



SCALE: 1" = 200'

The County of Leon, a Political Subdivision of
the State of Texas
Called 124.21 Acres
1393/330

N:10491540.08
E:3643931.39
LAT:31°22'45.45"
LON:96°00'52.58"

N:10491513.32
E:3644493.72
LAT:31°22'44.97"
LON:96°00'46.11"

N:10491376.78
E:3644203.95
LAT:31°22'43.73"
LON:96°00'49.51"

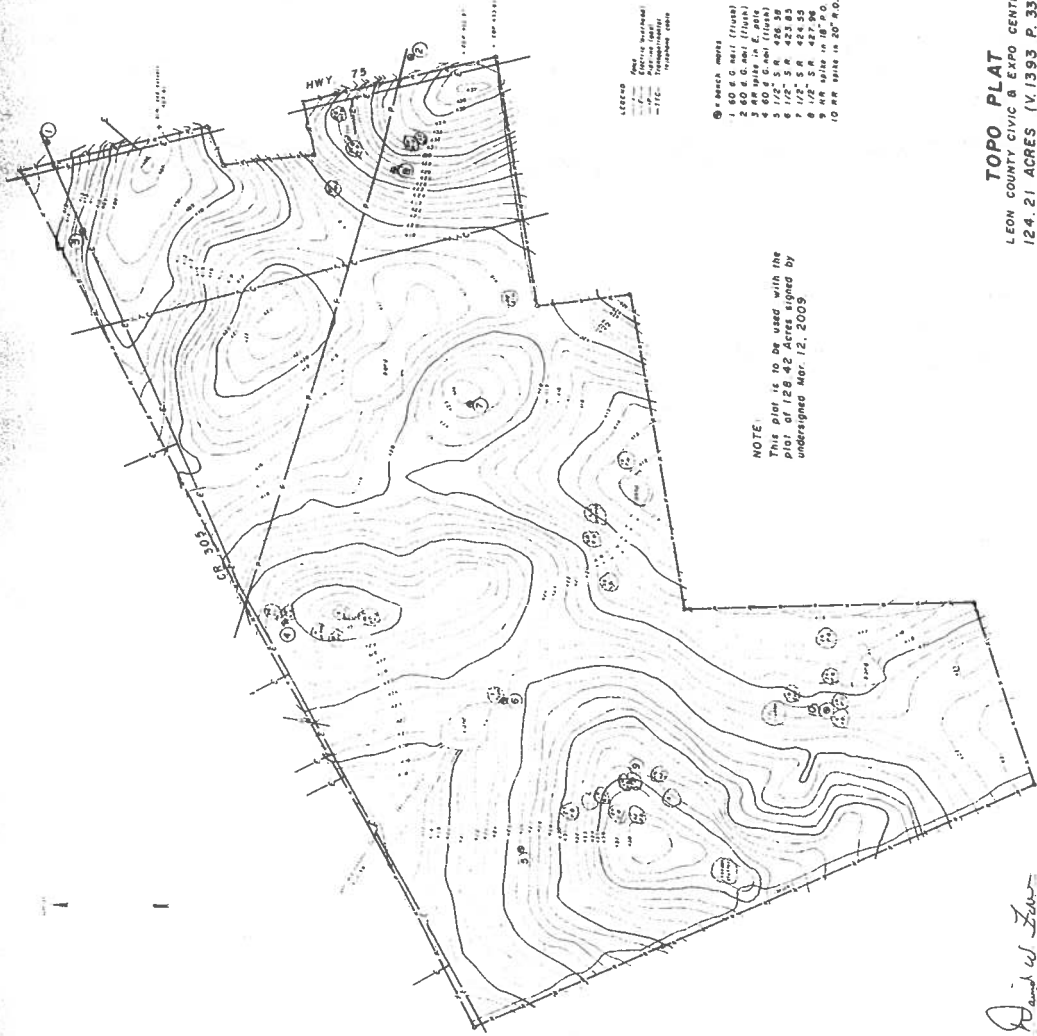
N:10491054.83
E:3644169.63
LAT:31°22'40.56"
LON:96°00'50.05"

Property line

"Preliminary, this document shall not be recorded for any purpose and shall not be used or viewed or relied upon as a final survey document."

**EXHIBIT
COORDINATES OF TOWER
POINTS STAKED AT THE
LEON COUNTY EXPO CENTER
JANUARY 16, 2025**

PREPARED BY:
ISBELL LAND SURVEYING
1366 CR 320/CENTERVILLE, TX/Ph. 979-255-9177
FIRM REGISTRATION NO. 10192000
JOB NO. 25002




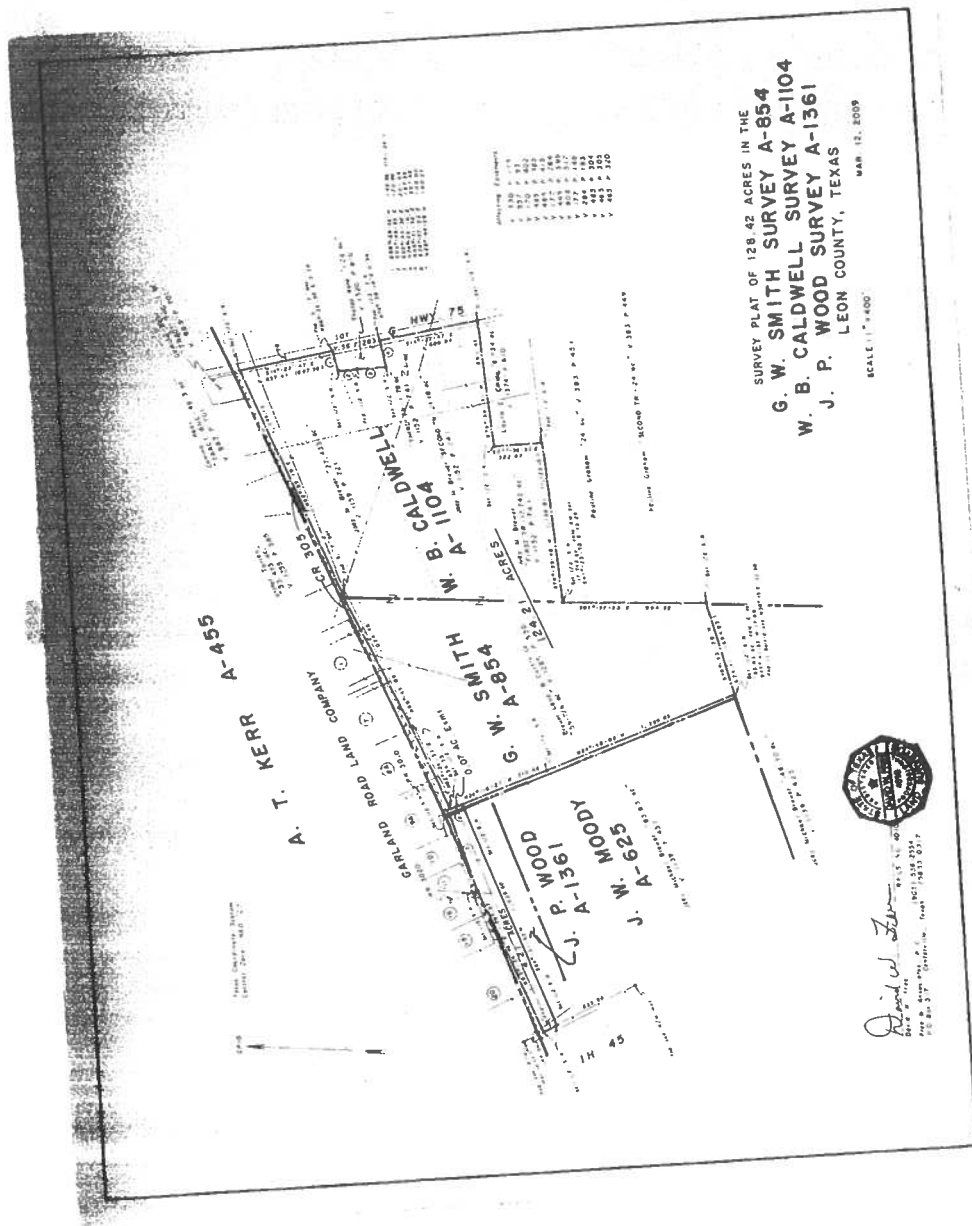
LEGEND
 --- Power line (overhead)
 --- Power line (underground)
 --- Telephone line
 --- Fire hydrant

- ① 4 inch nails
 1 60' x 60' (1/4 inch) 426.04
 2 60' x 60' (1/4 inch) 431.08
 3 60' x 60' (1/4 inch) 436.12
 4 60' x 60' (1/4 inch) 441.16
 5 1/2" x 8" 426.38
 6 1/2" x 8" 427.50
 7 1/2" x 8" 428.62
 8 1/2" x 8" 429.74
 9 1/2" x 8" 430.86
 10 1/2" x 8" 431.98

NOTE
 This plot is to be used with the
 plat of the same name and by
 reference to the plat signed by
 undersigned Mar. 12, 2003.

TOPO PLAT
 LEON COUNTY CIVIC & EXPO CENTER
 124.21 ACRES (V.1393 P.330)
 LEON COUNTY, TEXAS
 SCALE 1" = 200' MD-23, 2009

David W. Jones
 Surveyor
 State of Texas
 No. 13,134, 2003
 Exp. Mar. 31, 2009




SURVEY PLAT OF 128.42 ACRES IN THE
G. W. SMITH SURVEY A-854
W. B. CALDWELL SURVEY A-1104
J. P. WOOD SURVEY A-1361
A. T. KERR A-455
 LEON COUNTY, TEXAS

MAR. 12, 2009

SCALE 1" = 400'

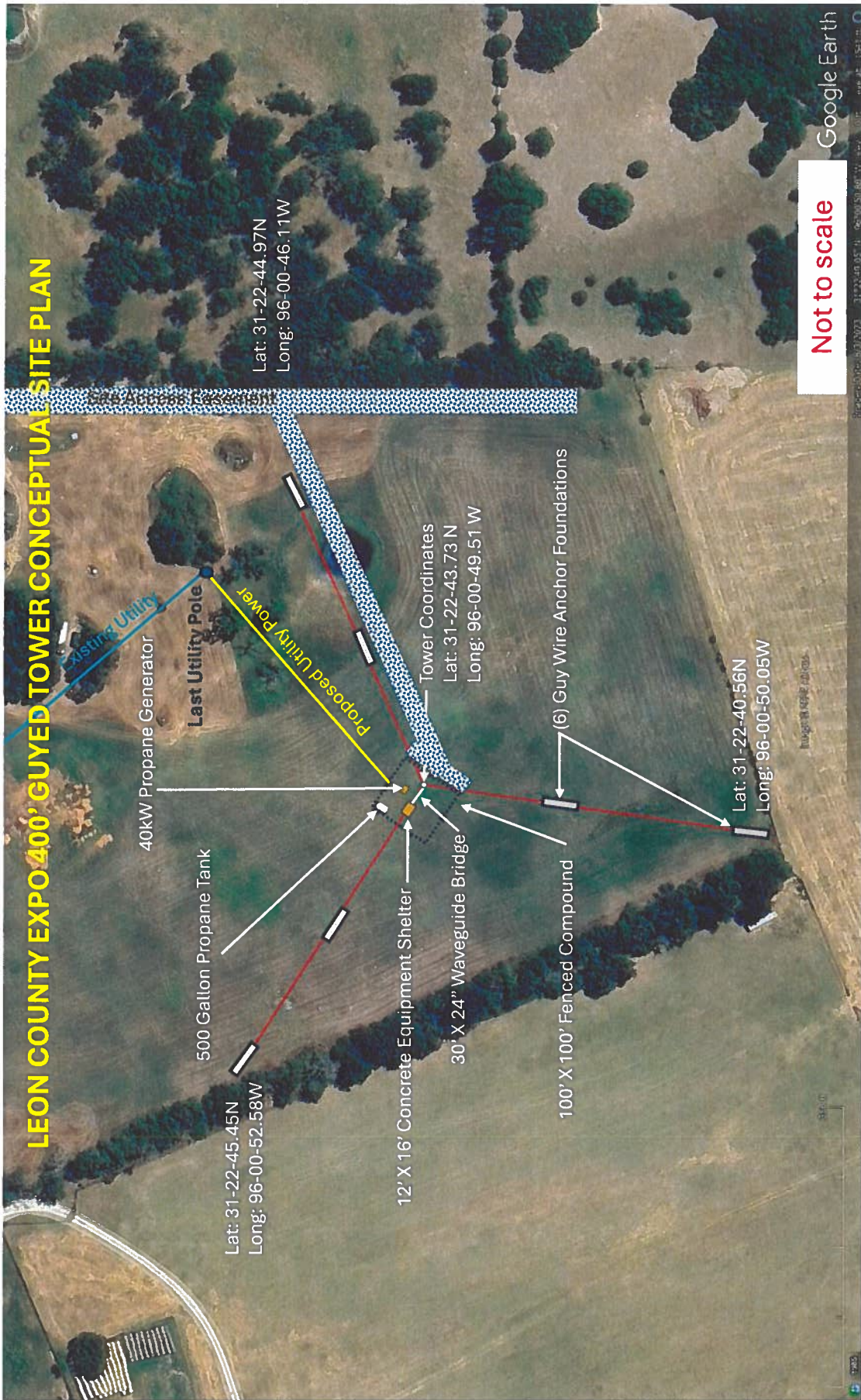
Offering Estimates

1	200	P. 181
2	150	P. 182
3	150	P. 183
4	150	P. 184
5	150	P. 185
6	150	P. 186
7	150	P. 187
8	150	P. 188
9	150	P. 189
10	150	P. 190
11	150	P. 191
12	150	P. 192
13	150	P. 193
14	150	P. 194
15	150	P. 195
16	150	P. 196
17	150	P. 197
18	150	P. 198
19	150	P. 199
20	150	P. 200



David W. Lutz
 State of Texas, Surveyor
 No. 10888, Expires 12/31/12
 2009

LEON COUNTY EXPO 400' GUYED TOWER CONCEPTUAL SITE PLAN



Lat: 31-22-44.97N
Long: 96-00-46.11W

Existing Utility

Last Utility Pole

Lat: 31-22-45.45N
Long: 96-00-52.58W

40kW Propane Generator

500 Gallon Propane Tank

12' X 16' Concrete Equipment Shelter

30' X 24' Waveguide Bridge

Tower Coordinates

Lat: 31-22-43.73 N
Long: 96-00-49.51 W

100' X 100' Fenced Compound

(6) Guy Wire Anchor Foundations

Lat: 31-22-40.56N
Long: 96-00-50.05W

0 500

Not to scale

Google Earth

ETTL | Engineers & Consultants

GEOTECHNICAL * MATERIALS * ENVIRONMENTAL * DRILLING * LANDFILLS

March 17, 2025

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

SUBJECT: Leon County – EXPO Tower
Buffalo, Texas
Geotechnical Investigation
ETTL Job No. G 6477-25

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 concerning this report, or if we can be of further assistance during construction, please 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, E.I.T.
Project Manager


Owen B. Sanderson, P.E.
Senior Engineer

Owen B.
Sanderson

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Date: 2025.03.14 10:31:59 -05'00'



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Geotechnical Investigation

**Leon County
EXPO Tower**

Buffalo, Texas

Submitted To:

TJ Foley
Leon County
Centerville, Texas

Prepared by;

ETTL Engineers & Consultants Inc.
Tyler, Texas

March 2025

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APPENDIX A

Plate I: Plan of Borings
 Log of Borings with Laboratory Test Data

APPENDIX B

Laboratory Test Reports

APPENDIX C

USGS Seismic Design Report

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 January 9th, 2025. The field operations were conducted on February 24th – 25th, 2025.

The purpose of this investigation was to define and evaluate the general subsurface conditions of the proposed tower site located at the Leon County Expo Center in Buffalo, 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 performed on selected soil samples to provide the data used to form 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 pre-marked by the client. E TTL did not confirm by a 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-foot diameter
 - 5.5 foot tall
 - 4.5 foot 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 structure. Google Earth data shows there used to be a small pond located near the northeast anchor. At the time of the drilling operations, the pond had been covered up with backfill. The general topography of the site is relatively flat.

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 track-mounted drilling rig utilizing solid stem auger drilling procedures was used to advance the borings. Soils were sampled utilizing a 1 3/8-inch I.D. by 24-inch-long split-spoon sampler coupled with the modified TxDOT hammer, (170-pound falling 24 inches). In conjunction with this sampling technique, the Standard Penetration Test was conducted. 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 1.25 to conservatively convert the N values from the TxDOT manual hammer to the standard N_{60} value for use in correlations to predict engineering properties. 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 60 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 2 feet to 15 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	15.0
B-2	30	Northwest Anchor	20.0
B-3	30	South Anchor	2.0
B-4	30	East Anchor	15.0

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 E TTL should be contacted to develop a plan for piezometer installation and monitoring to assess the groundwater levels more accurately 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	13
Dry Sieve Analysis (% Passing No. 40)	ASTM D 6913	13
Washed Sieve Analysis (% Passing No. 200)	ASTM D 1140	13
Atterberg Limits (Liquid & Plastic Limits)	ASTM D 4318	13
Moisture Content by Dry Weight	ASTM D 2216	13

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, Tyler Sheet, the proposed site is located in the Queen City Sand Formation (Eqc).

The Queen City Sand formation is described as fine-grained to locally medium-grained quartz sand found in a series of laminated or thinly stratified white and red sands and sandy clays, frequently merging into one another and forming a mottled sandy clay or clayey sand. Ironstone concretions, sometimes occurring as ledges, are common within the formation. Upper sands rest on a series of black, blue, and gray micaceous sands, blue, brown, and gray clays with thin strata

of sandstones and limestone. Thickness of the formation ranges from 100 to 400 feet and is generally thinning southeastward. The age is Eocene and can be found mapped throughout the Tyler Sheet.

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

<http://ngmdb.usgs.gov/Geolex>

<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)
B-1	1	Expansive clays	0 - 5
	2	Medium dense sands	5 - 18
	3	Very dense sands	18 - 30
B-2	1	Expansive clays	0 - 13
	2	Medium dense sands	13 - 18
	3	Very dense sands	18 - 30
B-3	1	Very loose sands	0 - 3
	2	Expansive clays	3 - 8
	3	Alternating medium-dense to dense sands	8 - 23
	4	Very dense sands	23 - 30
B-4	1	Very loose to loose sands	0 - 5
	2	Medium dense expansive clayey sands	5 - 13
	3	Dense sands	13 - 20
	4	Expansive clays	20 - 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	500	110	28	Stiff Clay w/o Free Water	SP, SP-SM, SM, SC
Medium Dense Sand (10<N<30)	Where occurs	1200	110	30	Stiff Clay w/o Free Water	SP, SP-SM, SM, SC
Dense Sand (N>30)	Where occurs	1500	115	32	Stiff Clay w/o Free Water	SP, SP-SM, SM, SC
Medium Stiff to 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. 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 be any of the following soil types: Clayey Sand (SC), Lean Clay (CL), or Fat Clay (CH), which exhibit the ability to change volume (shrink or swell) with the addition or subtraction of moisture. Expansive soils such as are found at various depths throughout the soil profile swell when they absorb moisture and shrink as they dry. Structures placed on these soils move up and down with such volume changes of the soil. When expansive soils are covered by an impermeable surface such as a structure or pavement, seasonal moisture fluctuation at the interior of the covered area tends to be reduced or eliminated due to the lack of exposure to natural wetting and drying conditions (i.e., wind, rain, sun, vegetative, etc.). At the perimeter of the structure, however, infiltration into the foundation soils from surface drainage could lead to local swelling of the clays (if they were dry at the start of construction) resulting in tilt or distortion of the foundation. Where areas immediately adjacent to the structure are paved, both the risk of swelling due to excess moisture absorption and shrinkage due to moisture loss are reduced significantly.

6.5 Vertical Heave Predictions

The assessment of the impact of expansive soils given below is predicated on soil moisture change that is a result of normal climatological fluctuation only. Factors such as poor drainage and consequent ponding water, plumbing leakage, excavation details (e.g., permeable backfill in trenches or beneath structures), and vegetation (trees and shrubs) can result in moisture changes (and consequent swelling or shrinkage) outside the ranges predicted herein. The predicted heave is also the predicted differential movement that could be experienced by a lightly loaded slab and/or foundation placed on native soils.

The actual movement experienced by a given portion of the structure is dependent on a complex interaction of the various factors noted above. The accuracy of predicted movement is determined by how well the prediction accounts for these factors. The TxDOT PVR method is widely accepted for the prediction of shrink/swell movement potential but is derived from empirical data and established correlations with Atterberg Limits and can be used in some soil types. Moisture conditions can be inconsistent in predicting the maximum free swell. The USACE method, on the other hand, is based on the measured swell of actual specimens and is, thus, considered superior to the PVR method in accounting for the soil heaving characteristics at the moisture condition assumed.

Footings and Anchor Blocks

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 change is considered to be low for the tower base and anchors if the subgrade is prepared per **Section 9.0** based on 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 is a prediction of vertical heave based on test results determined from a consolidometer restrain/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.

Moisture State¹	TxDOT PVR (in)
Dry	2.85
Moist	1.25
Existing Condition	2.1

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 level of acceptable movement varies with the design and other requirements, but methods are normally selected to limit slab/foundation movements to about 1.0 inch or less. Reductions in anticipated movements can be achieved by using methods developed in this area to reduce on-grade slab movements. The more commonly used method entails the removal and replacement with compacted select fill or moisture conditioning of the native soils. The use of these methods will reduce, but not eliminate, the risk of unacceptable movements. These methods of subgrade preparation are presented as options for the owner's consideration. The options may or may not be practical or economically feasible, depending on the expected performance of the proposed structure(s). The owner should be aware that these methods will not prevent the movement of soil-supported foundation elements and are only designed to reduce the magnitude of the predicted movement. Placement of the floor slab on grade represents a compromise between construction cost and risk of floor (and related element) distress. A more detailed explanation of these risks can be provided upon request. **Section 9.0** provides more details on the implementation of the methods set forth below.

6.6.1 Replacement of the Expansive Clay with Select Fill

Based on the conditions encountered at the site and assumed dry conditions, excavating 4.0 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**. E TTL 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 satisfied in the selection of 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. If it is desired

to virtually eliminate the risk of damage from vertical movement due to 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 essential to reduce the potential for excessive movement caused by 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 in subgrade with the engineering characteristics of **Layer 1**. 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'	6,000
Anchor Blocks	9'	3,200
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 load from the centroid of the footing. Where footings will be 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 anticipated 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 much less than the factored demand loading, and the maximum footing width recommended can be exceeded and still maintain 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
 $\tan \delta = \tan \phi$ (ϕ = drained friction angle of the soil for the case where concrete is cast against the soil. Use $\tan \delta = 0.8 * \tan \phi$ for precast concrete)

For undrained loading:

$$R_T = c; \text{ or the lesser of: } R_T = c \text{ vs. } R_T = 0.5 * V$$

- 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:

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

- γ = effective unit weight
- K_p = Passive pressure coefficient = $(\tan(45+\phi'/2))^2$ (equals 1 for undrained $\phi=0$ 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 such a manner as to provide solid contact of the side of the concrete with 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 founded below the groundwater are desired, we recommend using the excavation and recompacting procedure set forth in **Section 9.3**.

8.0 FLOOR SYSTEMS AND FLATWORK

Floor systems 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 consequent damage) as noted throughout this report can be tolerated. The level of acceptable movement varies with the design and other requirements, but methods are normally selected with the goal of limiting predicted slab movements to about 1.0 inch or less. Reductions in anticipated movements can be achieved by using subgrade modification methods developed for that purpose. The more commonly used method in this area consists of undercutting and replacing the expansive soils. The use of this method will reduce but *not eliminate* the risk 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 Slab/Flatwork on Grade Subgrade Preparation

PVR<1.0 inch

Over-excavate to a depth of existing grade minus 4 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=2.85 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 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 9.5**) 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 2 feet below grade will likely encounter local perched water or permeant groundwater and, if so, will experience infiltration that will need to be dealt with 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 depends 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. E TTL 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 E TTL 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 E TTL 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 site-wide-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 not 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 Not 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 not 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 not building-envelope or mold specialists.*



Telephone: 301/565-2733

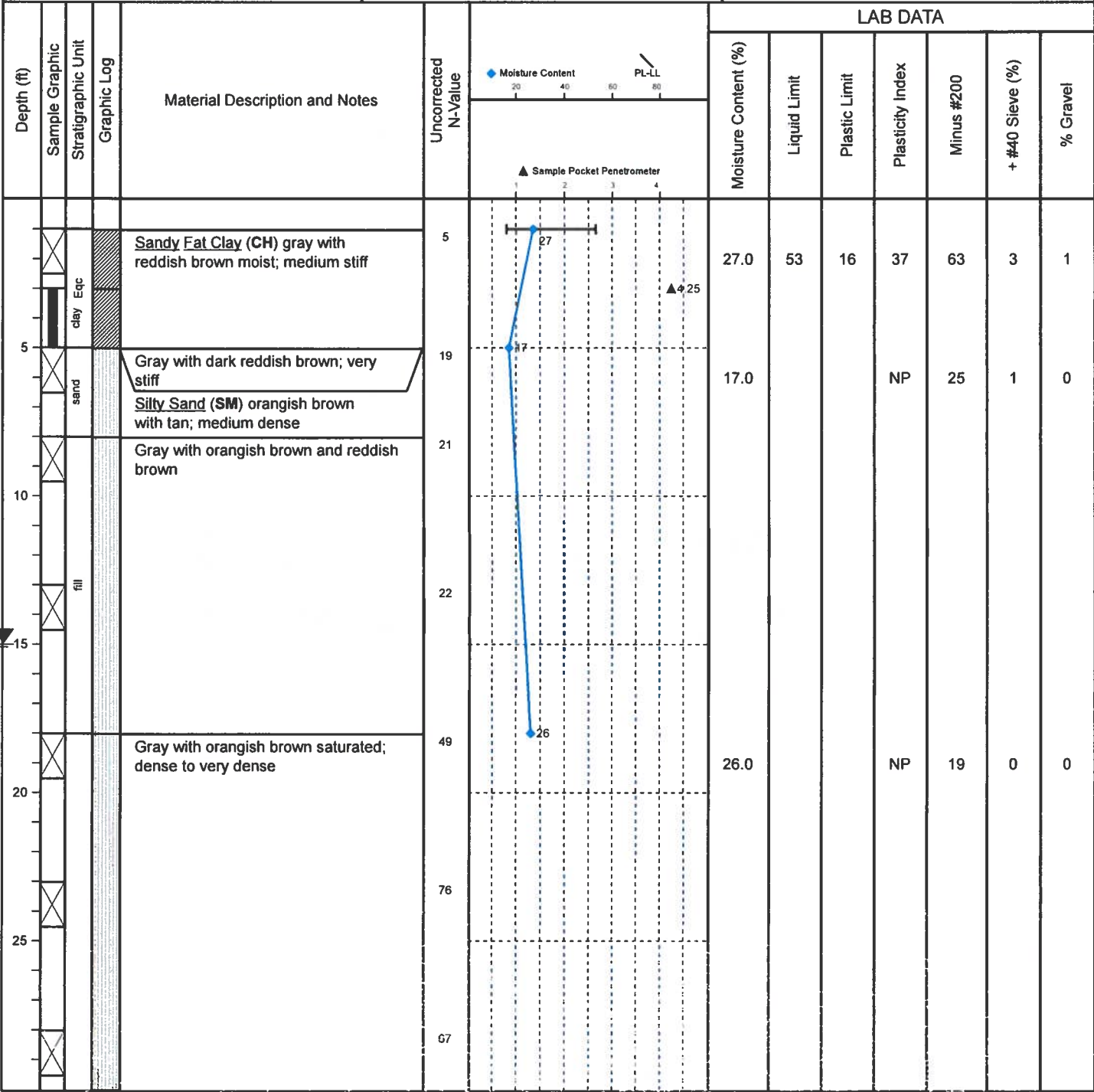
e-mail: info@geoprofessional.org www.geoprofessional.org

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APPENDIX A

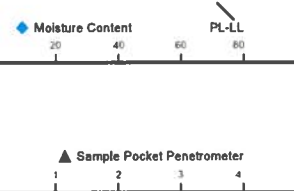
Plan of Borings and Boring Logs

Client Name: Ronald Goldsmith	Drilling Co.: E TTL	Remarks:
Job #: G 6477-25	Driller: Jawan	
Log Prepared By: James Werbiski	Rig Type: -	
Drilling Date(s): 02/24/2025 - 02/25/2025	Surface Elevation: N/A	
Drilling Method: Auger	Coordinates: 31.37895, -96.01398	
Water Encounter: N/A	∇ Cave-in At Time Of Drilling 21' Water Encounter N/A	∇ Delayed Water Level 15' Date Drilled 02/24/25



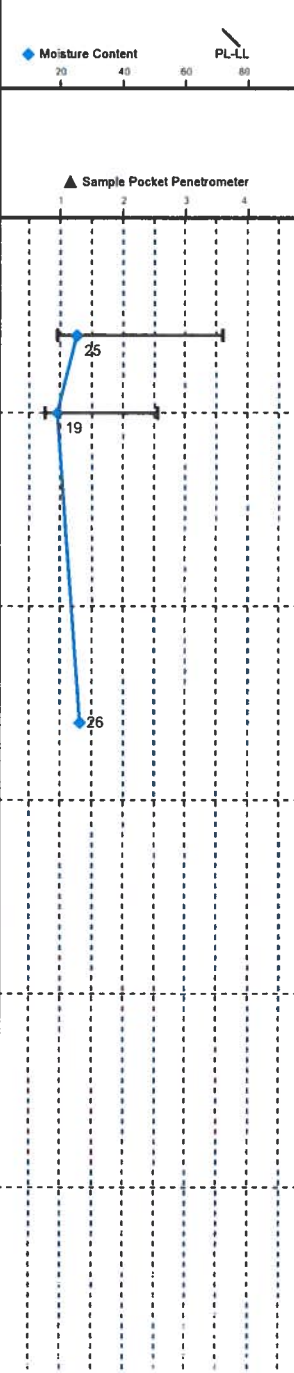
Client Name: Ronald Goldsmith	Drilling Co.: E TTL	Remarks:
Job #: G 6477-25	Driller: Jawan	
Log Prepared By: James Werbiski	Rig Type: -	
Drilling Date(s): 02/24/2025 - 02/25/2025	Surface Elevation: N/A	
Drilling Method: Auger	Coordinates: 31.37929, -96.01461	
Water Encounter: N/A	⚠ Cave-in At Time Of Drilling 23'	⚠ Delayed Water Level 20'
	Water Encounter N/A	Date Drilled 02/24/25

Depth (ft)	Sample Graphic Stratigraphic Unit	Graphic Log	Material Description and Notes	Uncorrected N-Value	LAB DATA						
					Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	+ #40 Sieve (%)	% Gravel
8	Eqc		<u>Fat Clay with sand (CH)</u> gray with dark reddish brown moist; trace organics; medium stiff		21.0	56	16	40	75	0	0
19			<u>Sandy Fat Clay (CH)</u> gray with reddish brown and brown; Iron-Oxide Staining; stiff; Mottled								
22			<u>Lean Clay with sand (CL)</u> orangish brown with gray and dark reddish brown; very stiff		23.0	46	17	29	74	0	0
25			<u>Silty Sand (SM)</u> gray moist; Iron-Oxide Staining; medium dense								
75			Light brown with gray saturated; very dense								
78			Gray								
50											



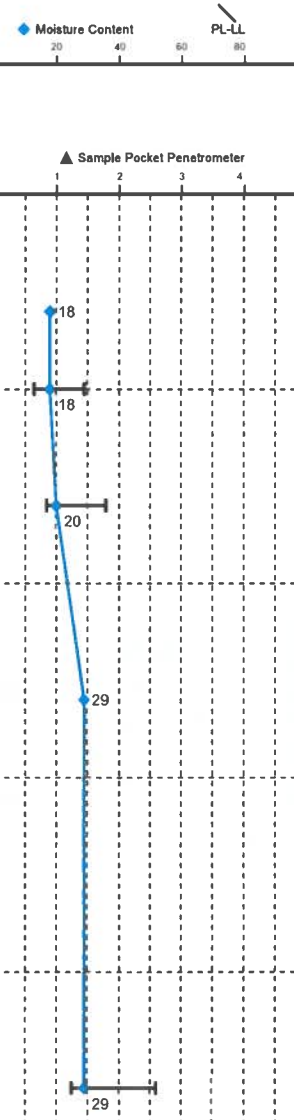
Client Name: Ronald Goldsmith	Drilling Co.: E TTL	Remarks:
Job #: G 6477-25	Driller: Jawan	
Log Prepared By: James Werbiski	Rig Type: -	
Drilling Date(s): 02/24/2025 - 02/25/2025	Surface Elevation: N/A	
Drilling Method: Auger	Coordinates: 31.37793, -96.0139	
Water Encounter: N/A	∇ Cave-in At Time Of Drilling 3'	∇ Delayed Water Level 2'
	Water Encounter N/A	Date Drilled 02/24/25

Depth (ft)	Sample Graphic	Stratigraphic Unit	Graphic Log	Material Description and Notes	Uncorrected N-Value	LAB DATA										
						Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	+ #40 Sieve (%)	% Gravel				
2				Silty Sand (SM) brown wet; very loose												
5				Fat Clay with sand (CH) gray with dark reddish brown moist; medium stiff		25.0	72	19	53	72	0	0				
14				Sandy Fat Clay reddish brown with gray		19.0	51	15	36	50	0	0				
45				Silty Sand (SM) tan; dense												
16				Lt. reddish brown; medium dense to dense		26.0			NP	25	2	1				
81				Light gray; very dense												
90																



Client Name: Ronald Goldsmith	Drilling Co.: E TTL	Remarks:
Job #: G 6477-25	Driller: Jawan	
Log Prepared By: James Werbiski	Rig Type: -	
Drilling Date(s): 02/24/2025 - 02/25/2025	Surface Elevation: N/A	
Drilling Method: Auger	Coordinates: 31.37924, -96.01267	
Water Encounter: N/A	☒ Cave-in At Time Of Drilling N/A	▼ Delayed Water Level 15'
	Water Encounter N/A	Date Drilled 02/24/25

Depth (ft)	Sample Graphic Unit	Graphic Log	Material Description and Notes	Uncorrected N-Value	LAB DATA										
					Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	+ #40 Sieve (%)	% Gravel				
4	Eqc		Silty Sand (SM) brown moist; very loose												
8			Brown with orangish brown; Iron-Oxide Staining; loose		18.0		NP	NP	48	6	4				
14			Clayey Sand (SC) grayish brown with reddish brown; Iron-Oxide Staining; medium dense		18.0	29	13	16	41	0	0				
34			Gray with dark reddish brown; dense		20.0	36	17	19	37	0	0				
32			Silty Sand (SM) light gray saturated; dense		29.0		NP	NP	19	0	0				
32			Fat Clay with sand (CH) brown with dark brown; moist; interbedded iron-ore seams; very stiff to hard		29.0	52	25	27	81	5	0				



Boring Log Descriptive Terminology

Key to Soil Symbols and Terms

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	Well-graded gravels, gravel sand mixtures, little or no fines.
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
				GM	Silty gravels, gravel-sand-silt mixtures.
	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	Well-graded sands, gravelly sands, little or no fines.
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SP	Poorly graded sands, gravelly sands, little or no fines.
				SM	Silty sands, sand-silt mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
				OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	Inorganic silts, micaceous or discontinuous fine sandy or silty silts, elastic silts.
				CH	Inorganic clays of high plasticity, fat clays.
				OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS				PT	Peat and other highly organic soils.

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Notes

SPT (Standard Penetration Test-ASTM D1586):

The number of blows of a 140 lb (63.6 kg) hammer falling 2.5 ft (750 mm) used to drive a 2 in (50 mm) O.D. Split Spoon sampler for a total of 1.5 ft (0.45 m) of penetration.

Written as follows:

first 0.5 ft (0.15 m) - second 0.5 ft (0.15 m) - third 0.5 ft (0.15 m)
(ex: 1-3-9)

Note: If the number of blows exceeds 50 before 0.5 ft (0.15 m) of penetration is achieved, the actual penetration follows the number of blows in parentheses

(ex: 12-24-50 (0.09 m), 34-50 (0.4 ft), or 100 (0.3 ft)).

WR denotes a zero blow count with the weight of the rods only.

WH denotes a zero blow count with the weight of the rods plus the weight of the hammer.

Soil Classifications are Based on the Unified Soil Classification System, ASTM D2487 and D2498. Also included are the AASHTO group classifications (M145). Descriptions are based on visual observation, except where they have been modified to reflect results of laboratory tests as deemed appropriate.

Order of Descriptors

- Group Name
- Consistency or Relative Density
- Moisture Condition
- Color
- Particle size descriptor(s) (coarse grained soils only)
- Angularity of coarse grained soils
- Other relevant notes

Criteria For Descriptors

Consistency of Fine Grained Soils

Consistency

Very Soft
Soft
Medium Stiff
Stiff
Very Stiff
Hard

N-Value (uncorrected)

< 2
2 - 4
5 - 8
9 - 15
16 - 30
> 30

Apparent Density of Coarse Grained Soils

Relative Density

Very Loose
Loose
Medium Dense
Dense
Very Dense

N-Value (uncorrected)

< 4
4 - 10
11 - 30
31 - 50
> 50

Moisture Condition

Dry
Moist
Wet

- Absence of moisture, dusty, dry to the touch.
- Damp, but no visible water.
- Visible free water.

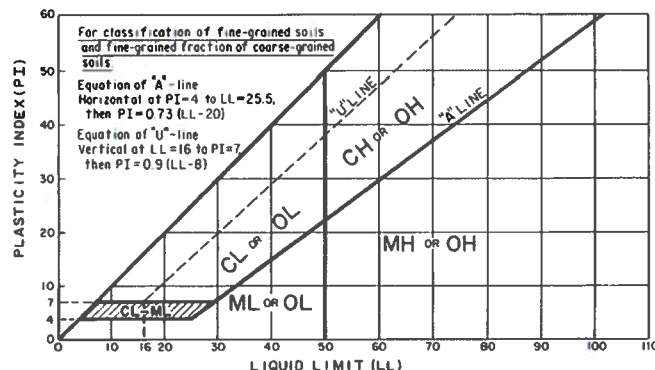
Definition of Particle Size Ranges

Soil Component

Size Range

Boulder > 12 in (300 mm)
Cobble 3 in (75 mm) - 12 in (300 mm)
Gravel No. 4 Sieve (4.75 mm) to 3 in (75 mm)
Sand No. 200 (0.075 mm) to No. 4 Sieves (4.75 mm)
Silt < No. 200 Sieve (0.075 mm)*
Clay < No. 200 Sieve (0.075 mm)*

*Use Atterberg limits and chart below to differentiate between silt and clay.



Angularity of Coarse-Grained Particles

- Angular** -Particles have sharp edges and relative plane sides with unpolished surfaces.
- Subangular** -Particles are similar to angular description, but have rounded edges.
- Subrounded** -Particles have nearly plane sides, but have no edges.
- Rounded** -Particles have smoothly curved sides and well-rounded corners and edges.

APPENDIX B

Laboratory Testing Reports



GEOTECHNICAL * MATERIALS * ENVIRONMENTAL * DRILLING * LANDFILLS

LABORATORY TEST DATA SUMMARY SHEET

PROJECT: Expo Tower
 ETL JOB NUMBER: G 6477-25
 PROJECT LOCATION: Leon County
 CLIENT: Ronald Goldsmith
 PROJECT MANAGER: James Werbicki

START DATE: 2/26/2025
 FINISH DATE: Varies
 TECHNICIAN(S): JC
 DATE SAMPLED: Varies

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	Bottom				LL	PL	PI					Dry Unit Weight (pcf)	Moisture Content (%)	Compressive Strength (ks)	Failure Strain (%)	Confining Pressure (psi)	Dry Unit Weight (pcf)	Moisture Content (%)
B-4	3.0	5.0		Brown w/ Or. Brown	SM Silty Sand	NP	NP	NP	18	48	6	4							
B-4	5.0	7.0		Grayish Br with Red Br & Gr	SC Clayey sand	29	13	16	18	41	0	0							
B-4	8.0	10.0		Gray w/ Dk. Red Brown	SC Clayey sand	36	17	19	20	37	0	0							
B-4	13.0	15.0		Lt. Gray	SM Silty Sand	NP	NP	NP	29	19	0	0							
B-4	23.0	25.0		Brown w/ Dk. Brown	CH Fat Clay with Sand	52	25	27	28	81	5	0							
B-1	1.0	3.0		Gray w/ Red Brown	CH Sandy Fat Clay	53	16	37	27	63	3	1							
B-1	5.0	7.0		Orangish Brown w/ Tan	SM Silty Sand	NP	NP	NP	17	25	1	0							
B-1	18.0	20.0		Gray w/ Or. Brown	SM Silty Sand	NP	NP	NP	26	19	0	0							
B-2	3.0	4.5		Gray w/ Dk. Red Brown	CH Fat Clay with Sand	56	16	40	21	75	0	0							
B-2	8.0	10.0		Or. Br w/ Gr & Dk. Red Br	CL Lean Clay with Sand	46	17	29	23	74	0	0							
B-3	3.0	5.0		Gray w/ Dk. Red Brown	CH Fat Clay with Sand	72	19	53	25	72	0	0							
B-3	5.0	7.0		Red Brown w/ Gray	SC Clayey sand	51	15	36	19	49	0	0							
B-3	13.0	15.0		Lt. Red Brown w/ Gray	SM Silty Sand	NP	NP	NP	26	25	2	1							

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

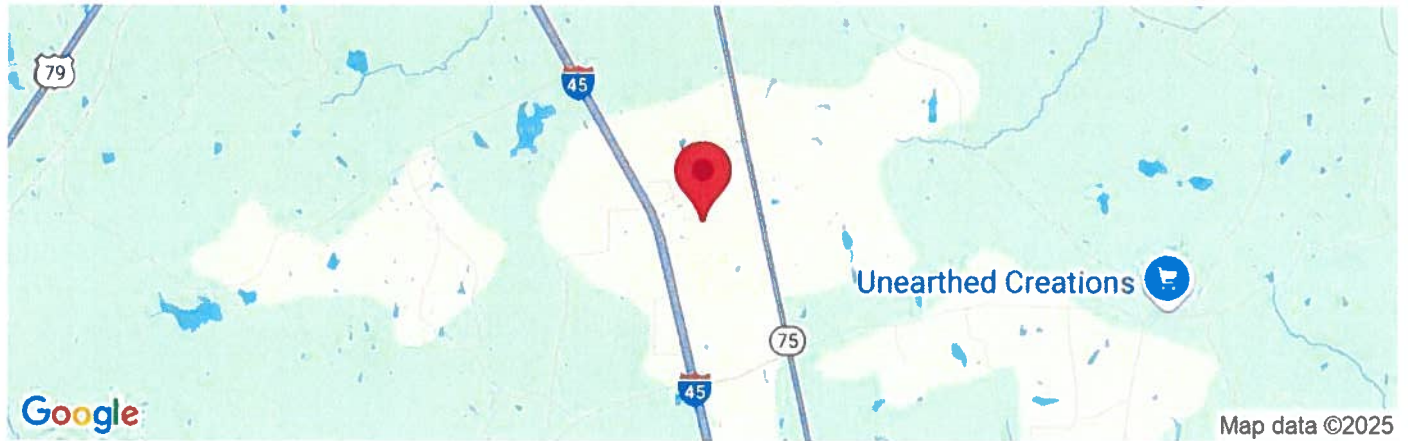
APPENDIX C

USGS Seismic Design Report



Leon County EXPO Tower

Latitude, Longitude: 31.378947, -96.013985



Date	3/14/2025, 9:12:57 AM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S _S	0.074	MCE _R ground motion. (for 0.2 second period)
S ₁	0.046	MCE _R ground motion. (for 1.0s period)
S _{MS}	0.119	Site-modified spectral acceleration value
S _{M1}	0.111	Site-modified spectral acceleration value
S _{DS}	0.079	Numeric seismic design value at 0.2 second SA
S _{D1}	0.074	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	B	Seismic design category
F _a	1.6	Site amplification factor at 0.2 second
F _v	2.4	Site amplification factor at 1.0 second
PGA	0.036	MCE _G peak ground acceleration
F _{PGA}	1.6	Site amplification factor at PGA
PGA _M	0.057	Site modified peak ground acceleration
T _L	12	Long-period transition period in seconds
SsRT	0.074	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.078	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.046	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.052	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)

Type	Value	Description
PGA_{UH}	0.036	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.948	Mapped value of the risk coefficient at short periods
C_{R1}	0.883	Mapped value of the risk coefficient at a period of 1 s
C_V	0.7	Vertical coefficient

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520 S. Main Street, Suite 2531
Akron, Ohio 44311

Phone 330.572.2100
www.gpdgroup.com

December 3, 2024

Statement of Compliance

GPD Group Inc., (GPD) has performed a NEPA Report dated December 3, 2024, for the site- Leon County Texas- EXPO.

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.

A handwritten signature in dark ink that reads "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.
 2024-ASW-13360-OE

Issued Date: 10/21/2024

Ronald R Goldsmith
 Ronald 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: Tower EXPO Tower
 Location: Buffalo, TX
 Latitude: 31-22-43.72N NAD 83
 Longitude: 96-00-49.51W
 Heights: 429 feet site elevation (SE)
 425 feet above ground level (AGL)
 854 feet above mean sea level (AMSL)

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, 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 Air Missions (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)
- Within 5 days after the construction reaches its greatest height (7460-2, Part 2)

See attachment for additional condition(s) or information.

While the structure does not constitute a hazard to air navigation, it would be located within or near a military training area and/or route.

This determination expires on 04/21/2026 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 (817) 222-5922, or debbie.cardenas@faa.gov. On any future correspondence concerning this matter, please refer to Aeronautical Study Number 2024-ASW-13360-OE.

Signature Control No: 633933780-636583686

(DNE)

Debbie Cardenas
Technician

Attachment(s)
Additional Information
Frequency Data

Map(s)

cc: FCC

Additional information for ASN 2024-ASW-13360-OE

Please contact Mr. Salvador Ordorica, Randolph AFB Airspace Manager at 210-652-5580, salvador.ordorica.2@us.af.mil, 7 days prior to the start of construction.

Frequency Data for ASN 2024-ASW-13360-OE

LOW FREQUENCY	HIGH FREQUENCY	FREQUENCY UNIT	ERP	ERP UNIT
6	7	GHz	55	dBW
6	7	GHz	42	dBW
10	11.7	GHz	55	dBW
10	11.7	GHz	42	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	901	MHz	500	W
806	824	MHz	500	W

