
5) Undrained Cohesion only applies to layers of cohesive soils (SC, CL, CH) with a PI >8

6.4 The Behavior of Expansive Soils

Expansive soils can include various types such as Clayey Sand (SC), Lean Clay (CL), or Fat Clay (CH), which have the capacity to change volume—shrinking or swelling—with fluctuations in moisture content. These soils, found at different depths throughout the soil profile, expand when they absorb moisture and contract as they dry. Structures built on these soils tend to move up and down as a result of these volume changes. When expansive soils are covered by an impermeable surface like a building or pavement, seasonal moisture variations inside the covered area are often reduced or eliminated because they are less exposed to natural wetting and drying factors such as wind, rain, sun, and vegetation. However, at the perimeter of the structure, infiltration of surface water into the foundation soils—or drainage—can cause local swelling of dry clays, which may lead to tilting or distortion of the foundation. When areas adjacent to the structure are paved, the risks of swelling from excess moisture absorption and shrinkage due to moisture loss are both minimized significantly.

6.5 Vertical Heave Predictions

The assessment of the impact of expansive soils described below is based solely on soil moisture changes resulting from normal climate fluctuations. Factors such as poor drainage and resulting ponding, plumbing leaks, excavation details (e.g., permeable backfill in trenches or beneath structures), and vegetation (trees and shrubs) can cause moisture variations (and subsequent swelling or shrinking) outside the ranges predicted here. The predicted heave also represents the potential differential movement that a lightly loaded slab or foundation on native soils could experience.

The actual movement of a part of the structure depends on a complex interaction of the various factors mentioned above. The accuracy of predicted movement relies on how well the prediction considers these factors. The TxDOT PVR method is widely accepted for predicting shrink/swell movement potential but is based on empirical data and established correlations with Atterberg Limits, and it can be applied to some soil types. Moisture conditions can vary, making it difficult to accurately predict maximum free swell. The USACE method, however, is based on measuring the actual swell of specimens, which makes it more reliable than the PVR method in accounting for soil heaving characteristics at the given moisture condition.

<u>Footings and Anchor Blocks</u>

The assumed embedment depth of the footings and anchor blocks, see **section 2.0,** will result in PVR <1.0." The potential for heaving due to moisture changes is considered low for the tower base and anchors if the subgrade is prepared in accordance with Section 9.0, based on the conditions at the time of drilling.

Slabs on Grade(SOG)

Based on the soils encountered, slabs on grade are at risk of vertical movements outside what is generally considered tolerable for most structures (>1.0"). See **TABLE 6.3**.

One commonly used method for quantifying the potential for subgrade movement at any given location is to calculate the Potential Vertical Rise (PVR) (TxDOT method -Tex 124 E). This calculation considers the inter-relationship between depth, Atterberg Limits (LL & PI), and fluctuations in soil moisture. Another method for predicting the potential of vertical heave (PVH) of the subgrade soils is the USACE Hand (manual) Method as described in FOUNDATIONS IN EXPANSIVE SOILS (TM 5-818-7, 1983) section 5-4. This method predicts vertical heave based on test results obtained from a consolidometer restraint-free swell test at in-situ moisture and density. The predictions below are based on an estimated seasonal moisture fluctuation zone of approximately 10 feet.

TABLE 6.3 – Vertical Heave Predictions

Moisture State[1]	TxDOT PVR (in)	USACE PVH (in)
Dry	3.1	-
Moist	1.1	-
Existing Condition	2.1	3.8

Notes:

1) As defined by the PVR method, PVH Dry = MC<(0.2*LL+9), Moist= MC>(0.47*LL+2)
2) Based on laboratory testing of specimens trimmed in-situ moisture. Significant drying of the upper strata prior to construction will increase the total predicted

6.6 Methods for Reducing Heave for On-Grade Structures

The acceptable movement level varies based on the design and other requirements, but methods are usually chosen to limit slab or foundation movements to about 1.0 inch or less. Anticipated movements can be reduced by using techniques developed specifically to minimize on-grade slab movements. The most common method involves removing and replacing native soils with compacted select fill or conditioning the soils with moisture. These methods will decrease, but not completely prevent, the risk of unacceptable movements. Such subgrade preparation options are presented for the owner's consideration and may or may not be practical or cost-effective, depending on the expected performance of the structure. Owners should be aware that these methods do not stop soil-supported foundation elements from moving; they only aim to reduce the size of the expected movement. Placing the floor slab on grade is a balance between construction costs and the risk of floor or related element damage. A detailed explanation of these risks can be provided upon request. **Section 9.0** offers more details on how to implement the methods described above.

6.6.1 Replacement of the Expansive Clay with Select Fill

Based on the conditions encountered at the site and assumed dry conditions, excavating 8.0 feet below the existing grade and replacing it with select fill will reduce the shrink/swell potential within 1.0 inch.

6.7 Seismic Site Classification

IBC 2015 requires density/shear modulus information extending to a depth of 100 feet for seismic site classification. The current scope does not include the required 100-foot soil profile with borings that are drilled to a maximum of 25 feet below the existing grade. Consequently, we have assumed that the density (blow count) of the soil/rock encountered at the terminal depth is representative of the profile to a depth of 100 feet. If the seismic site class definition is critical to the design, this assumption should be confirmed by further testing. Based on the site class noted below we do not believe further testing would benefit an improved site classification.

Based on the 2015 IBC, the seismic site class definition is **Class D "Stiff Soil",** and the Risk **Category is III** (provided). California's Office of Statewide Health Planning and Development (OSHPD) provides an online tool that calculates the seismic design values based on the overall project and site information listed above. A printout of this report is provided in **APPENDIX C**. ETTL does not warrant the accuracy of this report, and it is presented to the client for information purposes only.

For more insight regarding the information we have provided, please visit:
https://www.usgs.gov/natural-hazards/earthquake-hazards/hazards

7.0 FOUNDATION DESIGN RECOMMENDATIONS

Two independent design criteria must be met in selecting the type of foundation to support the proposed structure. First, the ultimate bearing capacity, reduced by an appropriate factor of safety (usually taken as 3 for DL plus sustained LL, and which varies depending on the loading case) (or resistance factor if LRFD analysis), should not be exceeded by the bearing pressure (factored for LRFD analysis) transferred to the foundation soils. Second, predicted total and differential vertical movements due to consolidation and/or expansion of the underlying soils during the operating life of the structure(s) should be within tolerable limits. For most structures similar to that of the current project, 1 inch of predicted total settlement or heave is widely considered an acceptable target for design. *It should be noted, however, that if the differential settlement or heaving of this magnitude were actually to occur, distress distortion, and/or tilting of the structure can be expected at least in some circumstances.*

7.1 General Considerations

Some conditions that may affect foundation and slab performance (e.g., plumbing leaks, poor drainage conditions, deep-seated heave, etc.) are difficult to account for in standard slab and foundation design procedures and are not considered elsewhere in this report. Such sources of moisture change could cause vertical movements and lead to significant distress. To virtually eliminate the risk of damage from vertical movement under these conditions, an option incorporating deep foundations with a suspended slab is recommended.

With ground-supported foundation/floor systems, measures must be taken to help assure subgrade moisture stability (see **Section 9.4**) to enhance the chances of satisfactory structure performance. Proper site design that prevents excessive drying or wetting of the subgrade soils around the structure is crucial to minimize the potential for excessive movement caused by the shrinkage and/or swelling of the foundation soils.

Provisions should be made to account for the possibility of some differential movement (see movement prediction in **Section 6.3** above) between the main structure and driveways, sidewalks, and other associated structures. Proper site design that prevents water from soaking into the subgrade soils around the structures and appurtenances (i.e., provides for rapid runoff away from them) is critical to reduce the potential for excessive movement caused by saturation of foundation soils and should help limit differential movement between exterior soil-supported elements and the structures.

7.2 Shallow Footings

Qualified personnel should inspect all footing excavations to ensure that the subgrade is composed of firm, undisturbed native soil or properly compacted select fill as recommended in this report. Water and/or loose material in footing excavations should be removed before the final shaping of the footing excavation and placement of concrete.

7.2.1 Bearing Capacity

In recommending bearing capacity and predicting settlement, we assume that footings will be bearing on subgrade with the engineering characteristics of **Layer 2**. The following recommendations for static loading are provided to cover footings that bear at a minimum depth of 6.0 feet below the surrounding finished grade. Recommended allowable bearing pressure varies with footing dimensions, shape, and founding depth and our recommendations are based on the most critical combination of these factors. In some cases, this basis may prove conservative. It was outside of our scope to provide specific recommendations for all possible combinations of factors and configurations, but ETTL can evaluate specific configurations and combinations of factors in order to provide a less conservative recommendation, where deemed beneficial.

The recommendations below also presume that there are not now, nor will there ever be a slope that will extend to an elevation lower than the base of the footing, the top of which slope falls within a distance of one footing width from the edge of the footing. If this is not the case, please contact this office for further recommendations regarding the impact of the sloping ground on predicted footing capacity.

We incorporate in our recommendations a predicted overall factor of safety of 3 against the concentric shear failure of the soil for the case of DL + LL. In the case of transient loadings (seismic or wind), a decrease in the safety factor is commonly recommended (typically 2.25, i.e., an increase of the allowable we have recommended for a FS=3 by 33%).

The recommended allowable *gross concentric* bearing pressure to be used for proportioning the footings with respect to soil shear failure (settlement considerations may dictate that a lesser allowable pressure be used) is provided in the table below.

TABLE 7.2.1 - Gross Concentric Allowable Bearing Capacity (psf)		
Foundation I.D.	**Embedment Depth**	**Bearing Capacity (psf)**
Center Pedestal	6'	3,740
Anchor Blocks	9'	4,000
On-Grade Structures	2'	2,000

An increase in footing depth (for depths below the assumed above) will generally result in an increase in recommended allowable *gross* bearing pressure; however, settlement reduction for increased footing depth is limited.

Recommended allowable bearing pressure is also dependent on the degree of eccentricity of the load from the centroid of the footing. Where footings are subject to eccentric loading, the allowable bearing, as recommended above, will need to be reduced appropriately; see **Section 7.2.3**.

7.2.2 Predicted Settlement

The relatively immediate settlement of the 11' x 11' footing bearing 6.0' BEG, loaded with ≈296 kips, is predicted to be 1.0 inch (total). Based on the soil types beneath the tower, the relative density of the soil, observed groundwater levels, and the anticipated loads on the various foundation elements, the magnitude of long-term settlement is not expected to be significantly more than the immediate settlement. The actual total settlement is calculated based on the unfactored working footing pressure (i.e., unfactored dead load plus unfactored sustained live load). It is often the case that the unfactored working pressure is significantly less than the factored demand loading, and the maximum footing width recommended can be exceeded while still maintaining settlements of 1 inch.

7.2.3 Eccentrically Loaded Footings

Allowable loading for eccentric footings is proportional to the degree of eccentricity and is lower than for a concentrically loaded footing. Equivalent allowable vertical uniform pressure (i.e., ignoring the effects of overturning moments) on an eccentrically loaded footing may be computed in accordance with the following:

$$q_{ae} = q_a * R_{ex} * R_{ey}$$

Where:

q_a = allowable uniform pressure for a concentrically loaded footing as given above.

R_{ex} = reduction coefficient for eccentricity about the x-axis
= $1 - 2 * e_x/B_x$ for cohesive soils (CL & CH)
= $1 - (e_x/B_x)^{0.5}$ for cohesionless soils (SM, SC, ML)

R_{ey} = reduction coefficient for eccentricity about the y-axis
= $1 - 2 * e_y/B_y$ for cohesive soils (CL & CH)
= $1 - (e_y/B_y)^{0.5\ for}$ cohesionless soils (SM, SC, ML)

e_x, e_y = eccentricity in the x and y direction, respectively

B_x, B_y = footing dimension in the x and y direction, respectively

Use the predominant soil type in the zone which is one footing width beneath the footing to calculate the reduction coefficient. If no one type is predominant, then use a weighted average based on the relative thicknesses in this zone. Total allowable vertical load with eccentricities e_x and e_y may be found by multiplying the gross area of the eccentrically loaded footing by q_{ae} as determined above.

7.2.4 Sliding Resistance for Footings on Grade

The ultimate (nominal) sliding resistance (**R_T**) should be checked for both drained and undrained loading conditions using the parameters listed in the **Soil Properties TABLE 6.3.1,** for the appropriate bearing stratum and the following formulae for nominal (un-factored) resistance:

For drained loading:

R_T $= V * \tan \delta$

- V = total un-factored vertical force on the footing base for the given loading condition
  $\tan \delta = \tan \varphi$ (φ = drained friction angle of the soil for the case where concrete is cast against the soil. Use $\tan \delta = 0.8 * \tan \varphi$ for precast concrete)

For undrained loading:

R_T = c; or the lesser of: **R_T** = c vs. **R_T** $= 0.5 * V$

- c = undrained shear strength (cohesion), also designated S_u
- Nominal resistance should be factored in as appropriate.
- 6" (min) of compacted, well-graded granular material is placed beneath the footing (where suitable) use

7.2.5 Passive Resistance of Below-Grade Footings

Ultimate (nominal) passive resistance **P_p** of the soil loaded by a footing block face should be computed by the following formula for both drained and undrained conditions and selecting the most critical condition:

$\mathbf{P_p}$ **(drained) = γ*(K_p)*z + 2*c' *(K_p)** $^{0.5}$ (second term often neglected)
$\mathbf{P_p}$ **(undrained) = γ*(K_p)*z + 2*c *(K_p)** $^{0.5}$ **(usually reduces to: γ*z + 2*c)**

- γ = effective unit weight
- K_p = Passive pressure coefficient = $(Tan(45+\varphi'/2))^2$ (equals 1 for undrained φ=0 condition)
- φ' = Effective angle of internal friction.
- Φ = Undrained angle of internal friction, generally = 0
- c,' c = drained/undrained cohesion
- z = depth where pressure is determined

The appropriate parameters to be used in the above equation are to be selected from **TABLE 6.3.1,** for the appropriate loading condition that controls (i.e., long-term(drained) or short-term (undrained)) for the soil against the face of the footing. A significant amount of lateral movement is required to fully mobilize ultimate passive pressure (as much as 6% of the depth to the base of the loaded face). To limit the lateral movement to about 1% (of the depth to the base of the loaded face) a factor of about 0.5 is typically recommended to reduce the above-computed resistance to the available mobilized nominal (un-factored) passive resistance given lateral displacement restrictions.

Passive resistance as computed above assumes that the footing excavation can be constructed in such a manner as to provide solid contact of the side of the concrete with the undisturbed sides of the excavation (which may be impractical in some situations). *Caution*: Lateral resistance against a vertical face should only be assumed where construction can be controlled to assure that the footing is cast against undisturbed earth or backfill between the excavation face and the footing edge is placed under density-controlled conditions (backfill should be placed to 100% ASTM D698). It should be noted that such heavy compaction against a wall face will result in earth pressures exceeding the usually assumed active or at-rest pressures. *Also, the temporary excavation face needs to be nearly vertical and extended to the bottom of the footing elevation. The portion of the sides of the excavation for the footing that is comprised of fat clay exposed to wetting or drying action and that is within 5' of the finished ground surface should be neglected with respect to computing passive resistance to account for possible softening or shrinkage of the zone.*

7.3 Anchor Resistance

Provided in **TABLE 6.3.1** are soil properties to be utilized in the design of the guy anchors to resist pullout by the inclined tensile load applied by the guy cables. Sliding and passive resistance can be calculated per **Section 7.2.5** above. Typically, anchors consist of buried concrete blocks. Certain conditions may warrant deep foundations, such as drilled shafts or helical piles, to be used as anchors in conjunction with the anchor block. For shallow block anchors without deep foundations, the allowable pullout resistance assumes that the anchor excavation can be constructed in a manner that provides solid contact between the side of the concrete and the *undisturbed sides of the excavation.* Shallow groundwater and loose sandy conditions were observed at boring location B-3; excavations below 2 feet will most likely result in caving conditions. Shoring and dewatering should limit this, but some soil sloughing off the sidewalls is likely and will result in a reduction of shear strength derived from the soil properties reported. For additional capacity, battered helical piles can be incorporated into the shallow anchor blocks above the groundwater. If anchors are to be constructed below the groundwater, we recommend using the excavation and compaction procedure outlined in **Section 9.3.**

8.0 FLOOR SYSTEMS AND FLATWORK

Floor systems or shallow mats placed on native subgrade will potentially be subject to differential movements as noted in **Section 6.3,** which may result in unacceptable distress to the floor and elements supported thereon. Managing and/or mitigating this risk can be addressed in several ways, including (in order of decreasing risk):

1. Removal and replacement of a portion of expansive clay from beneath the floor to reduce the degree of potential movement, generally to within a range of 1 inch.
    a. In order to reduce the predicted potential movements to 1 inch we recommend that the structure's subgrade be prepared as set forth in **Section 9.0.**
2. Isolation of the floor/structure from subgrade movements by structurally suspending it on deep foundations above a void or crawl space.

If the risk from predicted potential vertical differential movements as noted in **Section 6.3** and as modified per the provisions of **Section 9.2** of this report is not considered tolerable, a structurally suspended floor slab (option 2) can be used to significantly reduce the risk of distress. We presume that an evaluation by others of risk vs. cost will determine the appropriate design approach.

8.1 Slab on Grade

Floor slabs can be placed on a subgrade prepared as described in **Section 9.2**. Slab-on-grade construction should only be considered if the risk of slab movement (and potential resulting damage) as noted throughout this report can be tolerated. The acceptable level of movement varies with the design and other requirements, but methods are typically chosen with the goal of limiting predicted slab movements to about 1.0 inch or less. Reductions in expected movements can be achieved by using subgrade modification techniques developed for this purpose. The most common method in this area involves undercutting and replacing the expansive soils. While this method will reduce the risk, it will not eliminate the possibility of unacceptable movements.

9.0 EXCAVATION AND SITE WORK

To validate the design assumptions given above regarding allowable foundation loads, and, in order to provide a serviceable floor system (within the limitations stated above), the subgrade of the tower and anchors must be properly prepared. The following procedures are recommended as a minimum:

9.1 Site Preparation

Strip to remove all topsoil, and other deleterious materials from the subgrade to a minimum of 6 inches deep. Where trees are removed (or have been removed in the last year) from the structure footprint area, the entire root zone should be cut out and replaced with select fill. Root zones tend to be comprised of highly desiccated soil, which, if left in place, is prone to significant swelling, resulting in heaving of the slab. Verify that all stump holes are backfilled with properly compacted select fill.

9.2 On Grade Stuctures

PVR<1.0 inch

Over-excavate to a depth of existing grade minus 8 feet and replace with compacted select fill. Prior to placing fill, scarify the exposed subgrade to a depth of 12 inches, adjust the moisture content, and maintain it within a range of optimum to optimum +4% and recompact to a minimum density of 98% of the maximum density defined by ASTM D 698 (Standard Proctor). Maintain specified moisture content until the subgrade is covered with fill (**see section 9.6**) or slab.

PVR=3.8 inch

No over-excavation is required. After the subgrade is prepared per **section 9.1** above, scarify the exposed subgrade to a depth of 12 inches, adjust the moisture content, and maintain it within a range of optimum to optimum +4% and recompact to a minimum density of 95% of the maximum density defined by ASTM D 698 (Standard Proctor). Maintain specified moisture content until the subgrade is covered with fill (**see section 9.5**) or slab.

9.3 Anchor Blocks

Where the required excavations are below or approaching the groundwater depth, prudent provisions for managing or removing water and possibly shoring of excavations are advisable. Water removal may be as simple as pumping it from the excavation or may involve more complicated temporary dewatering measures such as well points. Construction during drier seasons of the year may alleviate potential problems to some extent. The exact nature of how best to handle the groundwater problem cannot, however, be determined until the source and quantity of flow have been explored more fully.

Qualified personnel should inspect excavations to ensure that the subgrade is composed of firm native material. Excavations should be completed with a smooth-edged bucket to avoid grooving the bearing surface. Water and/or loose material in footing excavations should be removed prior to the final shaping of the footing excavation and placement of concrete. If firm material is not obtainable in the base of the footing, a deeper founding depth may be required. However, excavations below groundwater depth are not recommended unless appropriate dewatering measures are implemented.

As indicated previously, recommendations provided herein, unless specifically stated otherwise, are contingent upon the proper placement of specified backfill against anchor blocks where the anchor block concrete cannot be placed directly against firm, undisturbed native soil (i.e., where anchors need to be formed in open excavations due to groundwater problems). It is imperative that backfill around the anchor block is properly compacted and should be placed against the compression face of each anchor block within the following limits: an area defined by the compression face of the block, a horizontal plane through the base of the block, a vertical plane between the tower and the anchor block at a distance from the face of the anchor block equal to the depth to the base and vertical planes parallel to the direction of the guy and spaced off each side of the anchor block a distance equal to one half the depth to the base.

9.4 Site Design

The following recommendations are derived from years of experience with structures founded on expansive soils and are considered essential to satisfactory structure performance, especially where the floor slab is to be placed on grade:

- Sidewalks should be sloped away from buildings and should not be tied to the structures (with the exception of entrance walkways). The joint between the sidewalk or pavement and the foundation should be sealed and maintained. Sidewalks should not impound water adjacent to the structure. The potential heave of native ground (see **Section 6.3**) adjacent to the structure needs to be taken into consideration when constructing the walk so as to avoid a sidewalk that impounds water adjacent to the structure.

- Any unpaved ground surface around the building or foundation as well as paved areas should be sloped away from the building on all sides so that water will rapidly drain away from the structure. A minimum slope of 5% is recommended for the unpaved areas 10 feet wide immediately adjacent to the structure. Drainage swales should have a minimum longitudinal slope of 2%. Roof drainage should be conveyed by an appropriate means for a distance of at least 15 feet from the building before it is allowed to drain into the subgrade. Water should not be allowed to pond near the building after the floor system has been placed.

- Trees should not be closer than their mature height to the structure and landscape beds should not be placed adjacent to the building unless they can be contained in watertight planter boxes and irrigation water can be prevented from seeping into the subgrade around the building. A horizontal moisture barrier (e.g., heavy gage polyethylene (e.g., StegoWrap 20 mil) permanently sealed to the foundation edge at the ground line and sloped away from the building) placed beneath planting beds and extending to a distance of at least 10' from the building perimeter (and as wide as necessary to cover the over excavated area) is an alternative to planter boxes provided it is maintained in a watertight condition (i.e., joints sealed and punctures repaired). Planting bed edging should not impound water. A root barrier around the entire structure perimeter will provide some added assurance against desiccation of the soil due to roots growing beneath the structure. Periodic root pruning may be required to limit the drying of soils beneath foundations due to vegetation. *Over irrigation adjacent to the structure can cause an increase in subsurface moisture contents that could lead to heaving.*

- To help limit surface water infiltration beneath the structure, backfill in the area above the over excavated area (minimum 5 feet wide adjacent to the structure) should be native lean or fat clay soil compacted to a minimum density of 95% of ASTM D 698 (Standard Proctor) at a moisture content of optimum + 2% or greater. This zone should be at least 2 feet thick. This backfill is not necessary where pavement abuts the structure and the joint is sealed. A vertical moisture barrier sealed to the side of the foundation and extending to a depth of about 8' will provide added protection against moisture fluctuation beneath the building footprint.

- Backfill for utility line ditches should be carefully controlled and should consist of a relatively impermeable material (clayey sand or lean clay), especially in the area beneath and immediately outside of the structure. Old utility lines should be removed from beneath the structure. Fill in new or old utility trenches should be placed to the same specifications as select fill. The top 6 inches under paving should be compacted to a density equal to that specified for the pavement subgrade.

- Utility connections to the building should be flexible to allow for anticipated soil movements (see predictions elsewhere in this report) that will be different than the anticipated movement of the structure to which they are connected (e.g., where attached to, or passing through a foundation element supported on piers). Drain lines should be placed so that potential movements beneath or adjacent to the building do not affect the functionality of the line.

- Any significant differential movement potential between structures supported on deep foundations and the native subgrade needs to be addressed in the design of utilities (See **Sections 6.3 and 9.1**) for predicted soil movements to assess whether or not the potential differential movement is deemed excessive and in need of mitigation). Common approaches to this issue entail utility conduits supported on deep foundations and/or utility corridors placed on the prepared subgrade. In either case, there will need to be a transition zone where the depth of soil preparation tapers from full depth to none at a 1:1 slope or flatter (verify that the resultant slope of the utilities can tolerate this differential slope and still be functional). The prepared zones beneath utility corridors should be twice the width of the corridor at its base tapering up to the ground surface at a 1:1 slope in all directions (i.e., perpendicular to the longitudinal axis of the corridor).

9.5 Imported Select Fill

Structural fills consist of, select fill, crushed stone, or flex base. Select fill shall consist of homogeneous soils (i.e., not sand with clay lumps) and must adhere to all the following soil properties:

- Classify as Silty Sand (SM), Clayey Sand (SC), Sandy Lean Clay (CL), Clayey Gravel (GC) free of organic matter and rocks larger than 3 inches in diameter
- Atterberg plasticity index (PI) between 5 and 20
- Liquid Limit of 40 or less, (ASTM D4318)
- Percent passing the No. 200 sieve 65% or less, (ASTM D1140)
- On-site material qualifies as select fill if placed as recommended below

In-lieu of select fill, crushed stone, or concrete base material meeting the requirements of TxDOT 2014 Standard Specifications Item 247, Type A (D for crushed concrete), Grade 3 or better can be used.

Atterberg limits testing of the fill at a rate of 1 test per 500 cubic yards of fill placed (minimum 1 test per fill area per lift and as visual changes occur) is recommended to verify that fill specifications are met. The material should be placed in the following manner:

- Prepare the subgrade in accordance with the recommendations discussed elsewhere herein. Sites that slope more than about 15% should be benched with 8-foot-wide benches prior to placing fill.

- Place subsequent lifts of select fill in thin, loose layers not exceeding 9 inches in thickness to the desired rough grade and compact to a minimum of 95% of the maximum density defined by ASTM D 698 (Standard Proctor). Maintain moisture within a range of -1/+3 of the optimum moisture content.

- Conduct in-place field density tests at a rate of one test per 3,000 square feet or a minimum of 2 tests per lift. *Density testing is essential to ensure that the soil beneath the structure is properly placed.*

- Prevent the excessive loss of moisture during construction (periodic sprinkling may be required).

9.6 General Fill

General fill may consist of any of the following soil classifications: SC, CL, CH, and may be used as fill to bring the site to grade in areas where other specific preparation is not desired and where differential soil movement is not critical for both the current and future development. Proper processing, placement, and testing of this material are essential. Listed below are several factors that must be considered if fat clay (CH) is to be used for general fill:

- The material must be relatively uniform (homogeneous). It cannot be blocky or shaley or a mixture of various types of soils or even various types of clays.

- The clay must be processed to ensure uniform moisture distribution (a good rule of thumb for gradation is 6" or less). Discing and ripping the clay may not be adequate for this process. Sometimes the use of a pulvermixer is necessary. Processing and watering the clay in place prior to picking it up and moving it is typically more effective since it allows more time for the water to penetrate the soil "clods." Some shaley clay cannot be effectively processed.

Place processed soil in loose lifts not exceeding 9" in thickness and compact to a density of 95% of ASTM D698 (Standard Proctor) and maintain moisture of optimum to optimum +4%. The fill must not be allowed to dry prior to the placement of succeeding lifts.

9.7 Control of Groundwater

As discussed in **Section 4.1**, excavations 8 feet below grade will likely encounter local perched water or permanent groundwater, and if so, will experience infiltration that needs to be addressed during construction.

Measures that will need to be taken will probably be as simple as excavating a trench to collect the seepage and pumping water from sumps adjacent to the excavation. In isolated areas, the saturated soils can be over-excavated and replaced with compacted select fill. Construction traffic should be excluded from the excavation virgin base area when it is wet, as this traffic may seriously damage the bearing soils. What measures are appropriate to deal with excess water, both during construction and permanently, depend on the nature and extent of the groundwater and the permeability of the strata through which it flows. Such a characterization was outside the scope of this study.

Where the groundwater is "perched" or is a relatively minor flow through pervious soils on top of a relatively shallow impervious stratum (called an "aquiclude"), an upstream trench drain around the perimeter of the worksite or a system of ditches routing flow to sumps may be effective in intercepting some or all of the flow, provided interception trench drains penetrate the aquifer sufficiently and are not bypassed by local areas of artesian flow. Minor flows that occur in secondary features of the soil profile such as joints or cracks can also usually be managed by the collection of the flow in a sump. Intercepted flow may either be pumped from the sump or trench or allowed to drain by gravity.

When the flow is not perched, a system of well-points often proves to be the most effective way of dewatering excavations. A system of temporary wells would be designed to provide sufficient drawdown of the groundwater below anticipated construction elevations. However, the design of such a system necessitates detailed knowledge of the quantity of groundwater flow and the permeability and extent of the aquifers. Obtaining this kind of data was beyond the scope of this investigation. Groundwater that is not permanently lowered (i.e., maintained at a head level below foundations), will exert buoyancy forces on foundations, possibly resulting in heaving.

9.8 Excavation Safety

The Federal Register, Volume 54, No. 209 (Revised July 1992), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) contains the "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". The contractor is solely responsible for designing and constructing stable, temporary excavations in accordance with these standards and should shore, slope, or bench the sides of the excavations as required to maintain the stability of both the excavation sides and bottom. ETTL has not performed stability analyses of any kind. The contractor's "responsible person," as defined in CFR Part 1926, should evaluate the soil exposed in the excavation as part of the contractor's safety procedure. In no case should the height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. Contractors should review the boring logs in **APPENDIX A** to determine the appropriate soil type per the aforementioned OSHA regulations.

10.0 LIMITATIONS

Geotechnical design work is characterized by the presence of a calculated risk that soil and groundwater conditions may not have been fully revealed by the exploratory borings. This risk derives from the practical necessity of basing interpretations and design conclusions on a limited sampling of the subsoil stratigraphy at the project site. The number of borings and spacing is chosen in such a manner as to decrease the possibility of undiscovered anomalies while considering the nature of loading, size, and cost of the project. The recommendations given in this report are based upon the conditions that existed at the boring locations at the time they were drilled. The term "existing groundline" or "existing subgrade" refers to the ground elevations and soil conditions at the time of our field operations.

It is conceivable that soil conditions throughout the site may vary from those observed in the exploratory borings. If such discontinuities do exist, they may not become evident until construction begins or possibly much later. Consequently, careful observations by the geotechnical engineer must be made of the construction as it progresses to help detect significant and obvious deviations of actual conditions throughout the project area from those inferred from the exploratory borings. Should any conditions at variance with those noted in this report be encountered during construction, this office should be notified immediately so that further investigations and supplemental recommendations can be made.

Construction plans and specifications should be submitted to ETTL for review prior to issuance for construction to help verify that the recommendations of this report have been correctly understood and implemented.

This company is not responsible for the conclusions, opinions, or recommendations made by others based on the contents of this report. The recommendations made in this report apply only to the proposed scope of work as defined in **SECTION 2.0 PROJECT DESCRIPTION** and may not be used for any other work without the express written consent of ETTL Engineers. The purpose of this study is only as stated elsewhere herein and is not intended to comply with the requirements of 30 TAC 330 Subchapter T regarding testing to determine the presence of a landfill. Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. No warranties are either expressed or implied.

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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APPENDIX A

Plan of Borings and Boring Logs

ETTL Engineers & Consultants	**PREPARED BY** ETTL Engineers and Consultants Whitehouse, TX	**PROJECT** Name: Tri-City Ranch Communication Tower Number: G 6564-25	**LOCATION** 31.163028, -95.877707 Leona, TX	**SYMBOL KEY** Soil Boring

ETTL | Engineers & Consultants

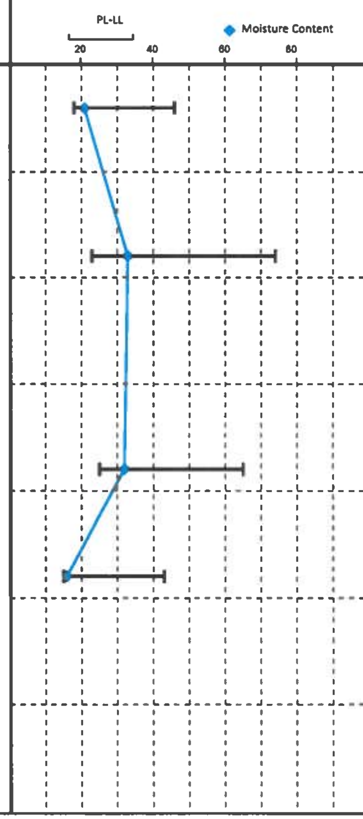
Soil Boring: B-1

Project: Tri-City Ranch Communication Tower	Project No.: G 6564-25
Location: 8117 Co Rd 106, Leona, TX	Ground Elev.: N/A
Date Start - Finsh: 07/07/2025 - 07/07/2025	Coordinates: 31.163064, -95.877792

MAIN OFFICE
3527 Star Road
Whitehouse, Texas 75791
903-595-4421

MATERIAL DESCRIPTION	Depth (ft)
Clayey Sand (SC) loose; reddish brown and gray; slightly moist; mottled	3.0
Sandy Lean Clay (CL) dense; gray; moist; abundant iron-oxide staining	5.0
Fat Clay with Sand (CH) hard; reddish brown & gray; slightly moist; iron-oxide staining **Fat Clay** (CH); gray with brown; moist; laminated; iron-oxide staining; mottled Very stiff	18.0
Sandy Fat Clay (CH) very stiff; brown; moist to very moist; abundant iron-oxide staining	23.0
Lean Clay with Sand (CL) hard; gray; slightly moist; abundant iron-oxide staining; mottled	25.0
Fat Clay with Sand (CH) very stiff; dark gray; moist; laminated	30.0

Field Strength Data N (Blows/ft) PP (tsf)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	% Plus #40	% Plus #4	Wet Density (PCF)	Compressive Strength (tsf)	Shrink/Swell Load Stress (KSF)	% Swell
N: 9	21.0	46	18	28	37.0	2.0	0.0				
PP: 4.5											
PP: 4.5+											
	33.0	74	23	51	98.0	0.0	0.0	113.5	1.51		
N: 16											
N: 24	32.0	65	25	40	52.0	19.0	5.0				
PP: 4.5	16.0	43	15	28	73.0	1.0	0.0				
N: 28											



Field Notes

Groundwater Readings

Graphic	Date	Time	Depth	Notes
▽			18'	Water encountered @ 18'
▼			23'	Water encountered @ 23'

Soil Boring: B-2

ETTL | Engineers & Consultants

Project: Tri-City Ranch Communication Tower	Project No.: G 6564-25
Location: 8117 Co Rd 106, Leona, TX	Ground Elev.: N/A
Date Start - Finsh: 07/07/2025 - 07/07/2025	Coordinates: 31.163786, -95.878042

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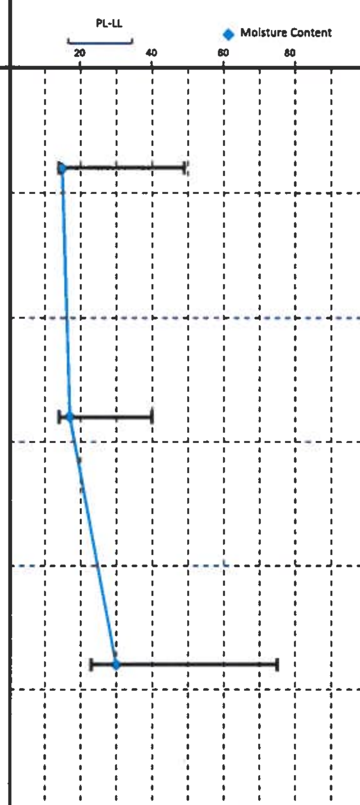
MATERIAL DESCRIPTION	Field Strength Data N (Blows/ft) PP (tsf)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	% Plus #40	% Plus #4	Wet Density (PCF)	Compressive Strength (tsf)	Shrink/Swell Load Stress (KSF)	% Swell
Sandy Lean Clay (CL) medium stiff; reddish brown & gray; dry to moist; mottled	N: 9											
Very hard	PP: 4.5+	15.0	49	14	35	70.0	0.0	0.0	131.4		5.89	3.9
	PP: 4.5+											
(13.0)	PP: 4.5+											
Lean Clay with Sand (CL) very stiff; gray; dry to moist; mottled	N: 26	17.0	40	14	26	77.0	0.0	0.0				
Stiff; moist; abundant iron-oxide staining (23.0)	N: 17											
Fat Clay (CH) very stiff; moist; laminated; abundant iron-oxide staining		30.0	75	23	52	98.0	1.0	0.0	114.9	1.92		
Very stiff; gray with brown	N: 16											



Field Notes

Groundwater Readings

Graphic	Date	Time	Depth	Notes
▽			18'	Water encountered @ 18'
▼			22'	Water encountered @ 22'

ETTL | Engineers & Consultants

Soil Boring: B-3

Project: Tri-City Ranch Communication Tower	Project No.: G 6564-25
Location: 8117 Co Rd 106, Leona, TX	Ground Elev.: N/A
Date Start - Finsh: 07/07/2025 - 07/07/2025	Coordinates: 31.162586, -95.878417

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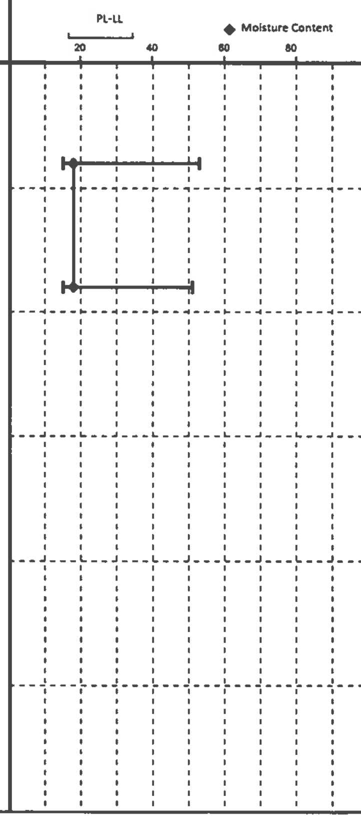
Whitehouse, Texas 75791
903-595-4421

Depth (ft)	Material Description	Field Strength Data N (Blows/ft) PP (tsf)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	% Plus #40	% Plus #4	Wet Density (PCF)	Compressive Strength (tsf)	Shrink/Swell Load Stress (KSF)	% Swell
	Fat Clay with Sand (CH) medium stiff; reddish brown & gray; dry to moist; mottled	N: 12											
5	Stiff to very stiff; brown; moist	PP: 3.5	18.0	53	15	38	83.0	1.0	0.0				
	Gray with brown; iron-oxide staining	PP: 4.5											
10	Gray; iron-oxide staining	PP: 4.5	18.0	51	15	36	82.0	0.0	0.0	126.1		5.26	6.5
13.0	**Sandy Fat Clay** (CH) very stiff; gray; moist; abundant iron-oxide staining; mottled	N: 19											
20	Brown	N: 17											
23.0	**Fat Clay with Sand** (CH) very stiff; gray with brown; laminated; abundant iron-oxide staining	N: 17											
		N: 21											



Field Notes

-

Groundwater Readings

Graphic	Date	Time	Depth	Notes
▽			18'	Water encountered @ 18'
▼			18'	Water encountered @ 18'

ETTL | Engineers & Consultants

Soil Boring: B-4

Project: Tri-City Ranch Communication Tower	Project No.: G 6564-25
Location: 8117 Co Rd 106, Leona, TX	Ground Elev.: N/A
Date Start - Finsh: 07/07/2025 - 07/07/2025	Coordinates: 31.162864, -95.876981

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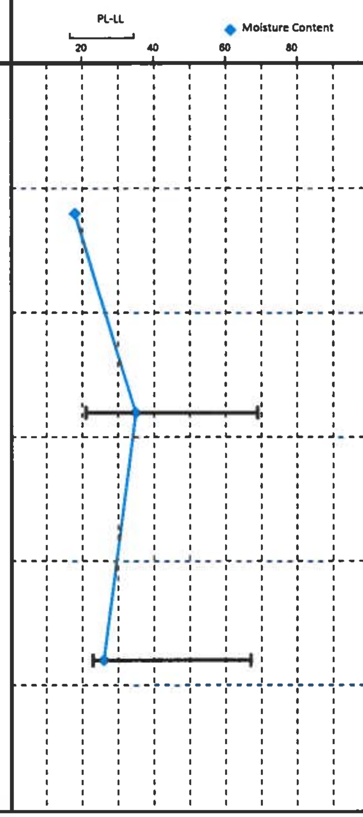
903-595-4421

Depth (ft)	MATERIAL DESCRIPTION	Field Strength Data N (Blows/ft) PP (tsf)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	% Plus #40	% Plus #4	Wet Density (PCF)	Compressive Strength (tsf)	Shrink/Swell Load Stress (KSF)	% Swell
1.0–3.0	**Silty Sand (SM)** loose; brown; very moist	N: 5; PP: 4.5											
3.0–5.0	**Fat Clay with Sand** (CH) hard; reddish brown & gray; moist; mottled	N: 14	18.0				41.0	0.0	0.0				
5.0–8.0	**Clayey Sand (SC)** medium dense; gray; moist; abundant iron-oxide staining	N: 17											
8.0–13.0	**Sandy Fat Clay** (CH) stiff; brown; moist to very moist; iron-oxide staining	N: 13	35.0	69	21	48	98.0	1.0	0.0				
13.0–23.0	**Fat Clay** (CH) stiff; gray and brown; laminated; iron-oxide staining	N: 16											
23.0–	**Fat Clay with Sand** (CH) very stiff to hard; dark gray; moist; traces calcareous nodules; blocky	PP: 4.5+	26.0	67	23	44	75.0	3.0	0.0	106.8	3.75		
		N: 36											



Field Notes

-

Groundwater Readings

Graphic	Date	Time	Depth	Notes
▽			8'	Water encountered @ 8'
▼			13'	Water encountered @ 13'

APPENNDIX B

Laboratory Testing Reports

ETTL Engineers & Consultants

GEOTECHNICAL★MATERIALS★ENVIRONMENTAL★DRILLING★LANDFILLS

LABORATORY TEST DATA SUMMARY SHEET

PROJECT: Tri-City Ranch Tower
ETTL JOB NUMBER: G 6564-25
PROJECT LOCATION: Leona, Texas
CLIENT: Mission Critical Partners
PROJECT MANAGER.: James Werbiski

START DATE: 7/11/2025
FINISH DATE: 7/16/2025
TECHNICIAN(S): JC
DATE SAMPLED: 7/7/2025

Boring No.	Depth (ft.)		Sample No.	Description of Sample	USCS Classification		Atterberg Limits			Moisture Content (%)	(%) Passing No. 200 Sieve	(%) Retained No. 40 Sieve	(%) Retained No. 4 Sieve	Unit Weight / Compression Tests					Consol. / Swell Tests Results			
	Top	Bott					LL	PL	PI					Wet Unit Weight (pcf)	Moisture Content (%)	Compressive Strength (tsf)	Failure Strain (%)	Confining Pressure (psi)	Wet Unit Weight (pcf)	Moisture Content (%)	Free Swell (%)	Restraining Pressure (ksf)
B-1	1.0	3.0		Redd Brown & Gray	SC	Clayey sand	46	18	28	21	37	2	0									
B-1	8.0	10.0		Gray w/ Brown	CH	Fat Clay	74	23	51	33	98	0	0	113.5	28.9	1.51	4.66	6.2				
B-1	18.0	20.0		Brown	CH	Sandy Fat Clay	65	25	40	32	52	19	5									
B-1	23.0	25.0		Gray	CL	Lean Clay with Sand	43	15	28	16	73	1	0									
B-2	3.0	5.0		Redd Brown & Gray	CL	Sandy Lean Clay	49	14	35	15	70	0	0						131.4	14.6	3.9	5.89
B-2	13.0	15.0		Gray	CL	Lean Clay with Sand	40	14	26	17	77	0	0									
B-2	23.0	25.0		Gray	CH	Fat Clay	75	23	52	30	98	1	0	114.9	28.4	1.92	7.33	15.2				
B-3	3.0	5.0		Brown	CH	Fat Clay with Sand	53	15	38	18	83	1	0									
B-3	8.0	10.0		Gray	CH	Fat Clay with Sand	51	15	36	18	82	0	0						126.1	18.7	6.5	5.26
B-4	5.0	7.0		Gray	Visual	Clayey sand				18	41	0	0									
B-4	13.0	15.0		Gray & Brown	CH	Fat Clay	69	21	48	35	98	1	0									
B-4	23.0	25.0		Dark Gray	CH	Fat Clay with Sand	67	23	44	26	75	3	0	106.8	27.2	3.75	7.54	15.1				

NT = Not Tested, Visual Classification
NP = Non Plastic, LL Attempted

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Longview, TX 903-758-0402 ★ Arlington, TX 817-962-0048 ★ Austin, TX 512-519-9312 ★ Texarkana, AR 870-772-0013
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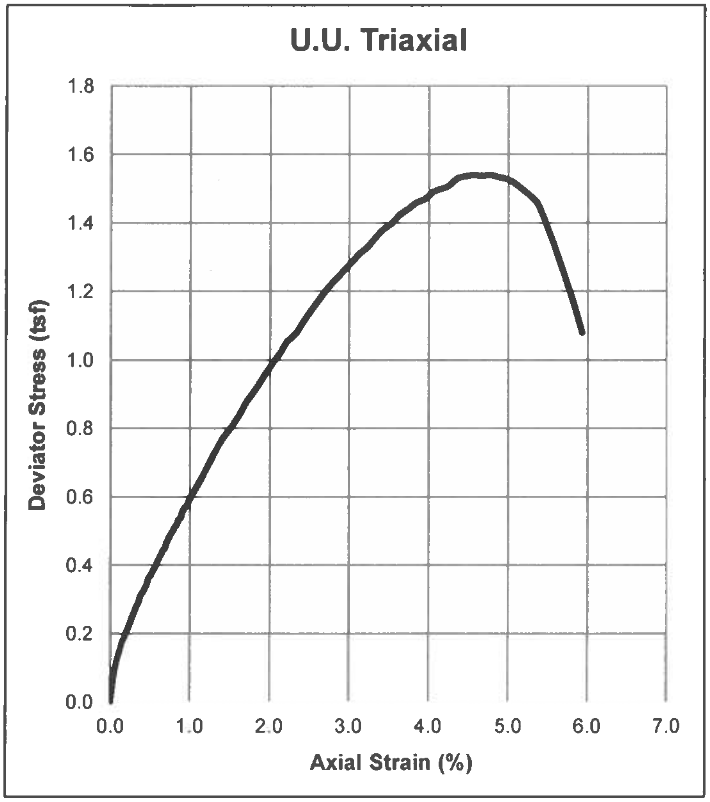
ASTM D 2850 Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

Project: Tri-City Ranch Tower
Client: Leon County
Location: Leona, Texas
Material: Gray w/ Brown Fat Clay (CH)

ETTL Project No.: G 6572-25
Boring No.: B-1
Depth (ft.): 8.0 - 10.0
Sample No.:

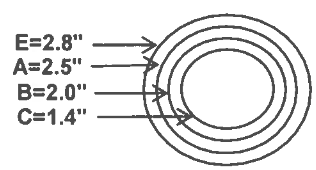


At Test Sample Properties

Height:	4.732	inches
Diameter:	2.014	inches
Height / Diameter Ratio:	2.35	
Initial Moisture Content:	28.9%	(trimmings)
Initial Dry Unit Weight:	88.1	lbs./ft^3
Initial Total Unit Weight:	113.5	lbs./ft^3
Specific Gravity:	2.70	(assumed)
Initial Void Ratio:	0.913	
Initial Saturation:	85.3	%
Pocket Pentronmeter:	N/T	tsf
Hand Torvane:	N/T	tsf
Rate of Strain:	1.0	%/min
Max. Deviator Stress (σ1-σ3):	**1.51**	**tsf**
Confining Pressure σ3:	**6.20**	**psi**
Strain At Failure:	**4.66**	**%**
Secant Modulus at 1/2 Peak Stress:	109	ksf
Strain at 50% Max Stress (e_{50}):	0.014	in/in
Atterbergs LL / PI:	74	51
Passing No. 200 Sieve:	98.0	%
Sampling Method:	Sample Trimmed, Shelby Tube	
Type of Specimen:	Undisturbed	
Date Sampled:	7/7/2025	
Specimen Trimming ID:	B	(trimmed)

Testing Remarks:
Membrane Correction Factor Applied to Deviator Stress (psi) = 0.36

Technician: Hunter Franks   Test Date: 7/11/2025   Report Date: 7/18/2025



Measured Angle of Fracture from Horizontal:
Sketch of Fracture:

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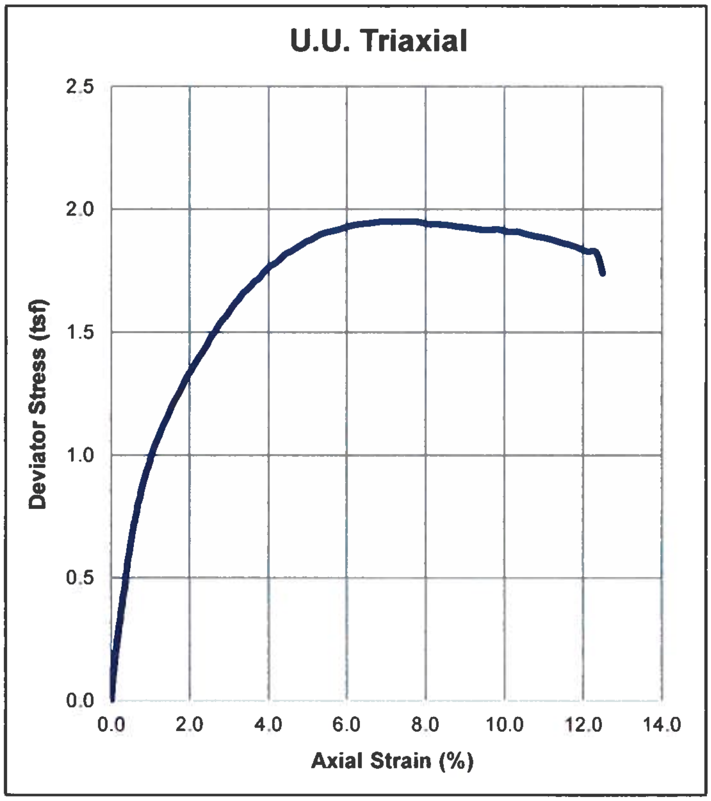
ASTM D 2850 Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

Project:	Tri-City Ranch Tower
Client:	Leon County
Location:	Leona, Texas
Material:	Gray Fat Clay (CH)

ETTL Project No.:	G 6572-25
Boring No.:	B-2
Depth (ft.):	23.0 - 25.0
Sample No.:	



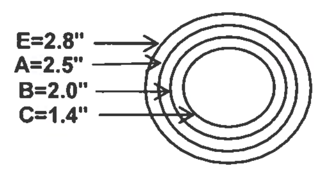
At Test Sample Properties

Height:	4.801	inches
Diameter:	2.020	inches
Height / Diameter Ratio:	2.38	
Initial Moisture Content:	28.4%	(trimmings)
Initial Dry Unit Weight:	89.5	lbs./ft^3
Initial Total Unit Weight:	114.9	lbs./ft^3
Specific Gravity:	2.70	(assumed)
Initial Void Ratio:	0.883	
Initial Saturation:	86.9	%
Pocket Pentronmeter:	N/T	tsf
Hand Torvane:	N/T	tsf
Rate of Strain:	1.0	%/min
Max. Deviator Stress (σ1-σ3):	**1.92**	**tsf**
Confining Pressure σ3:	**15.20**	**psi**
Strain At Failure:	**7.33**	**%**
Secant Modulus at 1/2 Peak Stress:	199	ksf
Strain at 50% Max Stress (e_{50}):	0.010	in/in
Atterbergs LL / PI:	75	52
Passing No. 200 Sieve:	98.0	%
Sampling Method:	Sample Trimmed, Shelby Tube	
Type of Specimen:	Undisturbed	
Date Sampled:	7/7/2025	
Specimen Trimming ID:	B	(trimmed)

Testing Remarks:

Membrane Correction Factor Applied to Deviator Stress (psi) = 0.57

Technician: Hunter Franks Test Date: 7/17/2025 Report Date: 7/18/2025



Measured Angle of Fracture from Horizontal: ____________

Sketch of Fracture:

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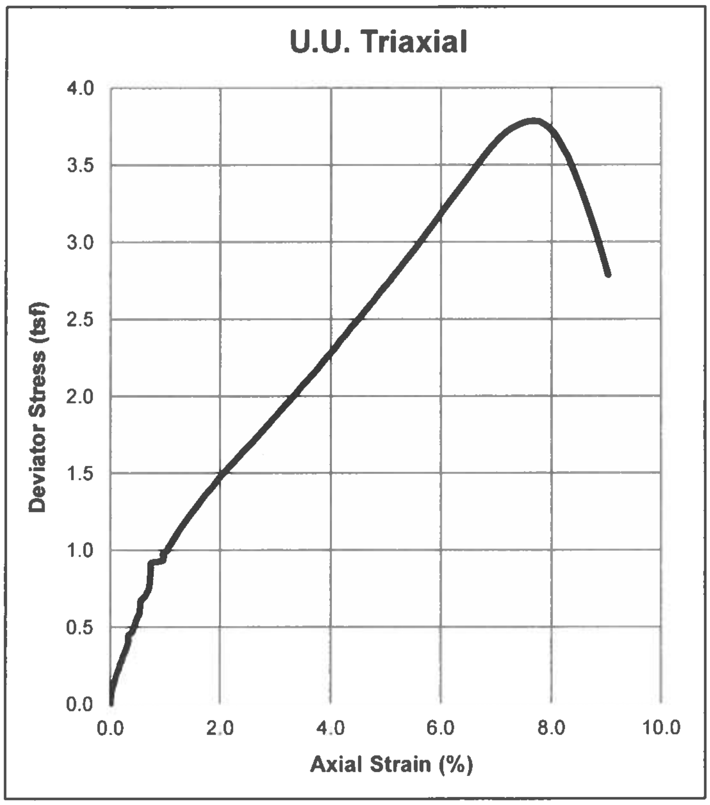
ASTM D 2850 Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

Project:	Tri-City Ranch Tower	ETTL Project No.:	G 6572-25
Client:	Leon County	Boring No.:	B-4
Location:	Leona, Texas	Depth (ft.):	23.0 - 25.0
Material:	Dark Gray Fat Caly w/ Sand (CH)	Sample No.:	



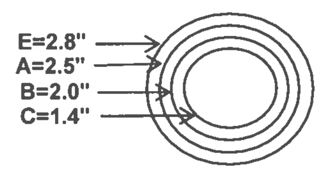
At Test Sample Properties

Height:	5.684	inches
Diameter:	2.778	inches
Height / Diameter Ratio:	2.05	
Initial Moisture Content:	27.2%	(trimmings)
Initial Dry Unit Weight:	83.9	lbs./ft^3
Initial Total Unit Weight:	106.8	lbs./ft^3
Specific Gravity:	2.70	(assumed)
Initial Void Ratio:	1.008	
Initial Saturation:	72.9	%
Pocket Pentronmeter:	N/T	tsf
Hand Torvane:	N/T	tsf
Rate of Strain:	1.0	%/min
Max. Deviator Stress (σ1-σ3):	**3.75**	**tsf**
Confining Pressure σ3:	**15.10**	**psi**
Strain At Failure:	**7.54**	**%**
Secant Modulus at 1/2 Peak Stress:	124	ksf
Strain at 50% Max Stress (e_{50}):	0.030	in/in
Atterbergs LL / PI:	67	44
Passing No. 200 Sieve:	75.0	%
Sampling Method:	2.8 in. Shelby Tube	
Type of Specimen:	Undisturbed	
Date Sampled:	7/7/2025	
Specimen Trimming ID:	E	(whole)

Testing Remarks:

Membrane Correction Factor Applied to Deviator Stress (psi) = 0.43

Technician: Hunter Franks Test Date: 7/10/2025 Report Date: 7/18/2025



Measured Angle of Fracture from Horizontal: ______

Sketch of Fracture:

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ASTM D 4546 One-Dimensional Swell or Settlement of Cohesive Soils, Method A/B Modified

Project Information

Project: Tri-City Ranch Tower
Client/Arch./Engr: Leon County
Project Location: Leona, Texas
ETTL Job No: G 6564-25

Sample Information

Location / Boring No: B-2
Sample No: Depth: 3.0 - 5.0 ft.
Material Origin: Geotechnical Boring
Sampling Info. provided By: ETTL Engineers & Consultants, Inc.
Material Description: Reddish Brown & Gray Sandy Lean Clay (CL)
Sample Type: Undisturbed Shelby Tube Trimmed at In-situ M.C.
Sampled By: ETTL Drilling Date Sampled: 7/7/2025
Technician: Hunter Franks Test Date: 7/10/2025

Test Data

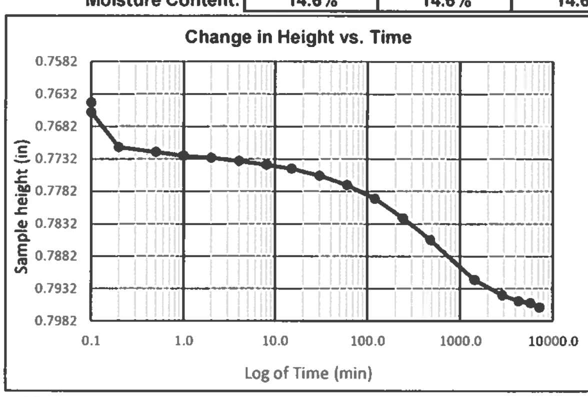
	Sample Data				
	Molded	Overburden Consol.	Restrain Swell	Free Swell	
Wt. of mold + Wet Wt.	199.31	199.31	199.31	203.91	grams
Wt. of mold	68.02	68.02	68.02	68.02	grams
Wet Wt. of sample	131.29	131.29	131.29	135.89	grams
Dry Wt. of sample	114.57	114.57	114.57	114.57	grams
Height of sample	0.7755	0.7645	0.7660	0.7961	inches
Diameter of sample	2.500	2.500	2.500	2.500	inches
Area of sample	4.909	4.909	4.909	4.909	in^2
Volume of sample	3.807	3.753	3.760	3.908	in^3
Degree of Saturation:	**86.0%**	**90.1%**	**89.5%**	**100.0%**	
Void Ratio e:	**0.453**	**0.433**	**0.435**	**0.492**	
Applied Pressure:	**125**	**627**	**5894**	**125**	psf
Assumed Specific Gravity	2.67	2.67	2.67	2.67	
Wet Unit Weight	131.4	133.3	133.0	132.5	pcf
Dry Unit Weight:	**114.7**	**116.3**	**116.1**	**111.7**	pcf
Moisture Content:	**14.6%**	**14.6%**	**14.6%**	**18.6%**	

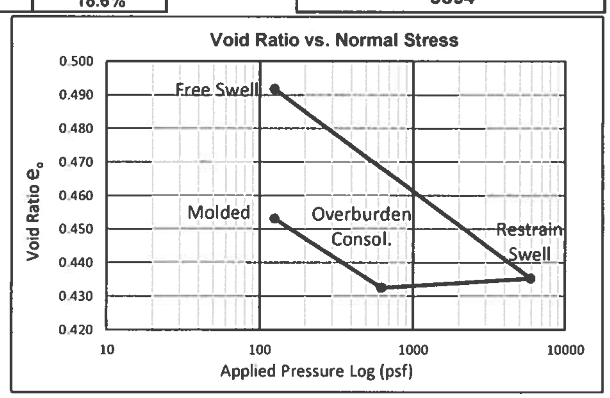
Atterberg Limits	
L.L.	P.L.
49	14
P.I.	-200%
35	70

*N/T = Not Tested

Pocket Penetrometer (tsf)	
Before Test	After Test
4.5+	2.75

USACE Swelling Index - Cs
0.035
Percent Moisture Absorption
4.0%
Percent Free Swell
3.9%
Restrain Pressure (psf)
5894





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ASTM D 4546 One-Dimensional Swell or Settlement of Cohesive Soils, Method A/B Modified

Project Information

Project: Tri-City Ranch Tower
Client/Arch./Engr: Leon County
Project Location: Leona, Texas
ETTL Job No: G 6564-25

Sample Information

Location / Boring No: B-3
Sample No: Depth: 8.0 - 10.0 ft.
Material Origin:: Geotechnical Boring
Sampling Info. provided By: ETTL Engineers & Consultants, Inc.
Material Description: Gray Fat Clay w/ Sand (CH)
Sample Type: Undisturbed Shelby Tube Trimmed at In-situ M.C.
Sampled By: ETTL Drilling Date Sampled: 7/7/2025
Technician: Hunter Franks Test Date: 7/10/2025

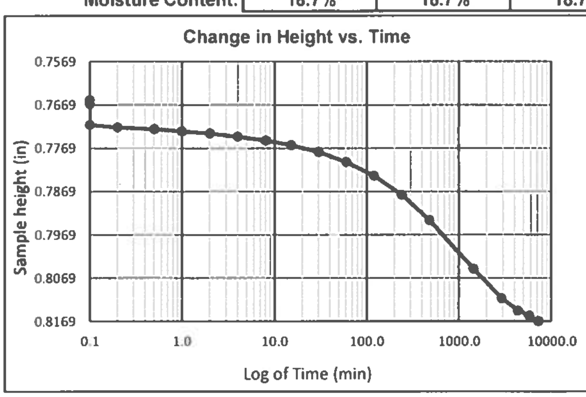
Test Data

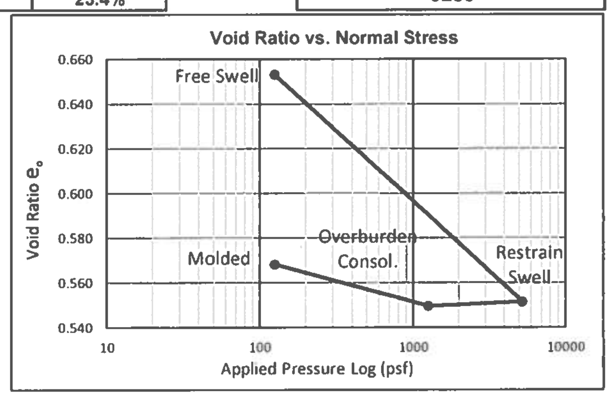
	Sample Data				
	Molded	Overburden Consol.	Restrain Swell	Free Swell	
Wt. of mold + Wet Wt.	190.72	190.72	190.72	195.74	grams
Wt. of mold	64.78	64.78	64.78	64.78	grams
Wet Wt. of sample	125.94	125.94	125.94	130.96	grams
Dry Wt. of sample	106.10	106.10	106.10	106.10	grams
Height of sample	0.7750	0.7658	0.7667	0.8169	inches
Diameter of sample	2.500	2.500	2.500	2.500	inches
Area of sample	4.909	4.909	4.909	4.909	in^2
Volume of sample	3.804	3.759	3.764	4.010	in^3
Degree of Saturation:	**87.9%**	**90.9%**	**90.6%**	**95.8%**	
Void Ratio e:	**0.568**	**0.550**	**0.551**	**0.653**	
Applied Pressure:	**125**	**1254**	**5255**	**125**	psf
Assumed Specific Gravity	2.67	2.67	2.67	2.67	
Wet Unit Weight	126.1	127.6	127.5	124.4	pcf
Dry Unit Weight:	**106.2**	**107.5**	**107.4**	**100.8**	pcf
Moisture Content:	**18.7%**	**18.7%**	**18.7%**	**23.4%**	

Atterberg Limits	
L.L.	P.L.
51	15
P.I.	-200%
36	82

*N/T = Not Tested

Pocket Penetrometer (tsf)	
Before Test	After Test
4.50	1.25
USACE Swelling Index - Cs	
0.064	
Percent Moisture Absorption	
4.7%	
Percent Free Swell	
6.5%	
Restrain Pressure (psf)	
5255	





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APPENDIX C

ASCE Seismic Design Report

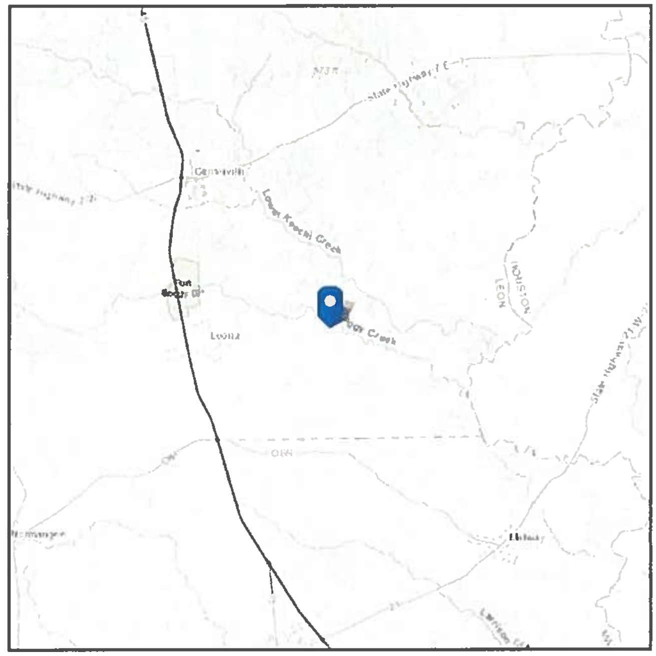
AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-22
Risk Category: III
Soil Class: D - Stiff Soil

Latitude: 31.16294
Longitude: -95.877894
Elevation: 253.53787950756885 ft (NAVD 88)



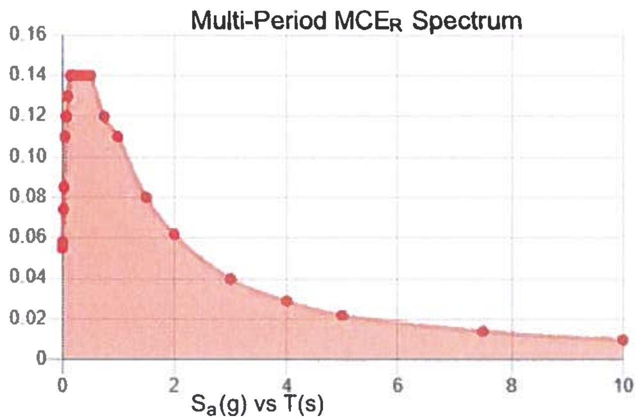
Seismic

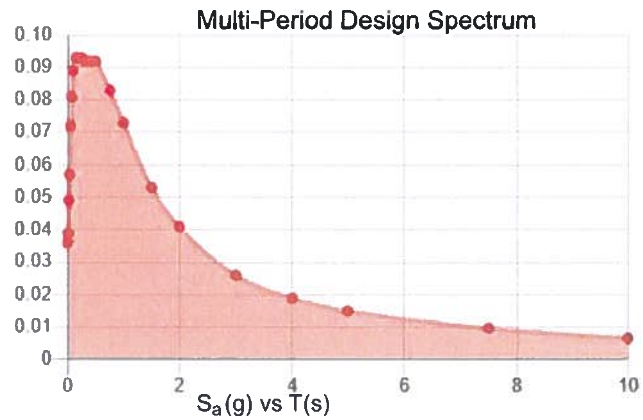
Site Soil Class: D - Stiff Soil

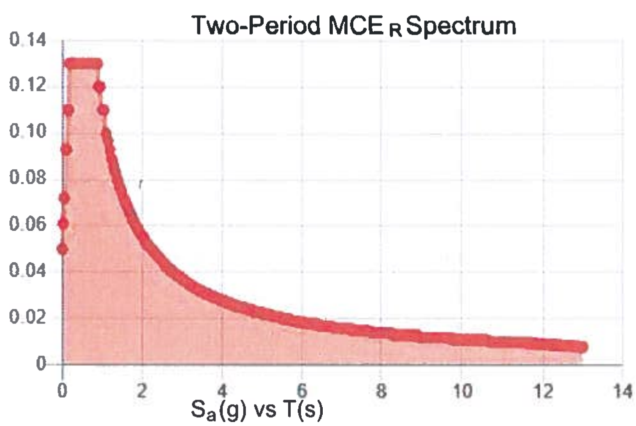
Results:

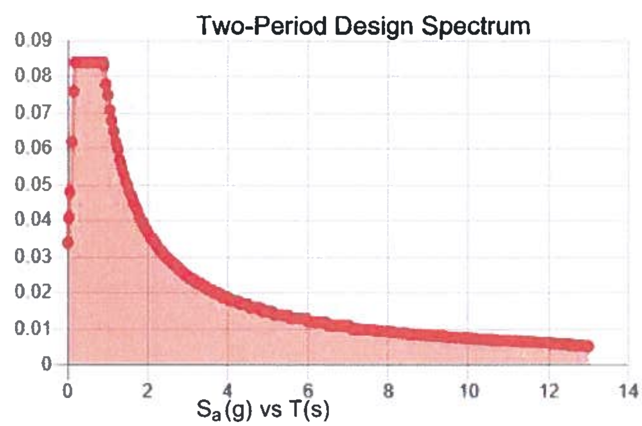
PGA_M :	0.049	T_L :	12
S_{MS} :	0.13	S_S :	0.095
S_{M1} :	0.11	S_1 :	0.052
S_{DS} :	0.084	V_{S30} :	260
S_{D1} :	0.075		

Seismic Design Category: B









MCE$_R$ Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Tue Jul 29 2025**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided "as is" and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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GPD GROUP
520 S. Main Street, Suite 2531
Akron, Ohio 44311
Phone 330.572.2100
www.gpdgroup.com

August 8, 2025

Statement of Compliance

GPD Group Inc., (GPD) has performed a NEPA Report dated August 8, 2025, for the site- Leon County Texas- Tri-Circle.

This NEPA Report was requested and performed to meet accepted standards set by Leon County Texas and the National Historic Preservation Act.

If you have any questions regarding this NEPA Report, please call us at (330) 572-2100.

Sincerely,

GPD Group, Inc.

Sheldon McLeod

Sheldon McLeod
Environmental Scientist

Mail Processing Center
Federal Aviation Administration
Southwest Regional Office
Obstruction Evaluation Group
10101 Hillwood Parkway
Fort Worth, TX 76177

Aeronautical Study No.
2025-ASW-5011-OE
Prior Study No.
2024-ASW-13360-OE

Issued Date: 05/21/2025

RONALD GOLDSMITH
RONALD R GOLDSMITH
1122 Clubhouse Dr.
Mansfield, TX 76063

**** DETERMINATION OF NO HAZARD TO AIR NAVIGATION ****

The Federal Aviation Administration has conducted an aeronautical study under the provisions of 49 U.S.C., Section 44718 and if applicable Title 14 of the Code of Federal Regulations, part 77, concerning:

Structure:	Antenna Tower Tri-Circle Ranch
County, State:	Leon, Texas

Collected Point(s):

Label	Latitude	Longitude	SE	DET AGL	AMSL
Tri-Circle Ranch	31-9-47.03N	95-52-40.05W	256 Ft	420 Ft	676 Ft

This aeronautical study revealed that the structure does not exceed obstruction standards and would not be a hazard to air navigation provided the following condition(s), if any, is(are) met:

As a condition to this Determination, the structure is to be marked/lighted in accordance with FAA Advisory circular 70/7460-1 M Change 1, Obstruction Marking and Lighting, a med-dual system-Chapters 4,8(M-Dual),&15.

Any failure or malfunction that lasts more than thirty (30) minutes and affects a top light or flashing obstruction light, regardless of its position, should be reported immediately to (877) 487-6867 so a Notice to Airmen (NOTAM) can be issued. As soon as the normal operation is restored, notify the same number.

It is required that FAA Form 7460-2, Notice of Actual Construction or Alteration, be e-filed any time the project is abandoned or:

_____ At least 10 days prior to start of construction (7460-2, Part 1)
__X__ Within 5 days after the construction reaches its greatest height (7460-2, Part 2)

This determination expires on 11/21/2026 unless:

(a) the construction is started (not necessarily completed) and FAA Form 7460-2, Notice of Actual Construction or Alteration, is received by this office.
(b) extended, revised, or terminated by the issuing office.

(c) the construction is subject to the licensing authority of the Federal Communications Commission (FCC) and an application for a construction permit has been filed, as required by the FCC, within 6 months of the date of this determination. In such case, the determination expires on the date prescribed by the FCC for completion of construction, or the date the FCC denies the application.

NOTE: REQUEST FOR EXTENSION OF THE EFFECTIVE PERIOD OF THIS DETERMINATION MUST BE E-FILED AT LEAST 15 DAYS PRIOR TO THE EXPIRATION DATE. AFTER RE-EVALUATION OF CURRENT OPERATIONS IN THE AREA OF THE STRUCTURE TO DETERMINE THAT NO SIGNIFICANT AERONAUTICAL CHANGES HAVE OCCURRED, YOUR DETERMINATION MAY BE ELIGIBLE FOR ONE EXTENSION OF THE EFFECTIVE PERIOD.

This determination is based, in part, on the foregoing description which includes specific coordinates, heights, frequency(ies) and power. Any changes in coordinates, heights, and frequencies or use of greater power, except those frequencies specified in the Colo Void Clause Coalition; Antenna System Co-Location; Voluntary Best Practices, will void this determination. Any future construction or alteration, including increase to heights, power, or the addition of other transmitters, requires separate notice to the FAA.This determination includes all previously filed frequencies and power for this structure.

If construction or alteration is dismantled or destroyed, you must submit notice to the FAA within 5 days after the construction or alteration is dismantled or destroyed.

This determination does include temporary construction equipment such as cranes, derricks, etc., which may be used during actual construction of the structure. However, this equipment shall not exceed the overall heights as indicated above. Equipment which has a height greater than the studied structure requires separate notice to the FAA.

This determination concerns the effect of this structure on the safe and efficient use of navigable airspace by aircraft and does not relieve the sponsor of compliance responsibilities relating to any law, ordinance, or regulation of any Federal, State, or local government body.

A copy of this determination will be forwarded to the Federal Communications Commission (FCC) because the structure is subject to their licensing authority.

If we can be of further assistance, please contact our office at 1-817-222-5922, or debbie.cardenas@faa.gov. On any future correspondence concerning this matter, please refer to Aeronautical Study Number 2025-ASW-5011-OE.

Signature Control No: 654991962-660089951 (DNE)

debbie.cardenas@faa.gov
Technician

Attachment(s)
Frequency Data
Map(s)

cc: FCC

LOW FREQUENCY	HIGH FREQUENCY	FREQUENCY UNIT	ERP	ERP UNIT
6	7	GHz	42	dBW
6	7	GHz	55	dBW
10	11.7	GHz	42	dBW
10	11.7	GHz	55	dBW
153.86	155.695	MHz	200	W
153.89	154.385	MHz	200	W
153.95	154.445	MHz	200	W
153.995	158.745	MHz	200	W
154.22	159.0075	MHz	200	W
156.15	159.49	MHz	200	W
698	806	MHz	1000	W
806	824	MHz	500	W
806	901	MHz	500	W

TOPO Map for ASN 2025-ASW-5011-OE