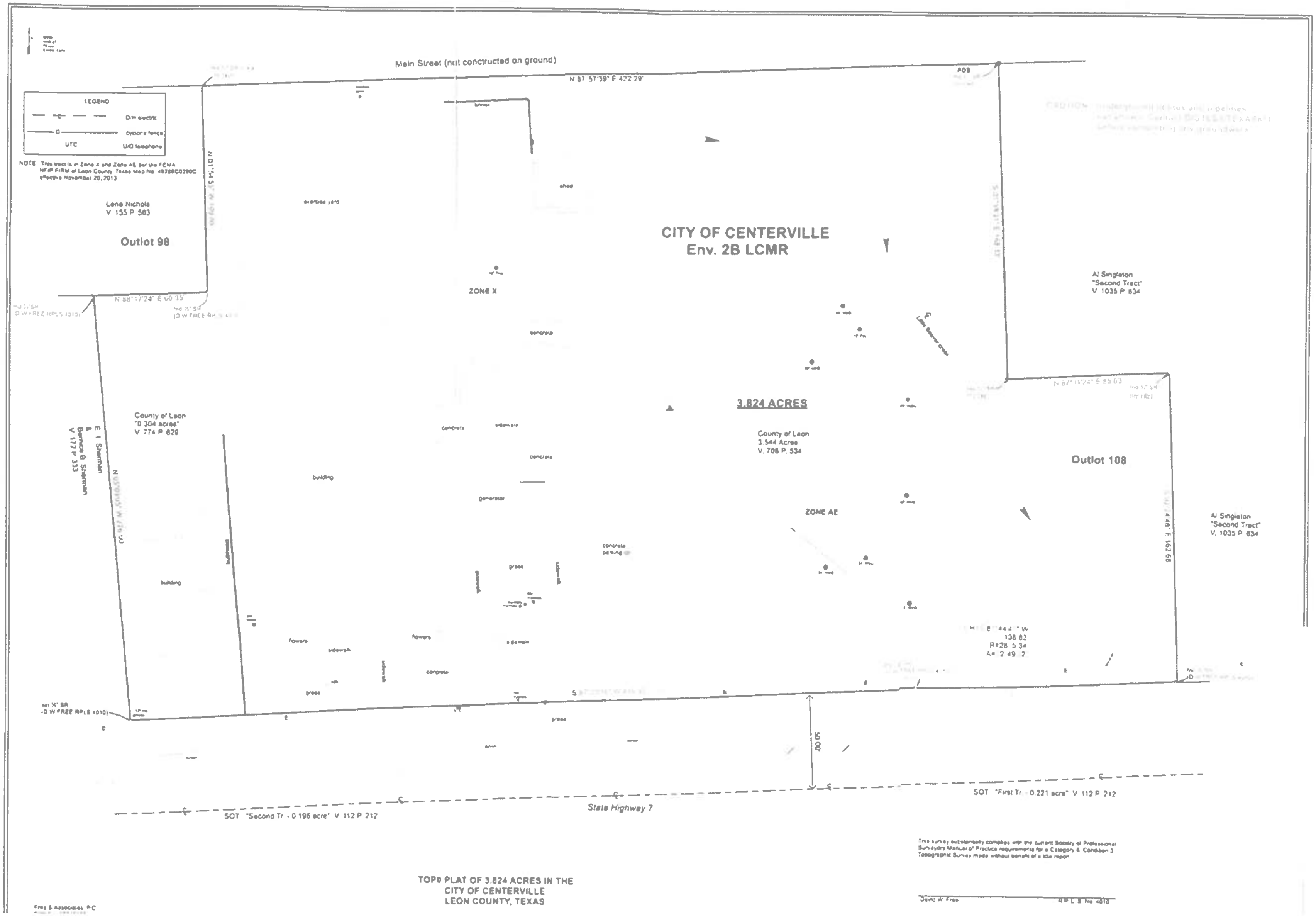
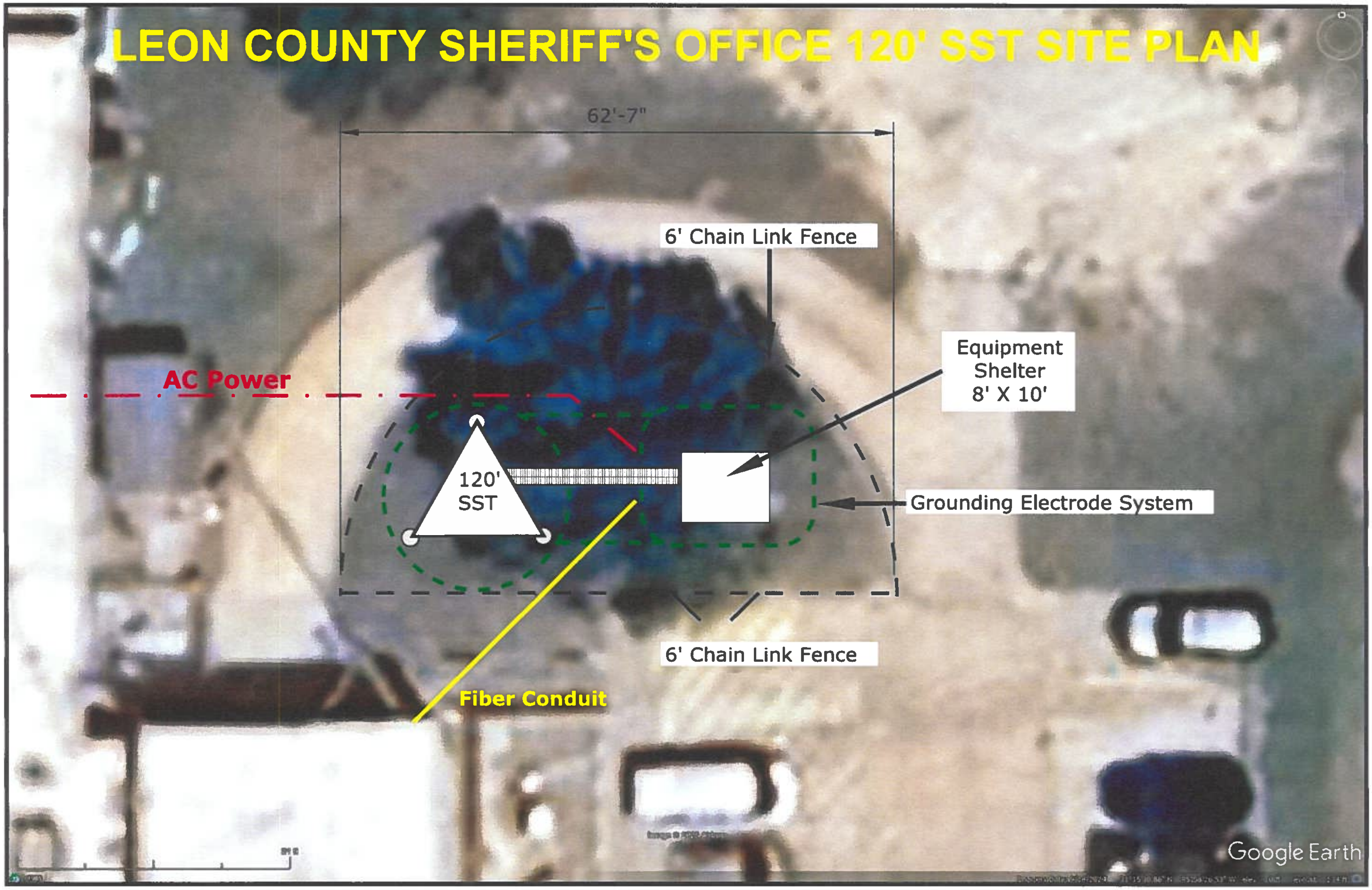

Main Street (not conctructed on ground)
N 87 57'39" E 422.29'
POB
LEGEND
O/H electric
cyclone fence
UTC
U/G telephone
NOTE This tract is in Zone X and Zone AE per the FEMA NFIP FIRM of Leon County Texas Map No. 48289C0390C effective November 20, 2013
Lena Nichols
V. 155 P. 563
Outlot 98
CITY OF CENTERVILLE
Env. 2B LCMR
ZONE X
Al Singleton
"Second Tract"
V. 1035 P. 834
3.824 ACRES
County of Leon
3.544 Acres
V. 708 P. 534
County of Leon
"0.304 acres"
V. 774 P. 829
E. I. Sherman
&
Bernice B. Sherman
V. 172 P. 333
Outlot 108
ZONE AE
Al Singleton
"Second Tract"
V. 1035 P. 834
concrete
building
generator
concrete parking
grass
sidewalk
flowers
shed
50.00'
SOT "First Tr. - 0.221 acre" V. 112 P. 212
SOT "Second Tr. - 0.196 acre" V. 112 P. 212
State Highway 7
TOPO PLAT OF 3.824 ACRES IN THE
CITY OF CENTERVILLE
LEON COUNTY, TEXAS
Free & Associates, P.C.


LEON COUNTY SHERIFF'S OFFICE 120' SST SITE PLAN
62'-7"
6' Chain Link Fence
Equipment
Shelter
8' X 10'
AC Power
120'
SST
Grounding Electrode System
6' Chain Link Fence
Fiber Conduit
Google Earth

March 17, 2025

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

SUBJECT: Leon County – Sheriff's Office Radio Tower
Centerville, Texas
Geotechnical Investigation
ETTL Job No. G 6478-25

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 further assist 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

James Werbiski, E.I.T.
Project Manager

Owen Sanderson

Owen B. Sanderson, P.E.
Senior Engineer

Owen B. Sanderson

Digitally signed by: Owen B. Sanderson
DN: CN = Owen B. Sanderson email = osanderson@ettlinc.com C = US O = ETTL OU = Geo.02
Date: 2025.03.18 10:39:39 -05'00'

STATE OF TEXAS
OWEN B. SANDERSON
120504
LICENSED
PROFESSIONAL ENGINEER

03/17/2025

Distribution: (PDF) Mission Critical Partners, LLC

Geotechnical Investigation

Leon County Sheriff's Office Radio Tower

Centerville, Texas

Submitted To:

TJ Foley
Leon County
Centerville, Texas

Prepared by;

ETTL Engineers & Consultants Inc.
Tyler, Texas

March 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 January 9th, 2025. The field operations were conducted on February 24^{th}, 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 Sheriff's Office in Centerville, 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. ETTL 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 (120ft tall) Self-Supporting (3 legs)

Drilled piers are anticipated for this project. 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 a grass-covered island with a single tree in the vicinity of the proposed tower. The general topography of the site is relatively flat.

4.0 FIELD OPERATIONS

The subsurface conditions at the site were defined by four (1) 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 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.

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. Upon completion of the drilling activities, the open boreholes were measured for groundwater. The phreatic surface is estimated to be at 15 feet BEG.

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 borehole from rising to the phreatic level during the time period of observation.

- A given boring 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 boring 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 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	6
Dry Sieve Analysis (% Passing No. 40)	ASTM D 6913	6
Washed Sieve Analysis (% Passing No. 200)	ASTM D 1140	6
Atterberg Limits (Liquid & Plastic Limits)	ASTM D 4318	6
Moisture Content by Dry Weight	ASTM D 2216	6
Unit Weight	ASTM D 7263	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, Tyler Sheet, the proposed site is located in 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. The 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 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		
Layer	**Layer Descriptions**	**Depth (ft)**
1	Interbedded layers of loose to medium-dense sands & clayey sands	0 – 23
2	Very dense sands	23 – 28
3	Sandy lean clay Hard	28 – 30

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.

6.3.1 Determining Representative Properties

There is insufficient data (i.e., less than 30 data points for a given parameter for a given soil layer) to warrant a rigorous statistical analysis. Experience has also shown that the average (i.e., best fit to the scattered data) can be unconservative for soils that are not homogeneous (e.g., randomly variable degrees of sand content). We have adopted what we call a P25/P75 approach (as originally promulgated by George Sowers) as an appropriate means for dealing with random variation in soil masses. The average of all applicable test results averaged with the lowest value is termed the "P25" value. The average of all applicable test results with the highest applicable value is termed the "P75" value. Rather than use the worst-case situation when sufficient data are available, we have used either the P25 value (when a low result would be conservative) or the P75 value (when a high result would be conservative) to predict parameters that are used to quantify the behavior of the soil mass. This procedure is only used when the variation in the data is anticipated to be spatially random. If there is a discernible pattern to the variation of the data (e.g., shear strength tends to be softer in low areas) then the data are grouped in accordance with the pattern prior to applying the method stated above (i.e., data are only averaged within groups).

Listed in **TABLE 6.3.1,** are the soil strata with the predicted P25 and the P75 (as appropriate) engineering properties selected to be applicable throughout the project. Note that properties in isolated situations may be adjusted more favorably when considering the specifics of the situation (contact ETTL for further information, if desired). These properties are derived from our testing of the soils as well as our experience with the soils in question together with published correlations.

TABLE 6.3.1 – Predicted Soil Engineering Properties						
Stratum / Material	Drained / Undrained Peak Strength Values				5Soil Type (For L-Pile Analysis)	Soil Class
	Moist Unit Wt. (pcf)1	2Angle of Internal Friction Φ	3Confined Modulus (Mt) (ksf)	4UU/UC Cohesion c (psf)		
Select Fill Compacted to 98% D698	125	30	250	1300	Stiff Clay w/o free Water (Reese	SC / Sandy CL
1	115	27	200	-	Sand (Reese)	SM
2	120	32	300	-	Sand (Reese)	SM
3	125	29	250	2000	Stiff Clay w/o free Water (Reese)	SC / Sandy CL/CH

Notes:

1) Buoyant unit weight = Moist Unit Wt. minus 62.4 pcf, where applicable. See boring logs.
2) Estimated drained friction angle. For sands drained cohesion, c' = 0. For pure clay soils drained cohesion is not predicted.
3) Undrained confined modulus (Mt) (ksf) derived from the stress-strain curve of the U.U. Triaxial test and/or correlations with SPT N-Values.
4) Unconsolidated/Undrained shear strength (psf), measured by U.U. triaxial test and/or correlations with SPT sampling.
5) Use default L-Pile values for K and e50.

6.4 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** (assumed). 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 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

Due to the geologic condition of shallow groundwater, the typically augured drilled shaft may be problematic. Drilled shafts will penetrate below the predicted groundwater table, and temporary surface casing, and/or slurry drilling techniques may be required to keep the hole from caving. (See **Section 7.2.9** below for more details.)

7.2 Drilled Piers

Drilled shafts have the advantage of being single elements that can provide both large vertical and large lateral capacity. Drilled shafts will consist of cylindrical excavations that are filled with high-slump concrete that is reinforced with a steel cage. Steel should be adequate to resist uplift loads and bending moments and shears from lateral loads. The reinforcing cage should be fitted with heavy-duty spacers (e.g., "ShaftSpacer" by Foundation Technologies - light plastic wheels are unacceptable) to maintain clearance between the steel cage and the side of the hole. Steel spacers are also unacceptable due to the corrosion potential increase. Only straight shafts are considered since under-reaming in hard clays, may prove problematic.

Sizes for which static vertical load design information is provided herein include 18" to 72", which are believed sufficient to adequately cover the anticipated loading ranges. Contact ETTL for design curves for additional shaft sizes, if needed. In general, the loads suggested as "allowable" can be increased by 33% for transient loads such as wind and seismic (except for cases where these loadings result in a net uplift on a given shaft). The information provided in this report is based on that found in Drilled Shafts: <u>Construction Procedures and LRFD Design Methods – FHWA GEC 010</u> - Federal Highway Administration, 2010.

7.2.1 Shaft Embedment Considerations

In general, shafts that are smaller in diameter and deeper are more economical than those which are larger in diameter and shallower since the volume of concrete is less for the former and capacity in the deeper soils is often much greater than in shallower soils. *The minimum recommended embedment is determined not only by load capacity (vertical and uplift) but by the* *depth necessary to resist heave as discussed below and must be deeper than the active zone which is predicted to be about 10' below the final grade.*

7.2.2 Vertical Capacity

Drilled piers mobilize both skin friction and end bearing to distribute the loads from the proposed structures to the subsoil. The amount of movement it takes to develop full ultimate skin friction is generally less than 0.5 inch, whereas the amount of movement necessary to develop ultimate end bearing is on the order of 3% to 5% of the tip diameter (in sands capacity is even available at tip movements in excess of 5%). To limit the settlement of the shaft to a generally accepted magnitude, the amount of end bearing that can be mobilized is limited (more so for larger shafts than for smaller ones). That is, a calculation of the "effective" ultimate capacity or the mobilized ultimate capacity at a limited settlement, involves adding the full ultimate skin friction capacity to a reduced (in some circumstances) ultimate end bearing.

Because of the myriad possible combinations of sizes and loading conditions and the unknown constrictions at any given location, capacity curves are provided which can be used to select size and embedment for individual, isolated shafts in native soil. It is usually the case that smaller diameter and deeper shafts are more economical than bigger diameter and shallower shafts.

Capacity curves titled "**DRILLED SHAFT CAPACITIES**" for the proposed structure are included in **APPENDIX D**. There is an individual plot for each of the shaft sizes selected showing recommended allowable (FS noted on the curve) skin friction (so indicated in the legend of the plot by a dashed line) and total load (indicated by the solid line curve labeled in the legend with the shaft diameter).

The vertical capacity read from the applicable curve represents the "effective ultimate" (i.e., total ultimate reduced to limit predicted tip settlement at ultimate load to 1" or less) divided by a safety factor (as noted). For drilled shafts, the safety factor of 3 is recommended because load testing is not routinely conducted to confirm design assumptions. *It should be noted that this capacity only represents the geotechnical capacity of the shaft. The designer needs to check whether other issues such as concrete strength may limit the capacity to something less than the geotechnical capacity.* If design tip elevations are significantly greater than the limit of exploration, additional exploration should be conducted to confirm the capacities assumed on the curves (where provided curves have been extended by others beyond the depth of exploration).

Note that the embedment from the curve represents depth below the existing ground elevation. Where fill will be placed above the existing grade at a given shaft location, add the thickness of the fill to the embedment depth determined from the procedure set forth herein to determine the preliminary required embedment depth below-finished grade (This is a conservative approach, especially where the fill thickness is not significant and it is often the case that the embedment from the curve can be used as the depth below finished grade). The elevation below the finished ground surface read from the appropriate design curve can be conservatively used as the required embedment length below the base of any pier cap. Alternatively, the capacity of a given pier can be determined by subtracting from the capacity of the shaft as determined by the procedure set out in this section (below), the capacity from the skin friction curve at the depth of the pier cap.

Where a shaft is subject to significant lateral load, the skin friction capacity of that portion of the top of the shaft that deflects laterally more than 1% of shaft diameter should be neglected (may be the case, especially for smaller diameter shafts with significant lateral load). Information regarding the depth to be ignored can be readily obtained from the lateral analysis curves derived from an L-Pile analysis (not a part of this study). The appropriate **DRILLED SHAFT CAPACITIES** curve should be examined to determine the skin friction capacity at the depth where deflection determined in the L-Pile analysis is equal to 1% of shaft diameter and this value should be subtracted from the capacity at the embedment depth to determine the design capacity for the shaft.

Limiting working loads to the level indicated by the curves should limit the settlement of isolated piers at working load (not the settlement at ultimate load) to something in the neighborhood of 0.5" or less. However, the settlement considered by the design curves is the tip settlement of the isolated pier, not the head. You will need to check the elastic compression on the pier (use say 67% of its actual length for "L" in the (P*L)/(A*E) formula for the approximate computation of elastic compression to see if it is a significant amount). As a rule of thumb, it should only be significant for very slender piers. The settlement of pier groups can be significantly greater than the settlement predicted for an isolated pier.

7.2.2.1 Modification for Significant Lateral Loads

Where a shaft is subject to significant lateral load, the skin friction capacity of that portion of the top of the shaft that deflects laterally more than 1% of shaft diameter should be neglected (may be the case, especially for smaller diameter shafts). Information regarding the depth to be ignored can be readily obtained from the lateral analysis curves generated in an L-Pile analysis. The appropriate **CAPACITIES** curve should be examined to determine the skin friction capacity at the depth below the top of the pile where deflection determined in an L-Pile analysis is equal to 1% of shaft diameter and this value should be subtracted from the capacity at the embedment depth to determine the recommended design capacity for the shaft.

7.2.3 Settlement of Pier Shafts

Limiting working loads to the level indicated by the curves should limit the settlement of isolated piers not subject to drag load effects at working load (not the settlement at ultimate load) to something in the neighborhood of 0.5" or less. However, the settlement considered by the design curves is the tip settlement of the isolated pier, not the head. The designer will need to check the elastic compression on the pier (use say 67% of its actual length for "L" in the (P*L)/(A*E) formula for the approximate computation of elastic compression to see if it is a significant amount). As a rule of thumb, it should only be significant for very slender piers. The settlement of pier groups can be significantly greater than the settlement predicted for an isolated pier and, thus, requires a separate evaluation, outside the scope of this investigation.

7.2.3.1 Soil Induced Uplift Loads

The surficial soils within the moisture zone of influence are predicted to result in negligible heave induced with moisture infiltration. Tensile loads are not predicted to impact the design of the drilled shafts.

7.2.4 Group Effects

7.2.4.1 Cohesive Profiles (SC, CL, CH, MH)

Pier groups in purely cohesive soil profiles should be checked as follows (*AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS,* Section 10.8.3.6): If the pile cap is in firm contact with firm native soil or properly compacted select fill, the group of piers is assumed to act as an equivalent shaft with a perimeter equal to the outside perimeter of the group and an end area equal to the area encompassed by the perimeter of the group. To compute the average allowable unit skin friction resistance on the peripheral surface of the group, read the values from the skin friction curve at the embedment depth selected (which is the allowable skin friction (Ultimate/SF) for the individual pier) and divide it by the surface area of the pier (perimeter of the pier times the embedment depth). Multiply this value times a factor of 1.8 to convert pier skin friction to group skin friction (The factor of 1.8 is a conservative value to convert adhesion of cohesive soil to piers to the average cohesion of the soil over the embedment depth, which is what controls in the analysis of skin friction of the equivalent pier (or soil block). Multiply this value times the perimeter of the group times the embedment depth of the piers to predict the total recommended allowable side shear for the group.

To compute unit recommended allowable end bearing for the group, subtract the total allowable capacity read from the design curve labeled as "72" (i.e., the shaft diameter in inches as indicated in the legend) at the embedment depth from the skin friction capacity at the same depth and divide the result by the end area of the "72" pier (19.6 sq ft). Multiply this value times the plan area of the group to predict the total recommended allowable end bearing for the group. Add the total group end bearing to the total group side shear to arrive at the predicted total allowable group capacity.

The capacity (tension or compression) of the group (equivalent shaft) may not exceed the sum of the individual capacities of the shafts comprising the group. A pier cap (encompassing the entire group) in contact with the ground is required for each shaft to develop full theoretical resistance.

*Minimum recommended center-to-center spacing of shafts in a group is two shaft diameters (2D). Such close spacing, however, can significantly impact group lateral load capacity (see Section **7.2.6.1**).*

7.2.4.2 Group Settlement

Settlement of the group should also be evaluated as it can be significantly more than what would be anticipated for a single shaft. Generally, *but not always*, if the perimeter of the pile group equals or exceeds the sum of the perimeters of the individual shafts, there should not need to be any reduction in vertical capacity for group action. Evaluation of group settlement was outside the scope of this investigation, but ETTL can assist in these evaluations if provided with specifics regarding configurations and loads.

7.2.5 Uplift

In this instance, a value of 70% is recommended to calculate the capacity *of an individual shaft* in uplift as a percentage of downward skin friction capacity. Read the skin friction curve from the chart at the embedment depth (which is allowable skin friction (Ultimate/SF noted)). Multiply this value by 0.7. The resistance value calculated using this skin friction value is compared to the uplift load applied to the top of the shaft minus the shaft weight and, as long as the resistance is greater, the predicted factor of safety against uplift failure is equal to the safety factor noted on the curve combined with any factors by which the load has been modified.

7.2.6 Lateral Load

A lateral load analysis depends on soil properties as well as the stiffness of the drilled shaft being analyzed and, so, entails a cooperative process involving both the structural engineer and the geotechnical engineer. Because of the myriad possible combinations of sizes and loading conditions and the unknown constrictions at any given location, the L-Pile analysis has not been performed at this time. ETTL can assist with lateral load analysis once details regarding shaft diameter, reinforcing, head fixity, and lateral load have been preliminarily determined and if such conditions are deemed to warrant more detailed analysis.

Programs such as the L-Pile program by Ensoft calculate the stiffness of drilled shafts accounting for reinforcement as well as cracking (i.e., stiffness reduction) for each combination of loads. Soil parameter values that should be used in a lateral load analysis are listed in **TABLE 6.3.1**, **Soil Properties**, above. For piers embedded in fat clay (if any) that is exposed to drying action (e.g. piers at the edges of pier caps), we recommend that the portion of the shaft that is 5' or shallower below the finished ground surface adjacent to the cap be neglected for lateral support in order to help account for possible shrinkage of the clay away from the sides of the shaft in the upper zone.

Analyses of both the fixed head and free head conditions can be made. The analysis also depends on the percentage of steel reinforcement, as well as the magnitude of the vertical load to use in conjunction with the maximum horizontal load. The critical combination of loads yielding the maximum horizontal deflection consists of the maximum horizontal load together with the minimum vertical load. This combination results in the severest moment and least effective moment of inertia (due to the cracking of the section). The L-Pile program considers soil stiffness as well as the shaft stiffness under whatever combination of vertical load, lateral load, and the moment the user specifies.

ETTL can assist with lateral load analysis once details regarding shaft diameter, reinforcing, head fixity, and lateral load have been preliminarily determined and if such conditions are deemed to warrant more detailed analysis.

7.2.6.1 Group Action of Laterally Loaded Piers

A group of piles or piers loaded laterally will generally have a total lateral capacity (for a given lateral deflection of the pile heads) less than the sum of the individual lateral capacities based on an isolated pile. This is due to the fact that a pile moving toward another pile that is also moving experiences reduced resistance because of the relaxation of the soil behind the leading pile as the leading pile moves away from the soil behind it. This reduction is accounted for in LPile or GROUP software analyses by incorporating a *p-multiplier* applied to each of the piles in the group based on center-to-center spacing and group configuration as well as direction of loading. *FHWA GEC 010,* Section 11.5.1 provides suggested multipliers as well as a methodology for use in this analysis.

7.2.7 Lateral Load Resistance of Pier Caps

Resistance to lateral loads can be developed via a combination of passive earth pressure acting against the face of footings and pile/pier caps and lateral resistance developed by deep foundations. The resistance of piers to lateral loads is discussed elsewhere in this report. A portion of ultimate passive earth pressure can be applied to the face(s) of footings and pier caps to resist lateral loads. ***Caution****: Lateral resistance against the vertical face of pier caps or spread footings 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 (which needs to be nearly vertical and extended to the bottom of cap elevation) and the footing edge is placed under density-controlled conditions (backfill should be placed to 100% ASTM D698).*

In determining the total resistance to lateral loads, the degree of lateral movement that can be tolerated must be considered. This is related to the fact that there is a direct relationship between lateral load and horizontal deflection for piers and there is also a direct relationship between lateral movement and the degree of passive resistance that can be mobilized. The magnitude of lateral movement needs to be consistent for each contributing element in computing total allowable resistance. Lateral support to the cap afforded by a floor slab placed in contact with the cap should also be considered.

Ultimate passive resistance can be approximated as a triangular pressure distribution utilizing an equivalent fluid weight of 200 pounds per cubic foot. For a lateral deflection limitation of about 0.5" the amount of passive resistance that can be mobilized for pile cap or footing thicknesses in the range of 3' to 10' can be taken as 45% of the ultimate resistance. If the deflection of the supporting piles is less than 0.5" the mobilized passive pressure must be reduced accordingly. (This relationship is rather complex and ETTL can provide further assistance when provided with specifics of a given situation). To determine the total allowable lateral load for the given lateral deflection limitation, the mobilized passive pressure should be added to the resistance from the individual piles (modified for group effects as detailed elsewhere in the report), and the sum reduced by an appropriate safety factor (typically 1.5).

Passive resistance should only be counted upon provided that there will be no excavation within a distance from the edge of the footing or pile cap equal to 1.6 times the depth of the base of the footing. Such excavation would disturb the strength of the passive wedge. If the lateral loads are primarily due to intermittent loads such as wind or seismic, then excavation adjacent to the footing might be allowed (based on the ability of the piles to carry any lateral load at the time of the excavation) provided that any soil removed would be replaced at a minimum density of 100% of ASTM D698. *Also, the temporary excavation face needs to be nearly vertical and extended to the bottom of the cap elevation. The portion of the sides of the excavation for the cap 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.*

7.2.8 Drilled Pier Load Testing Program

The information provided herein for the design of piers is based on a factor of safety of 3. If a design based on a lower factor of safety (e.g., 2) is to be considered, a load test program is recommended to more accurately assess capacity. Unless the project entails a large number of piers, this approach would not generally be economical. ETTL can assist you in planning a test program should you desire to pursue this further.

7.2.9 Drilled Pier Construction issues

The construction of all drilled piers should be monitored by personnel familiar with their installation. As a minimum, it is recommended that a representative of this firm be present before and during drilled pier construction in order to monitor test piers and production pier installation procedures. Free water and/or loose material at the base of excavations should be removed, as appropriate, prior to the placement of concrete.

Groundwater observations indicate that shaft tip depth will be below the water table and that a dry auger method of construction may not be feasible. Temporary casing and/or slurry drilling procedures may be required if the shafts do not remain open prior to concreting. If over drilled casing is utilized the skin friction in the cased zone should be evaluated by this firm or ignored. Free water and/or loose material at the base of excavations should be removed with an approved "Muck Bucket" prior to the placement of concrete. Additionally, concrete can be pumped via a tremie pipe placed at the bottom of the shaft to displace any water accumulated in the bottom of the shaft to the surface. At no time should concrete free fall into a shaft that contains water. In any case, it is recommended that contract documents provide alternates with or without casing and dry or slurry displacement construction procedures.

Concrete should be designed and placed with a relatively high slump (7 to 9 inches) to provide solid contact of the shaft with the side of the hole. Close engineering supervision is essential during the installation of the foundation units in order to ensure that construction is performed in accordance with the plans and specifications. Also, to help ensure proper construction of the drilled piers, close coordination between the drilling and concreting operations is considered to be of primary importance. Concrete should be placed at each drilled pier location *immediately* after the completion of drilling. Concrete placement in the shaft should be at a rate of at least 40' of shaft per hour. *In no case should a shaft remain open overnight.*

Construction documents must specify that all foundation units should be constructed in accordance with ACI 336.1 "Standard Specification for the Construction of Drilled Piers," latest edition. Only contractors familiar with and competent in the employment of these methods should be considered for the work. The actual capacity of the completed foundation is directly related to the degree of conformance to correct construction procedures.

8.0 EXCAVATION AND SITE WORK

We are not currently aware of any slabs on grade or flatwork that may be affected by soil heaving related to moisture fluctuations of the expansive clays, see **Section 6.3**. Any structures, flatwork, and/or fencing will be subject to potential movements if the moisture content of the expansive clays were to fluctuate post-construction. If the desire is to reduce these movements to a more tolerant amount, recommendations can be provided upon request.

At a minimum, the subgrade should be prepared per the following.

- Cut out and remove the topsoil and organics.

- Scarify the exposed subgrade to a depth of 12 inches, adjust the moisture content to, and maintain it within a range of optimum Moisture Content to optimum +4% and recompact to a minimum density of 95% of the maximum density defined by ASTM D 698 (Standard Proctor)

- Place select fill to finished slab subgrade. Specifications for the placement of select fill are covered below.

- Testing to verify these procedures is essential to the proper performance of the structure.

8.1 Site Design

The following recommendations are derived from years of experience with structures founded on non-expansive soils and are considered essential to satisfactory structure performance:

- Sidewalks should be sloped away from the building and not tied to the structure.

- The ground surface around the building and the paved areas should be sloped away from the building on all sides so that water will drain away from the structure. Water should not be allowed to pond near the structure during or after rainfall events.

- Adequate drainage should be provided to minimize any increase in moisture content of the foundation soils. Roof drainage should be conveyed by an appropriate means at least 15 feet from the building before it is allowed to drain into the subgrade.

Backfill for utility line ditches should be carefully controlled. It should be placed at a density similar to the surrounding soil. A density of 95 percent of ASTM D 698 (Standard Proctor) may be used as a rule of thumb.

8.2 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).

8.3 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.

9.0 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 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.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX A

Plan of Borings and Boring Logs

ETTL Engineers & Consultants	PREPARED BY	PROJECT	LOCATION	SYMBOL KEY
	ETTL Engineers and Consultants Whitehouse, TX	Name: Leon County - Sheriffs Office Radio Tower Number: G 6478-25	606 East St Mary's St, Centerville, TX 75833, USA Centerville, TX	◎ Soil Boring

Leon County - Sheriffs Office Radio Tower

Centerville, Texas

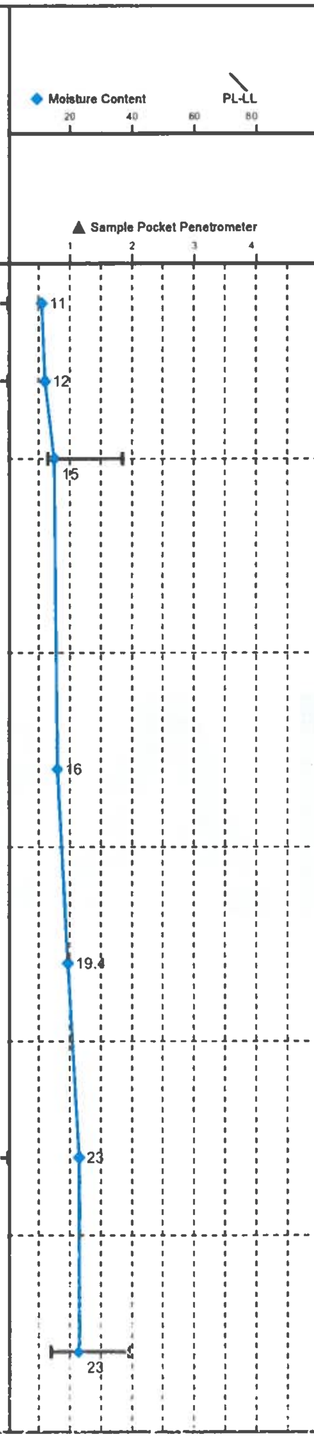
ETTL | Engineers & Consultants

B-1

Drilling Co.:	ETTL Drilling Department	Job #:	G 6478-25
Driller:	Marco	Drilling Date(s):	02/24/2025 - 02/24/2025
Logged By:	James Werbiski	Weather:	-
Rig Type:	CME-55	Surface Elevation:	N/A
Method:	Auger	Coordinates:	31.25852, -95.97415
Boring Depth:	30.0'	Seepage	15'
		Cave-in At Time Of Drilling	N/A
		Stable GW Reading	15'

Remarks:
Hole cave-in observed at final GW reading.

Material Description and Notes	Uncorrected N-Value	Moisture Content (%)	Wet Density (PCF)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	+ #40 Sieve (%)	% Gravel
Silty Sand (SM); orangish brown with gray; moist	6	11.0			NP		20	0	0
	15	12.0			NP		23	0	0
Clayey Sand (SC); reddish brown with dark brown; moist	19	15.0		37	13	24	44	6	2
Silty Sand (SM); brown; very moist	4								
Brown with orangish brown; saturated	9	16.0					41	5	3
Light brown and gray; saturated	13	19.4	124.9						
Dark gray; saturated	72	23.0			NP		25	2	1
Lean Clay (CL); dark gray with gray; very moist	57	23.0	124.9	39	14	25	60	1	0



Boring Log Descriptive Terminology

Key to Soil Symbols and Terms

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	Well-graded gravels, gravel sand mixtures, little or no fines.
				GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	Silty gravels, gravel-sand-silt mixtures.
				GC	Clayey gravels, gravel-sand-clay mixtures.
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	Well-graded sands, gravelly sands, little or no fines.
				SP	Poorly graded sands, gravelly sands, little or no fines.
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	Silty sands, sand-silt mixtures.
				SC	Clayey sands, sand-clay 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.
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, 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

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	N-Value (uncorrected)
Very Soft	< 2
Soft	2 - 4
Medium Stiff	5 - 8
Stiff	9 - 15
Very Stiff	16 - 30
Hard	> 30

Apparent Density of Coarse Grained Soils

Relative Density	N-Value (uncorrected)
Very Loose	< 4
Loose	4 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Moisture Condition

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

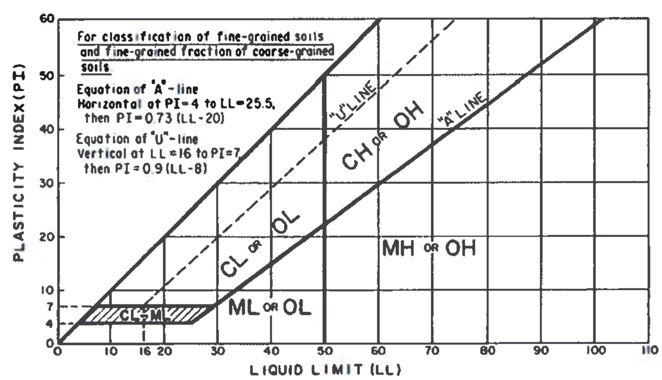
Definition of Particle Size Ranges

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.

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.



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.

Soil Classifications are Based on the Unified Soil Classification System, ASTM D2487 and D2488. 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.

APPENNDIX B

Laboratory Testing Reports

ETTL Engineers & Consultants

GEOTECHNICAL★MATERIALS★ENVIRONMENTAL★DRILLING★LANDFILLS

LABORATORY TEST DATA SUMMARY SHEET

PROJECT: Leon County - Sheriff's Office Radio Tower
ETTL JOB NUMBER: G 6478-25
PROJECT LOCATION: Centerville, Texas
CLIENT: Ronald Goldsmith
PROJECT MANAGER.: James Werbiski

START DATE:
FINISH DATE: 3/1/2025
TECHNICIAN(S): JC
DATE SAMPLED: 2/24/2025

Boring No.	Depth (ft.) Top	Depth (ft.) Bott	Sample No.	Description of Sample	USCS Classification		Atterberg Limits LL	PL	PI	Moisture Content (%)	(%) Passing No. 200 Sieve	(%) Retained No. 40 Sieve	(%) Retained No. 4 Sieve	Unit Weight / Compression Tests: Wet Unit Weight (pcf)	Moisture Content (%)	Compressive Strength (tsf)	Failure Strain (%)	Confining Pressure (psi)	Consol. / Swell Tests Results: Dry Unit Weight (pcf)	Moisture Content (%)	Free Swell (%)	Restraining Pressure (ksf)
B-1	1.0	3.0		Orangish Brown w/ Gray	SM	Silty Sand	NP	NP	NP	11	20	0	0									
B-1	3.0	5.0		Orangish Brown w/ Gray	SM	Silty Sand	NP	NP	NP	12	23	0	0									
B-1	5.0	7.0		Red Br w/ Dk. Brown	SC	Clayey sand	37	13	24	15	44	6	2									
B-1	13.0	15.0		Brown	SM	Silty Sand	NP	NP	NP	16	41	5	3									
B-1	18.0	20.0		Lt. Brown w/ Gray	Visual	Silty Sand								124.9	19.4							
B-1	23.0	25.0		Dk. Gray	SM	Silty Sand	NP	NP	NP	23	25	2	1									
B-1	28.0	30.0		Dk. Gray w/ Gray	CL	Sandy Lean Clay	39	14	25	23	60	1	0	124.9	23							

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

Tyler, TX - Main Office 1717 East Erwin Street Tyler, Texas 75702 Phone: 903-595-4421
Longview, TX 903-758-0402 ★ Arlington, TX 817-962-0048 ★ Austin, TX 512-519-9312 ★ Texarkana, AR 870-772-0013
WWW.ETTLINC.COM

Laboratory Determination of Density (Unit Weight) of Soil Specimens, ASTM D 7263

Method B, Direct Measurement

Project Information

Project: Leon County - Sheriffs Office Radio Tower
Client/Arch./Engr.: Ronald Goldsmith
Project Location: Centerville, Texas
ETTL Job No.: G 6478-25

Sample Information

Boring No: B-1
Sample No.: S-6 Depth (ft): 18.0 - 20.0
Material Origin: Geotechnical Boring
Sampling Info. provided By: James Werbiski
Material Description: Lt. Brown & Gray Silty Sand (Visual)
Sampled By: ETTL Drilling Department Date Sampled: 2/24/2025
Technician: Hunter Franks Test Date: 2/27/2025
Sample Type: Trimmed SPT Sample

Test Data

Minimum of 3 Readings

* N/T = Not Tested

Diameter (in)	Height (in)							
1.387	1.026	Sample Properties						Weight of Water (pcf)
1.342	0.976	Max D698 (pcf)	Opt. M.C. (%)	LL	PL	PI	% .-200	
1.359	1.006			39	14	25	60	62.4
Average Diameter (in)	Average Height (in)	**Sample Moist Weight (g)**	Sample Dry Weight (g).	Area (in^2)	Volume (in^3)	**Saturated Unit Weight (pcf)**	**Moist Unit Weight (pcf)**	**Dry Unit Weight (pcf)**
1.363	1.003	47.99	40.18	1.459	1.46	**127.9**	**124.9**	**104.6**

Phase Volumes:

Solids (V_s)-in^3	0.920	Measured Specific Gravity	2.67	Tare #	220
Voids (V_v)-in^3	0.543	e - Void Ratio	0.590	Tare Wt. (g)	31.55
Water (V_w)-in^3	0.478	η - Total Porosity	37.1%	Wet Wt.(g)	79.46
		Degree of Saturation	88.0%	Dry Wt. (g)	71.66
				ASTM D 2166, Moisture Content	19.4%

From Trimmings

ETTL | Engineers & Consultants

Laboratory Determination of Density (Unit Weight) of Soil Specimens, ASTM D 7263

Method B, Direct Measurement

Project Information

Project: Leon County - Sheriffs Office Radio Tower
Client/Arch./Engr.: Ronald Goldsmith
Project Location: Centerville, Texas
ETTL Job No.: G 6478-25

Sample Information

Boring No: B-1
Sample No.: S-8 Depth (ft): 28.0 - 30.0
Material Origin: Geotechnical Boring
Sampling Info. provided By: James Werbiski
Material Description: Dk. Gray w/ Gray Sandy Lean Clay (CL)
Sampled By: ETTL Drilling Department Date Sampled: 2/24/2025
Technician: Hunter Franks Test Date: 2/27/2025
Sample Type: Trimmed SPT Sample

Test Data

Minimum of 3 Readings

* N/T = Not Tested

Diameter (in)	Height (in)	Max D698 (pcf)	Opt. M.C. (%)	LL	PL	PI	% .-200	Weight of Water (pcf)
1.428	1.816							
1.410	1.810							
1.426	1.790			39	14	25	60	62.4

Average Diameter (in)	Average Height (in)	Sample Moist Weight (g)	Sample Dry Weight (g).	Area (in^2)	Volume (in^3)	Saturated Unit Weight (pcf)	Moist Unit Weight (pcf)	Dry Unit Weight (pcf)
1.421	1.805	93.87	76.32	1.586	2.86	126	124.9	101.6

Phase Volumes:

Solids (V_s)-in^3	1.748
Voids (V_v)-in^3	1.115
Water (V_w)-in^3	1.073

Measured Specific Gravity	2.67
e - Void Ratio	0.638
η - Total Porosity	38.9%
Degree of Saturation	96.2%

Tare #	200
Tare Wt. (g)	29.85
Wet Wt.(g)	123.64
Dry Wt. (g)	106.10
ASTM D 2166, Moisture Content	23.0%

From Trimmings

APPENDIX C

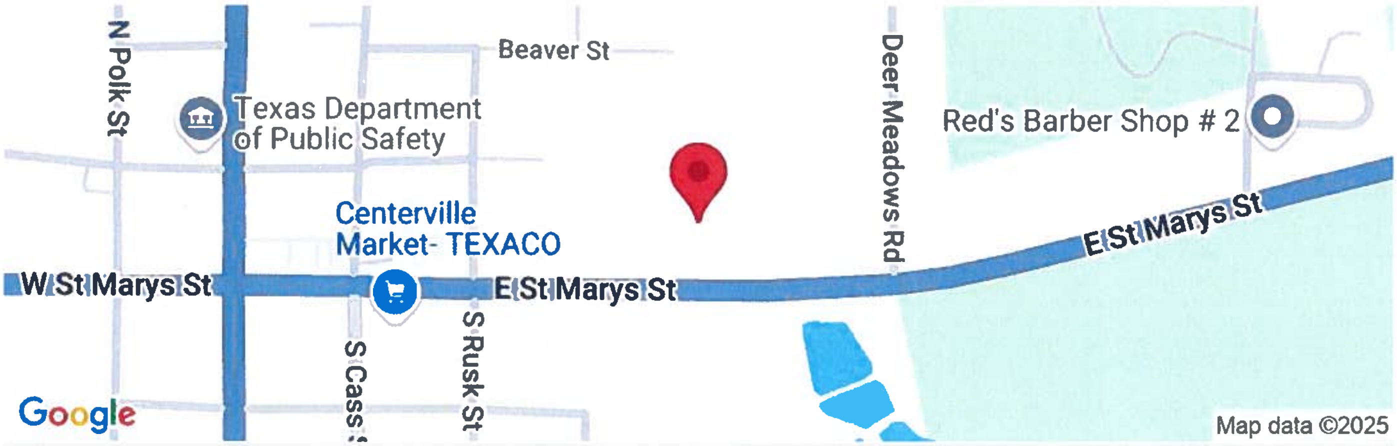
USGS Seismic Design Report

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

OSHPD

Sheriffs Office - Radio Tower

Latitude, Longitude: 31.258524, -95.974146



Date	3/18/2025, 7:38:10 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	0.073	MCE_R ground motion. (for 0.2 second period)
S_1	0.046	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.117	Site-modified spectral acceleration value
S_{M1}	0.11	Site-modified spectral acceleration value
S_{DS}	0.078	Numeric seismic design value at 0.2 second SA
S_{D1}	0.073	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	0.035	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.056	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	0.073	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.077	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}	0.035	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}	0.884	Mapped value of the risk coefficient at a period of 1 s
C_V	0.7	Vertical coefficient

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APPENDIX D

Drilled Shaft Capacity Curves

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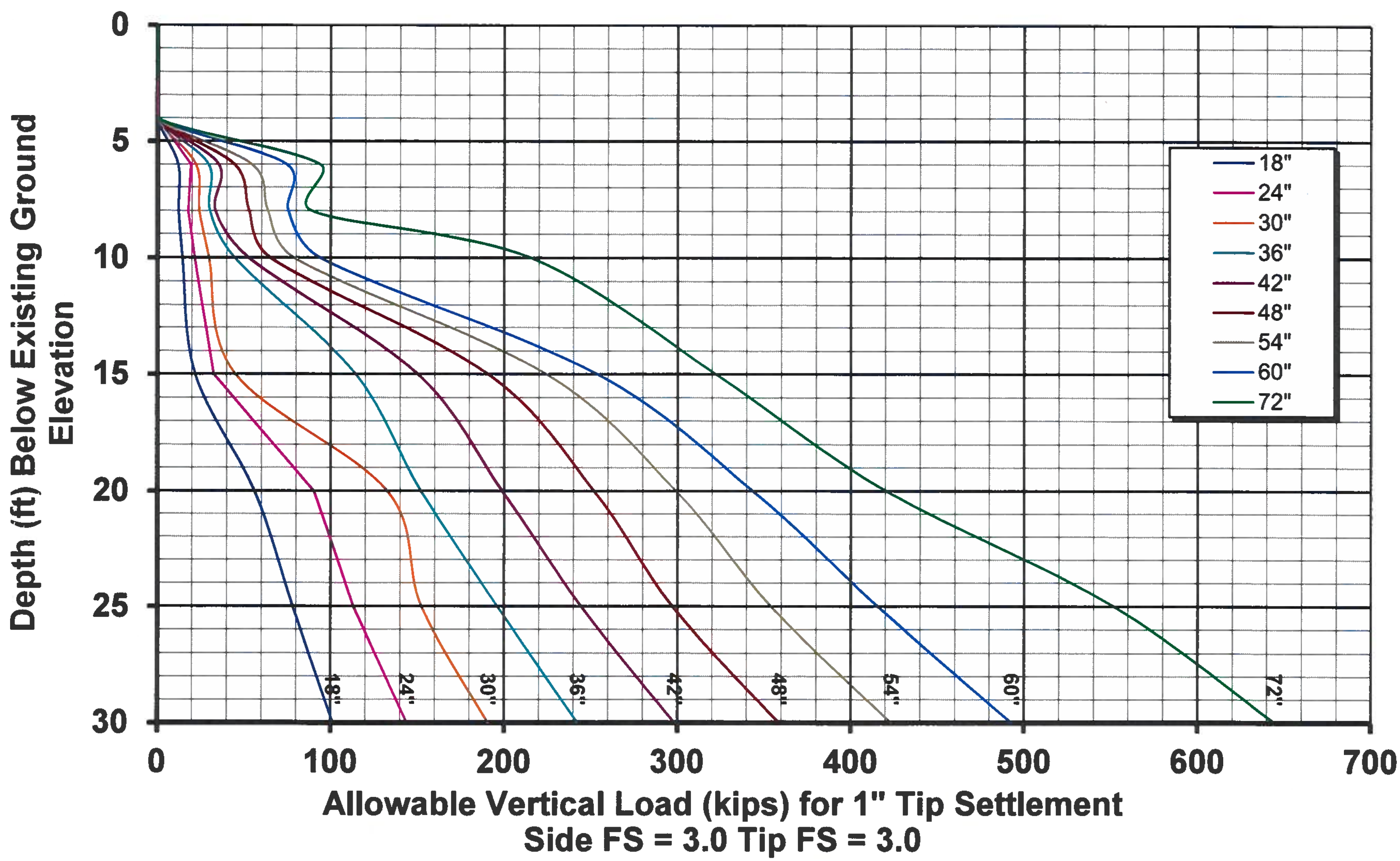
DRILLED SHAFT CAPACITIES



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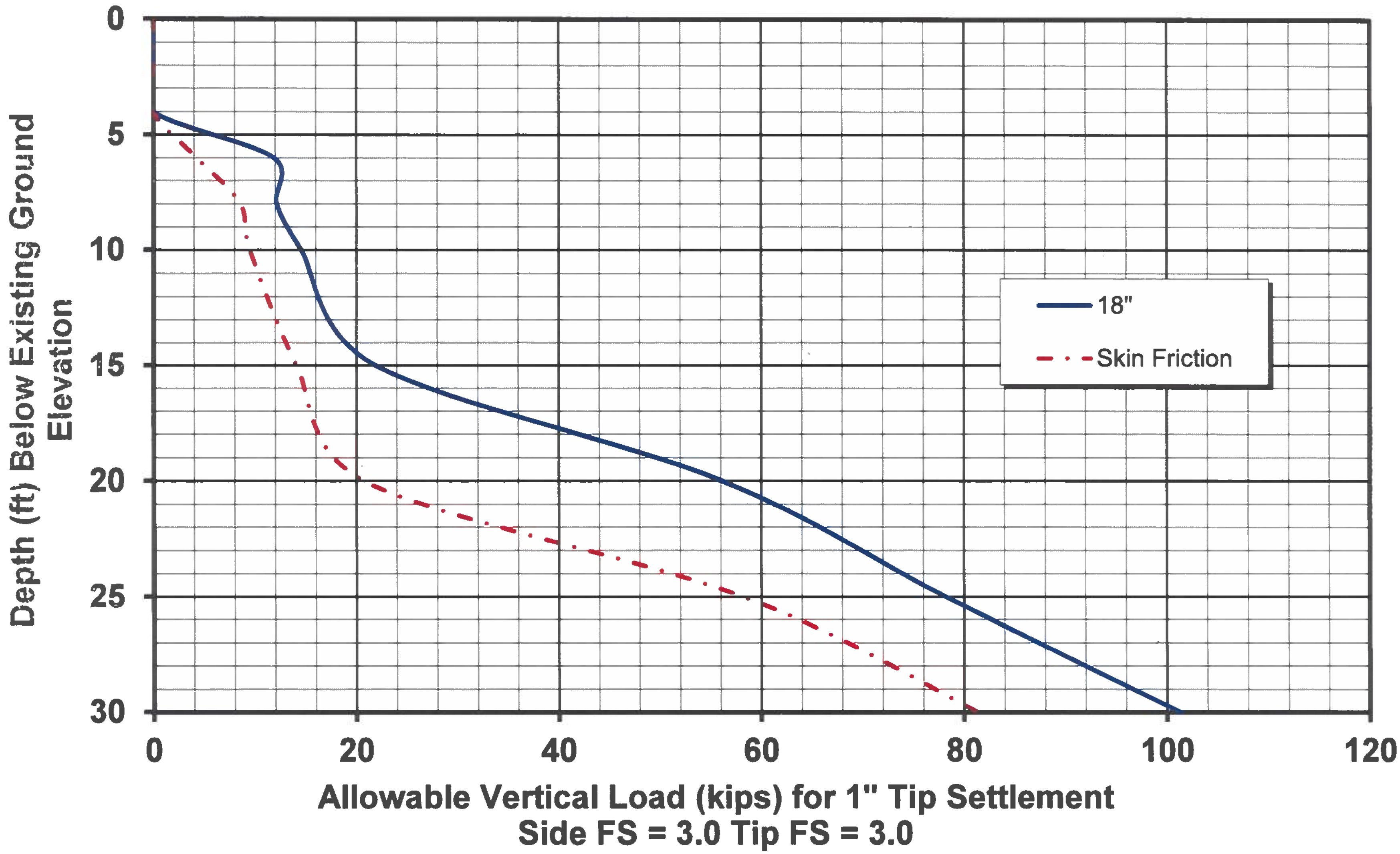
DRILLED SHAFT CAPACITIES



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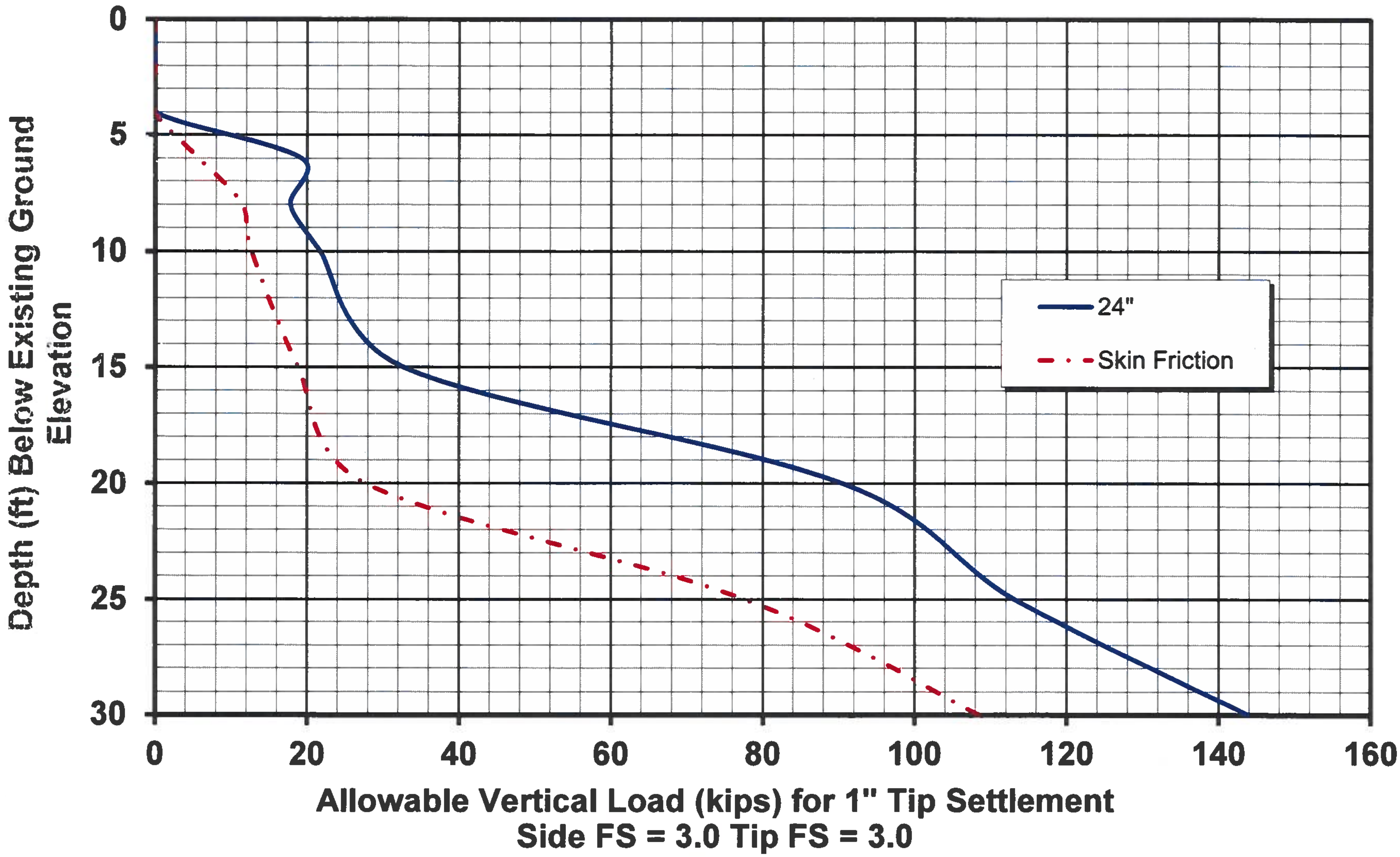
DRILLED SHAFT CAPACITIES



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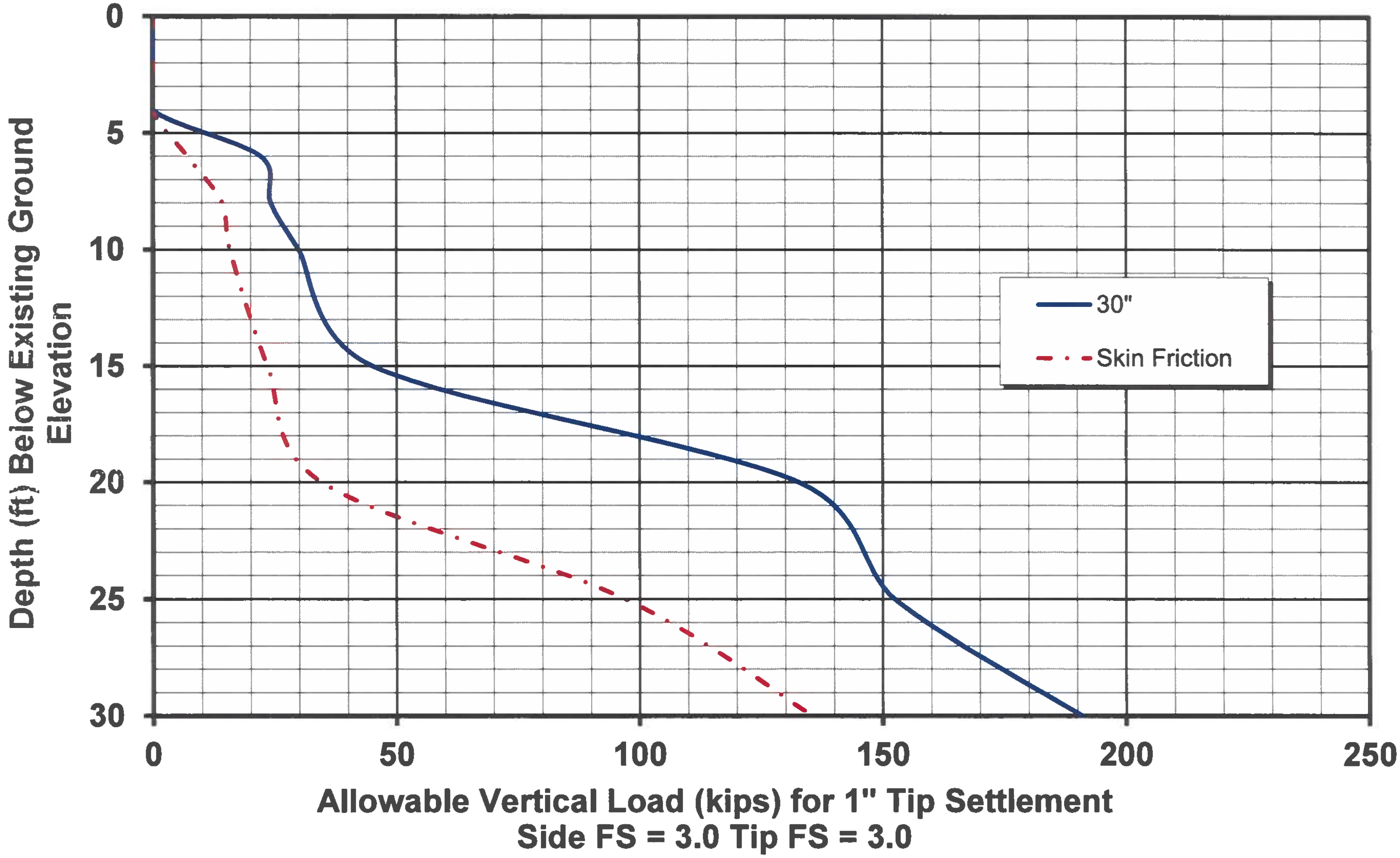
DRILLED SHAFT CAPACITIES



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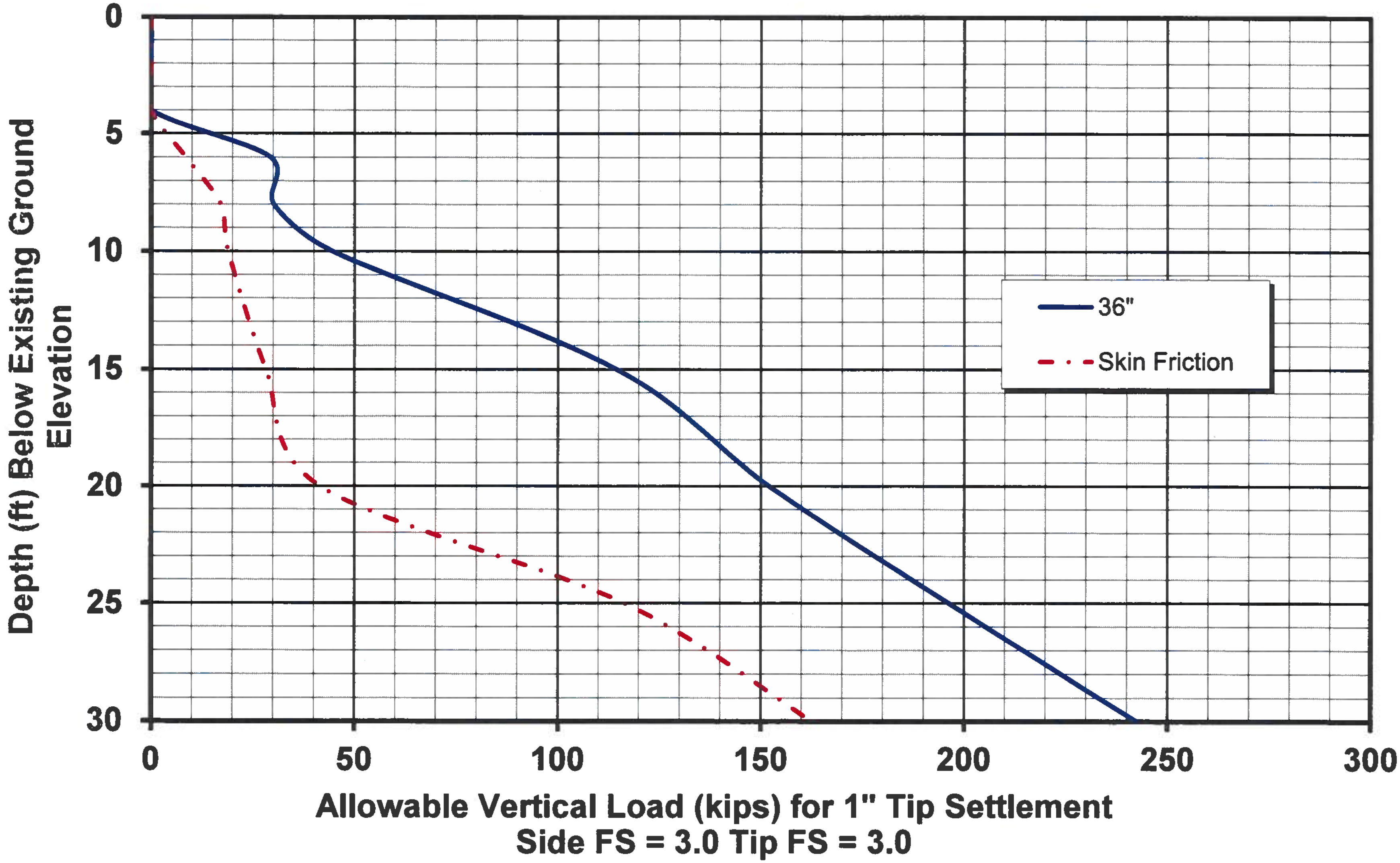
DRILLED SHAFT CAPACITIES



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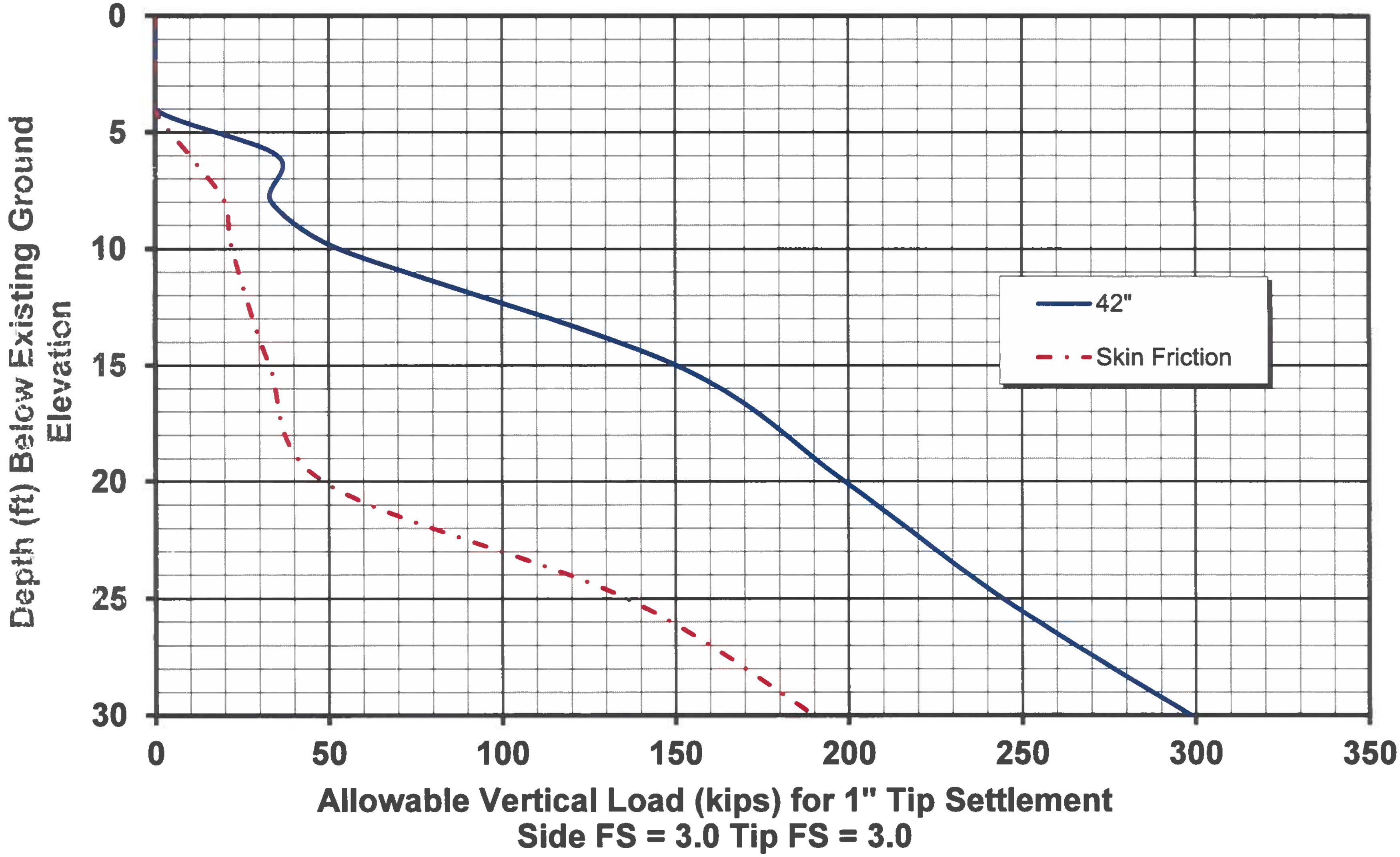
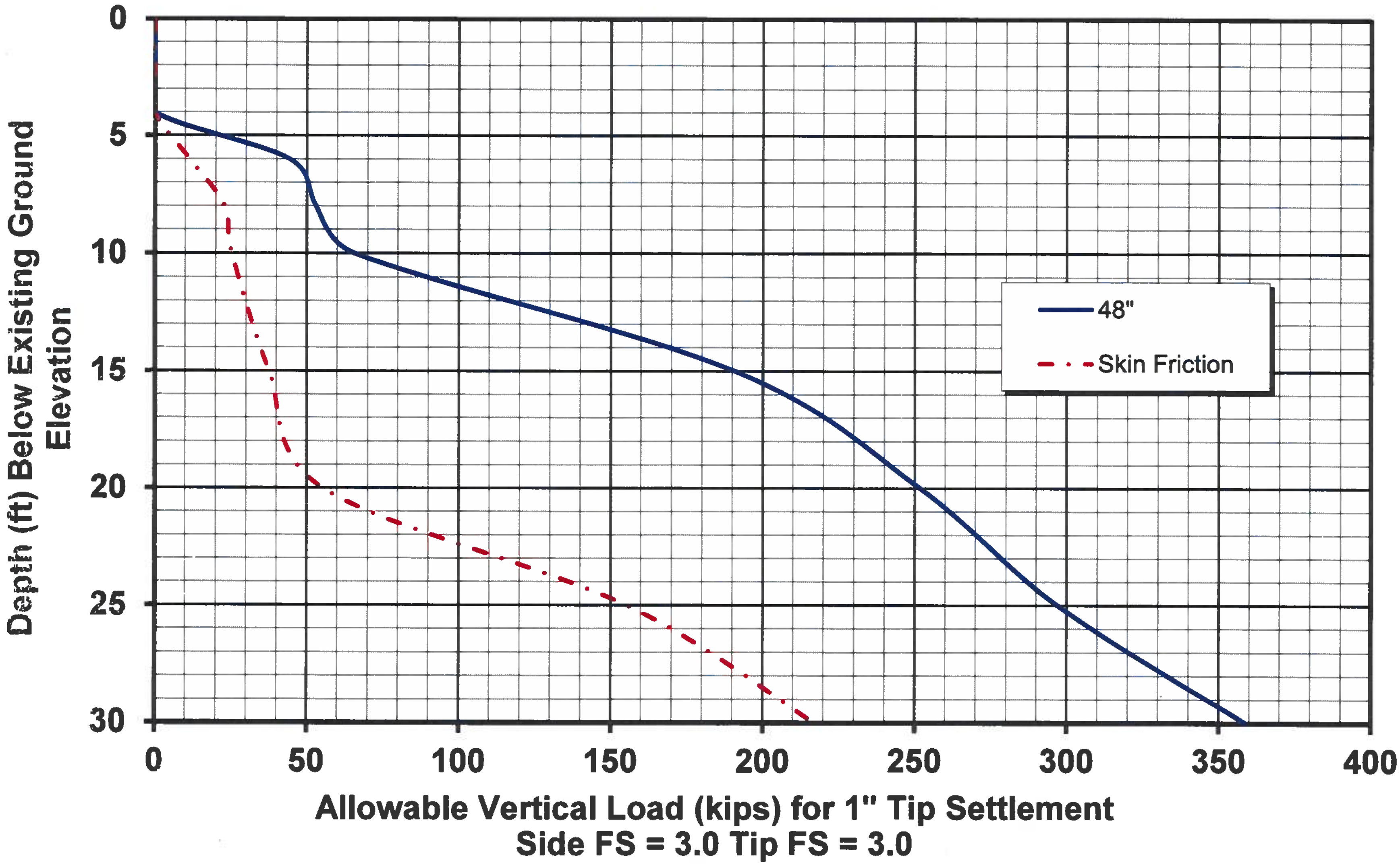


DRILLED SHAFT CAPACITIES



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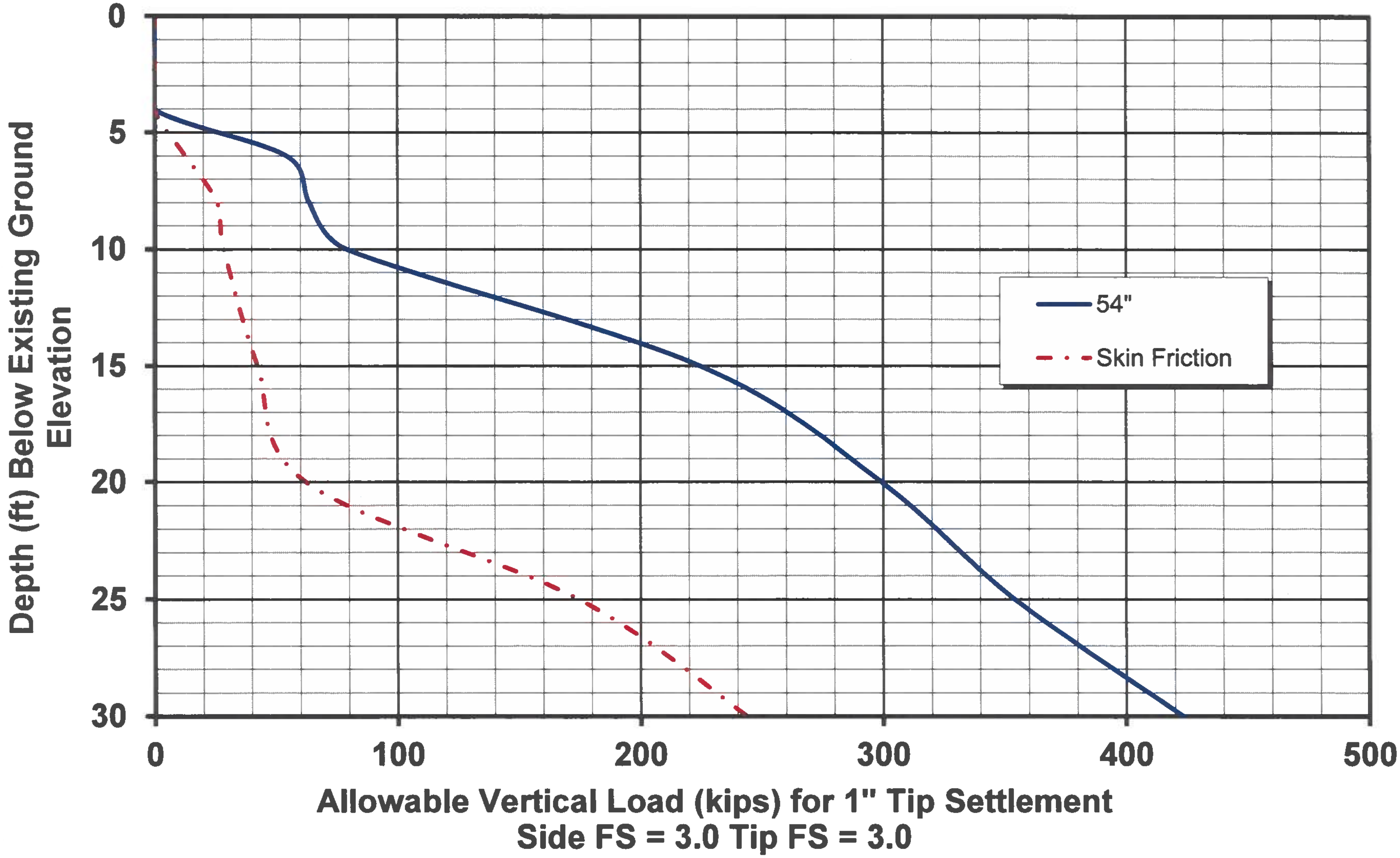

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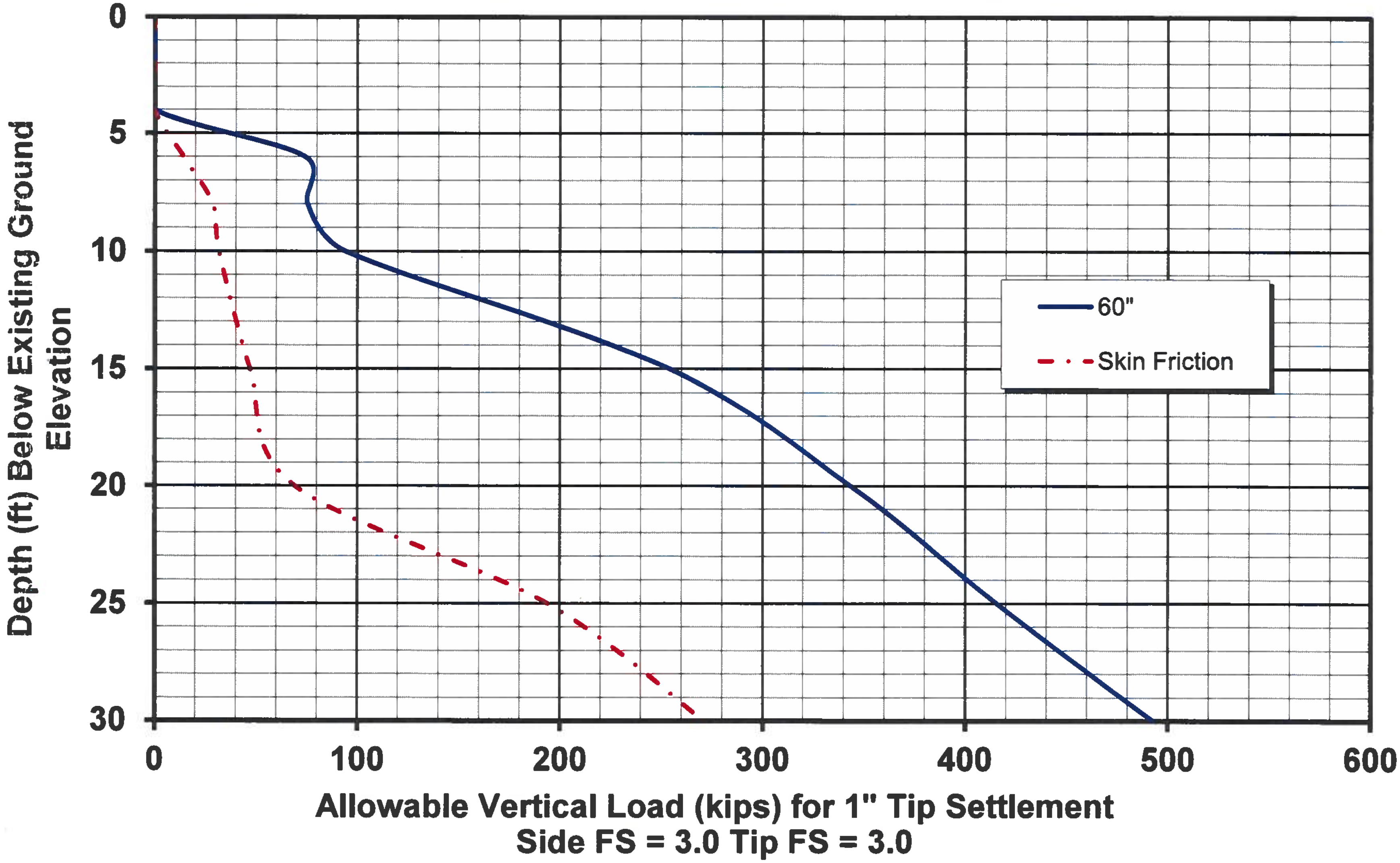

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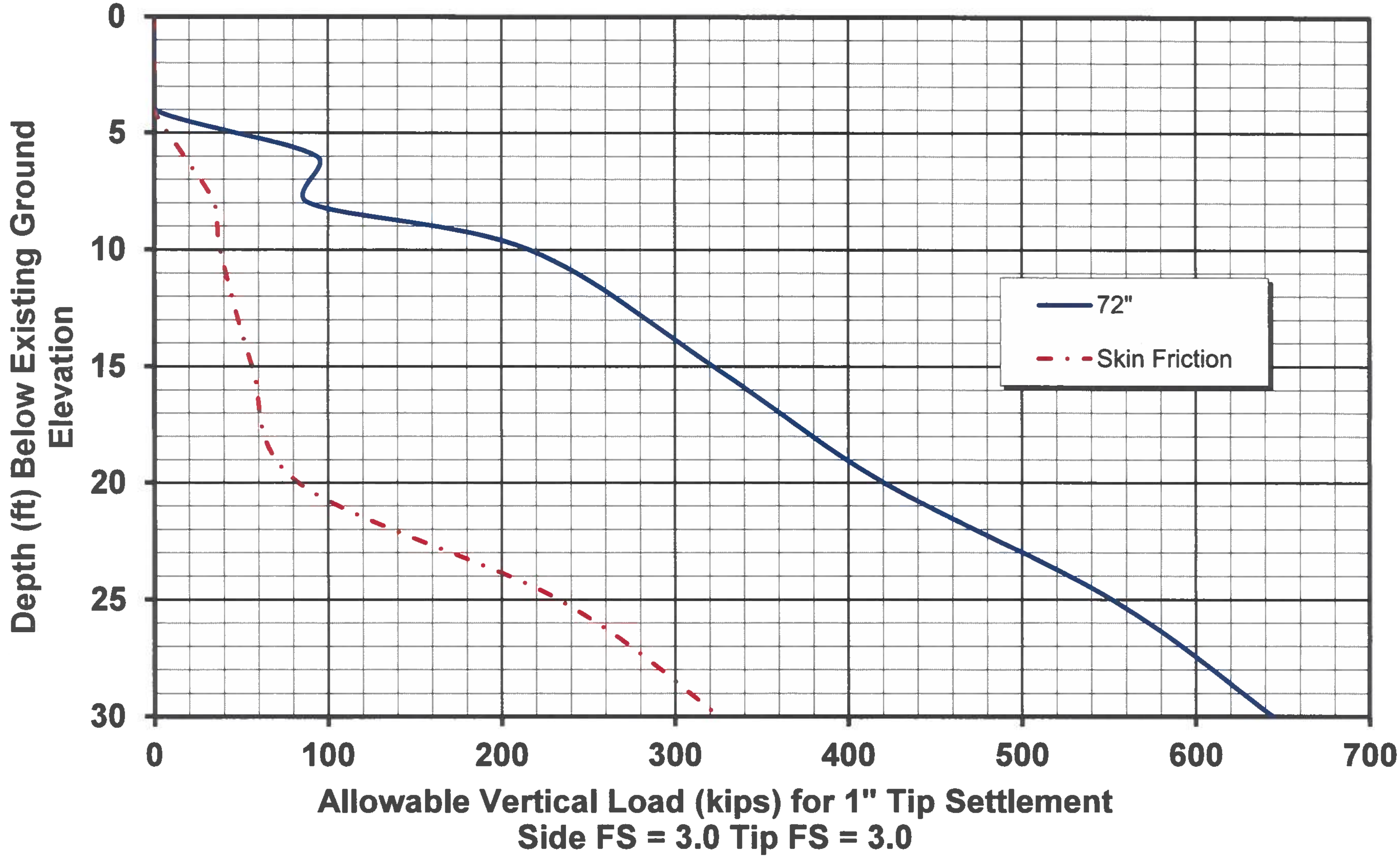
DRILLED SHAFT CAPACITIES



GPD GROUP

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

Phone 330.572.2100
www.gpdgroup.com

November 8, 2024

Statement of Compliance

GPD Group Inc., (GPD) has performed a NEPA Report dated November 8, 2024, for the site- Leon County Texas- Sheriff's Office.

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

Antenna Structure Registration

TOWAIR Determination Results

*** NOTICE ***

TOWAIR's findings are not definitive or binding, and we cannot guarantee that the data in TOWAIR are fully current and accurate. In some instances, TOWAIR may yield results that differ from application of the criteria set out in 47 C.F.R. Section 17.7 and 14 C.F.R. Section 77.13. A positive finding by TOWAIR recommending notification should be given considerable weight. On the other hand, a finding by TOWAIR recommending either for or against notification is not conclusive. It is the responsibility of each ASR participant to exercise due diligence to determine if it must coordinate its structure with the FAA. TOWAIR is only one tool designed to assist ASR participants in exercising this due diligence, and further investigation may be necessary to determine if FAA coordination is appropriate.

DETERMINATION Results

Structure does not require registration. There are no airports within 8 kilometers (5 miles) of the coordinates you provided.

Your Specifications

NAD83 Coordinates	
Latitude	31-15-30.7 north
Longitude	095-58-26.8 west
Measurements (Meters)	
Overall Structure Height (AGL)	42.7
Support Structure Height (AGL)	36.6
Site Elevation (AMSL)	94.2
Structure Type	
LTOWER - Lattice Tower	

Tower Construction Notifications
Notify Tribes and Historic Preservation Officers of your plans to build a tower.

Federal Communications Commission
45 L Street NE
Washington, DC 20554

Phone: 1-877-480-3201
ASL Videophone:1-844-432-2275
Submit Help Request