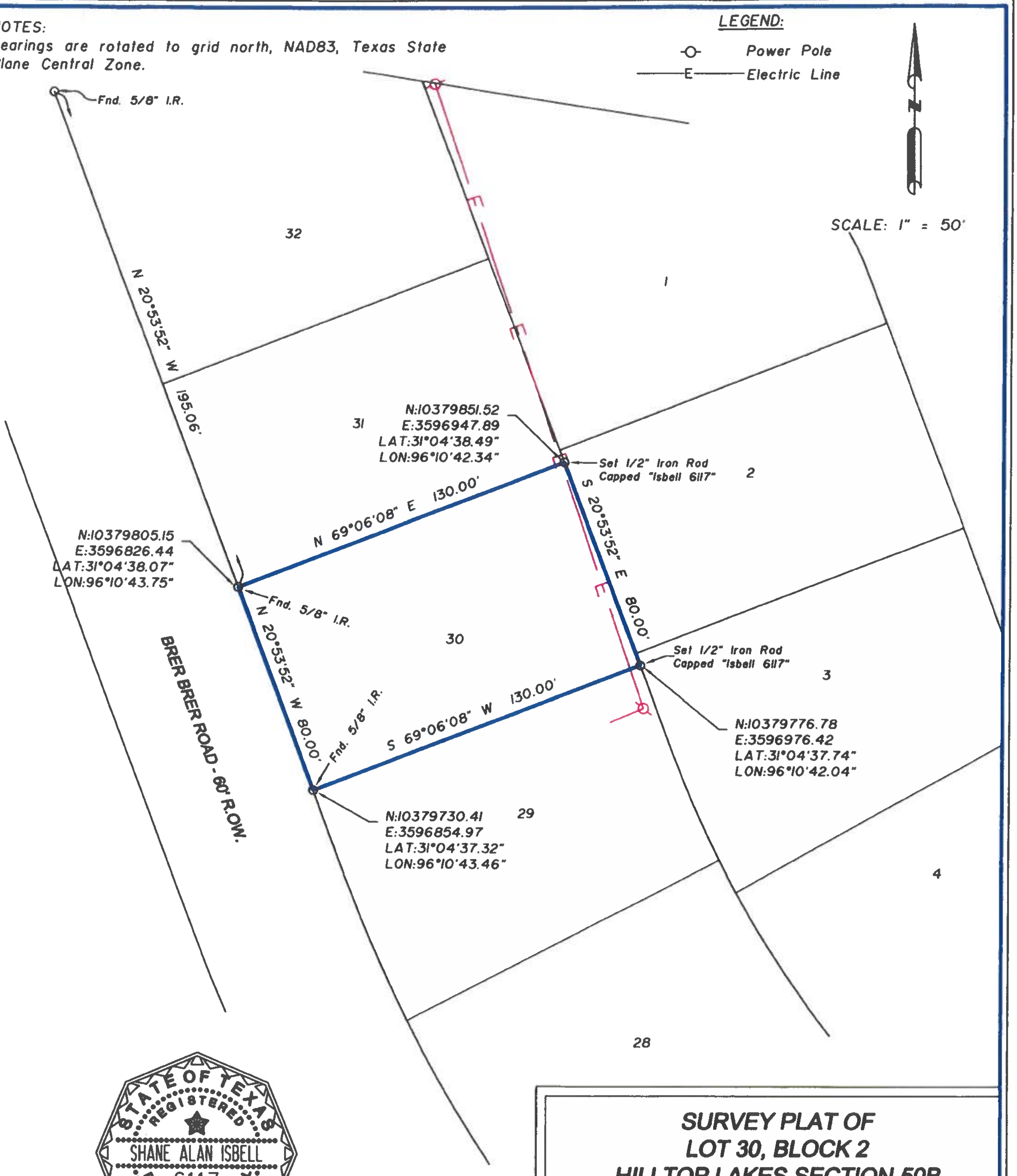
NOTES:
Bearings are rotated to grid north, NAD83, Texas State Plane Central Zone.



SHANE ALAN ISBELL
6117

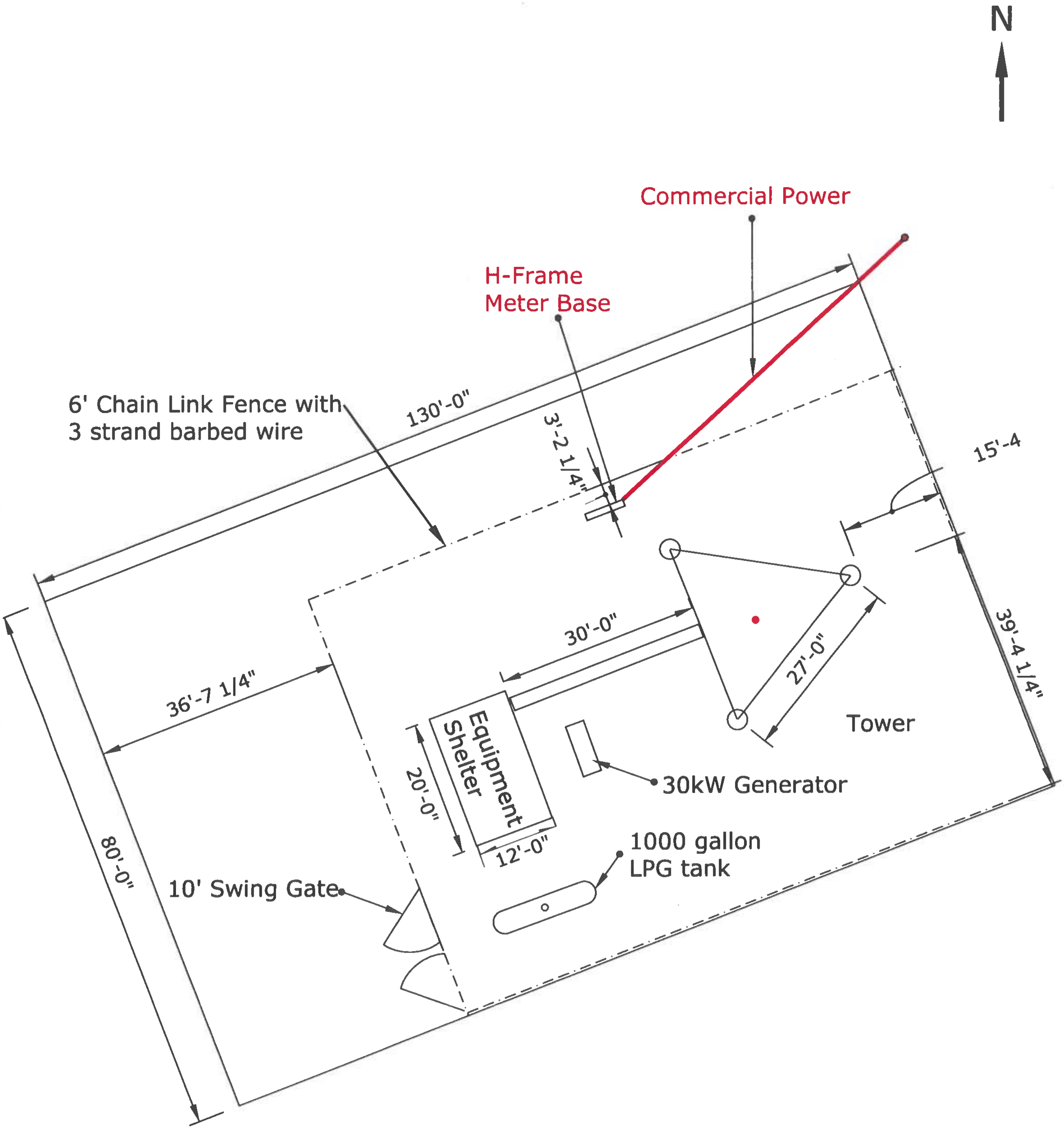
Shane A. Isbell

Registered Professional Land Surveyor No. 6117

SURVEY PLAT OF
LOT 30, BLOCK 2
HILLTOP LAKES SECTION 50B
J. M. VIESCA XI LEAGUE GRANT, A - 30
LEON COUNTY, TEXAS
OCTOBER 24, 2025

PREPARED BY:
ISBELL LAND SURVEYING
1366 CR 320/CENTERVILLE, TX/Ph. 979-255-9177
FIRM REGISTRATION NO. 10192000
JOB NO. 25081

Conceptual Site Plan R1_20251027



Tower Center Coordinates
Latitude: 31-04-38.01 N
Longitude: 96-10-42.56 W

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GEOTECHNICAL ✶ MATERIALS ✶ ENVIRONMENTAL ✶ DRILLING ✶ LANDFILLS

April 10, 2026

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

SUBJECT: Leon County – Hilltop Lakes Radio Tower
Hilltop, Texas
Geotechnical Investigation
ETTL Job No. G 6669-259

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

James Werbiski, E.I.T.
Project Manager

Owen Sanderson

Owen B. Sanderson, P.E.
Manager of Engineering Services

Owen B. Sanderson

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Date: 2026.04.13 09:29:26 -05'00'

OWEN B. SANDERSON
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04/10/2026

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Geotechnical Investigation

Leon County
Hilltop Lakes
Radio Tower

Hilltop, Texas

Submitted To:

TJ Foley
Leon County
Centerville, Texas

Prepared by;

ETTL Engineers & Consultants Inc.
Tyler, Texas

April 2026

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 October 27th, 2025. The field operations were conducted on March 31st, 2026.

The purpose of this investigation was to define and evaluate the general subsurface conditions of the proposed tower site located at Hilltop Lakes, Hilltop, Texas. A site map depicting the project location is included in **APPENDIX A**, as is a Plan of Borings showing the selected boring locations 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 that form the basis of 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 self-supported 380-foot radio tower.

A deep foundation pier design is 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 the finished grade.

3.0 SITE DESCRIPTION

Based on Google Earth Data and on-site observations, the site is heavily wooded with mature trees. The topography of the site is unknown at this time, and a site survey has not been provided.

4.0 FIELD OPERATIONS

The subsurface conditions at the site was defined by one (1) sample core boring. The field boring logs were prepared as drilling and sampling progressed. The plan of borings and final logs are included in **APPENDIX A**. Descriptive terms and symbols used on the logs are in accordance with the Unified Soil Classification System (ASTM D 2487). A reference key is provided on the final page of this report.

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

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 not observed during flight auger drilling. Upon completion of the drilling activities, the open boreholes was measured for groundwater. The phreatic surface is estimated to be at depths below the boring termination depth of 30 feet below existing grade (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 bore hole 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 borehole, which often makes it impossible to distinguish between groundwater and drilling fluid.

If the designer believes that groundwater levels could significantly impact the project, ETTL should be contacted to develop a plan for installing and monitoring piezometers to more accurately assess groundwater levels at the site.

5.0 LABORATORY TESTING

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

Laboratory tests were conducted on samples recovered from the borings to evaluate the physical and engineering properties of the different strata, 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	5
Dry Sieve Analysis (% Passing No. 40)	ASTM D 6913	5
Washed Sieve Analysis (% Passing No. 200)	ASTM D 1140	5
Atterberg Limits (Liquid & Plastic Limits)	ASTM D 4318	5
Moisture Content by Dry Weight	ASTM D 2216	5

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 Sparta Formation (Es).

The Sparta Formation consists of fine to medium-grained quartz sand that is light gray to brownish-gray in color. It is slightly cohesive from silt and clay matrices that could be massive or locally cross-bedded. Interbeds of sandy clay are more abundant in the upward proportions of the formation. The formation is locally carbonaceous and weathers to various shades of light gray. Its base contains hard, brown, ferruginous sandstone. The lower part is 170 +/- feet thick and the upper part is absent. Locally the formation includes Tyler Greensand Member which is described as quartz-glauconite sand, grayish-green in color. This member can be massive and locally cross-bedded. It weathers to reddish brown with abundant ironstone concretions.

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	Very loose to loose silty sands (SM)	0 – 18
2	Dense sands (SM/SC)	18 – 27
3	Hard Expansive clay (CH)	27 – 30

The classifications are based on weathering, depositional environment, mineralogy, color change, lithology, and structure. Detailed in 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 distinct boundaries between soil types, but in reality, transitions between types are generally gradual.

6.2.1 Soil Properties

Listed in **TABLE 6.2.1** are the predicted properties for each layer or stratum of soil encountered. These properties are derived from our soil testing, published correlations, and our experience with the soils in question. They may not be applicable to design criteria outside the scope of this investigation. Test data from which these properties are derived are included on the logs and/or test reports in the **APPENDIX**. Where soils vary within a layer, predicted properties for the layer represent the most conservative configuration.

TABLE 6.2.1 - Soil Properties

Description of Layer	Depth	Drained (Sand)/Undrained (Clay) Conditions			Soil Type (For L-Pile Analysis) [4]	Soil Class
		Cohesion c (psf) [2]	Moist Unit Wt. (pcf)[1]	Angle of Internal Friction [3]		
Compacted Select Fill	Where occurs	-	120	30	Sand (Reese)	SM, SC, CL
Loose Sand (N<10)	Where occurs	-	110	28	Sand (Reese)	SM
Dense to Very Dense Sand (N>30)	Where occurs	-	120	32	Sand (Reese)	SM, SC
Hard Clays	Where occurs	2500	125	-	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, -200%>30.

6.3 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 drilled to a maximum depth 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 improve the site classification.

Based on the 2015 IBC, the seismic site class definition is **Class D "Stiff Soil",** and the Risk **Category is II** (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 when selecting the foundation type for the proposed structure.

1) 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, pressure induced by the structure onto the foundation soils (factored for LRFD analysis).

2) 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, a predicted total settlement or heave of 1 inch is widely considered an acceptable design target. *It should be noted, however, that if the differential settlement or heaving of this magnitude (1 inch) were to occur, cosmetic distress, including cracks in walls and floors and door frame distortion, etc., can be expected at least in some circumstances.*

7.1 General Considerations

Due to the geologic conditions of deep, very loose to loose sands, temporary surface casing and/or slurry drilling techniques may be required to prevent 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 filled with high-slump concrete reinforced with a steel cage. Steel should be adequate to resist uplift loads, 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 increased corrosion potential. Only straight shafts are considered, as under-reaming in hard clays may be problematic.

Sizes for which static vertical load design information is provided herein include 18" to 72", which are believed sufficient to cover the anticipated loading ranges adequately. 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: Construction Procedures and LRFD Design Methods – FHWA GEC 010 - 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 that are larger in diameter and shallower, since the volume of concrete is less for the former and capacity in 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. It 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 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. In contrast, 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 exceeding 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 capacity.

Because of the myriad possible combinations of sizes and loading conditions, and the unknown constraints at any given location, capacity curves are provided to select size and embedment for individual, isolated shafts in native soil. It is usually the case that smaller-diameter, deeper shafts are more economical than larger-diameter, 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 the capacity from the skin friction curve at the depth of the pier cap from the capacity of the shaft, as determined by the procedure set out in this section (below).

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 the shaft diameter should be neglected (this 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 in the design curves is the tip settlement of the isolated pier, not the head settlement. 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 to approximate the elastic compression) to see if it is significant. 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 the shaft diameter should be neglected (this 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 the 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 in the design curves is the tip settlement of the isolated pier, not the head settlement. 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 to approximate the elastic compression and see if it is significant). As a rule of thumb, it should only be significant for very slender piers. The settlement of pier groups can be significantly greater than that 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 by moisture infiltration. Tensile loads are not predicted to impact the design of the drilled shafts.

7.2.4 GROUP EFFECTS

Stipulations and recommendations for group action provided below assume a pile cap in firm contact with the ground and good installation practices that minimize soil relaxation along the shaft sidewall. If this is not the case, please get in touch with this office to modify these recommendations.

7.2.4.1 Cohesionless and Mixed Profiles

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, Section 10.8.3.6 recommendations specify a reduction in allowable pier capacity for piers in sandy soils and mixed soil profiles (as is representative of this site) when the center-to-center spacing of multiple row groups is less than 4 shaft diameters. The capacity modification factor varies linearly from 1.0 at a spacing of 4 diameters to 0.8 at 3 diameters, and to 0.67 at 2.5 diameters. For single-row groups, the reduction factor ranges from 1.0 at a spacing of 3 diameters to 0.9 at a spacing of 2 diameters. This factor should be applied in addition to the safety factor and applies to both the allowable compression and tension loads as derived elsewhere in this report.

Where shaft groups are tipped in a strong soil overlying a soft cohesive soil, additional evaluation is required.

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7.2.4.2 Group Settlement

Settlement of the group should also be evaluated, as it can be significantly greater than that 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 be any reduction in vertical capacity for group action. Evaluation of group settlement was outside the scope of this investigation, but ETTL can assist with such evaluations if provided with specifics on configurations and loads.

7.2.5 Uplift

In this instance, a value of 70% is recommended for calculating the capacity of an individual shaft in uplift as a percentage of the 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 from this skin friction value is compared with the uplift load applied at the top of the shaft, minus the shaft weight. As long as the resistance is greater, the predicted factor of safety against uplift failure equals the safety factor indicated 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 thus entails a collaborative 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, reinforcement, head fixity, and lateral load have been preliminarily determined, and if such conditions warrant more detailed analysis.

Programs such as Ensoft's L-Pile calculate the stiffness of drilled shafts, accounting for reinforcement and cracking (i.e., stiffness reduction) for each load combination. Soil parameter values that should be used in a lateral load analysis are listed in **TABLE 6.2.1**, **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 performed. The analysis also depends on the percentage of steel reinforcement and the magnitude of the vertical load used in conjunction with the maximum horizontal load. The critical combination of loads that yields the maximum horizontal deflection consists of the maximum horizontal load and the minimum vertical load. This combination results in the most severe and least effective moment of inertia (due to section cracking). The L-Pile program considers soil and shaft stiffnesses for any combination of vertical load, lateral load, and moment that the user specifies.

ETTL can assist with lateral load analysis once details regarding shaft diameter, reinforcement, head fixity, and lateral load have been preliminarily determined, and if such conditions 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 because a pile moving toward another moving pile experiences reduced resistance due to the relaxation of the soil behind the leading pile as it moves away from that soil. This reduction is accounted for in LPile or GROUP software analyses by applying a p-multiplier to each pile in the group, based on center-to-center spacing, group configuration, and 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 through a combination of passive earth pressure acting against the faces of footings and pile/pier caps, and lateral resistance provided 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 allowable lateral displacement must be considered. This is related to the direct relationship between lateral load and horizontal deflection for piers, and between lateral movement and the degree of passive resistance that can be mobilized. The magnitude of lateral movement must be consistent across all contributing elements when computing the 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 of the individual piles (modified for group effects, as detailed elsewhere in the report), and the sum should be 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 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.*

7.2.8 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 assess capacity more accurately. Unless the project involves a large number of piers, this approach would generally not 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.

Temporary casing and/or slurry drilling procedures may be required if the shafts do not remain open prior to concreting. 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 accumulated water at the bottom 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 ensure solid contact between the shaft and the side of the hole. Close engineering supervision is essential during the installation of the foundation units to ensure 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 primary. Concrete should be placed at each drilled pier location *immediately* after drilling is completed. 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 be constructed in accordance with ACI 336.1, "Standard Specification for the Construction of Drilled Piers," the latest edition. Only contractors familiar with and competent in using 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 proposed for the project. Due to the deep, loose sands, small incidental structures may be susceptible to vertical movement if the subgrade is not properly prepared.

At a minimum, the subgrade should be prepared as follows.

- Cut out and remove the topsoil and organics.
    - Considerations for the disturbance of the surficial soils should be considered upon the removal of the trees and roots
- 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 ±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 any stability analyses. 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 in accordance with the aforementioned OSHA regulations.

9.0 LIMITATIONS

Geotechnical design work is characterized by the calculated risk that exploratory borings may not have fully revealed soil and groundwater conditions. 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 their spacing are chosen to reduce the likelihood of undiscovered anomalies while considering the nature of the loading, the size, and the cost of the project. The recommendations in this report are based on the conditions that existed at the boring locations when 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 even later. Consequently, the geotechnical engineer must carefully observe the construction as it progresses to 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.***

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
ETTL Engineers and Consultants
Whitehouse, TX

PROJECT
Name: Hilltop Lakes Radio Tower
Number: G6669-259

LOCATION
Hilltop, Texas
31.077473, -96.178764

SYMBOL KEY
⊙ Soil Boring

ETTL | Engineers & Consultants

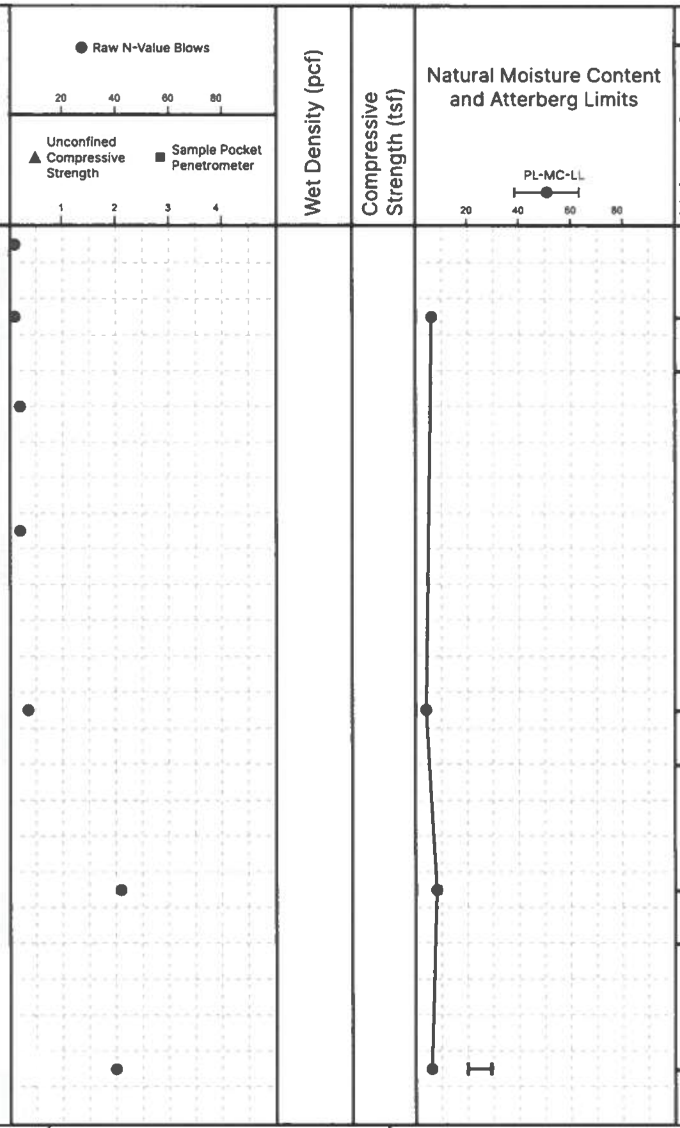
LOG OF BORING B-1

PROJECT: Hilltop Lakes Radio Tower
Hilltop Lakes, Texas

DRILLING METHOD: Flight Auger
DRILL RIG: -

DATE: 03/31/2026
PROJECT NO.: G6669-259
SURFACE ELEVATION: N/A

Corporate Office
3527 Star Road
Whitehouse, TX 75791
(903) 595-4421



USCS Group Symbol	MATERIAL DESCRIPTION	Field Strength Data	Wet Density (pcf)	Compressive Strength (tsf)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	% Plus #40	% Plus #4	% Swell	Restraining Pressure (ksf)
SM	**Silty Sand** (**SM**) very loose; light brown; dry	N= 2											
		N= 2			6.0	NV	NP	NP	7.0	1.0	0.0		
		N= 4											
		N= 4											
	Loose	N= 7			4.0	NV	NP	NP	10.0	0.0	0.0		
	Dense; reddish brown with gray	N= 42			8.0	NV	NP	NP	18.0	0.0	0.0		
SC	23 **Clayey Sand** (**SC**) dense; tan with light gray; dry	N= 40			6.0	29	20	9	45.0	0.0	0.0		

Date	Depth (ft)	Cave-In (ft)	Hours After	Status

Key to Abbrevations:

N - SPT Data (Blows/Ft)
P - Pocket Penetrometer (tsf)
T - Torvane (tsf)
L - Lab Vane Shear (tsf)
N.M. - Not Measured

Notes:
Borehole observed to be open and dry upon completion

GPS Coordinates: 31.077225, -96.178489
Drilled By: Lewis Drilling
Logger: James Werbiski

ETTL | Engineers & Consultants

LOG OF BORING B-1

PROJECT: Hilltop Lakes Radio Tower
Hilltop Lakes, Texas

DRILLING METHOD: Flight Auger
DRILL RIG: -

DATE: 03/31/2026
PROJECT NO.: G6669-259
SURFACE ELEVATION: N/A

Corporate Office
3527 Star Road
Whitehouse, TX 75791
(903) 595-4421

Depth (ft)	Sample Type	USCS Group Symbol	Stratigraphic Unit	MATERIAL DESCRIPTION	Field Strength Data	Wet Density (pcf)	Compressive Strength (tsf)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Minus #200	% Plus #40	% Plus #4	% Swell	Restraining Pressure (ksf)
		SC		27												
30	☒	CH		**Fat Clay** (CH); brownish gray with reddish brown; dry; laminated; mottled 30	N= 47			17.0	62	17	45	91.0	0.0	0.0		
35																
40																
45																

Date	Depth (ft)	Cave-In (ft)	Hours After	Status

Key to Abbrevations:
N - SPT Data (Blows/Ft)
P - Pocket Penetrometer (tsf)
T - Torvane (tsf)
L - Lab Vane Shear (tsf)
N.M. - Not Measured

Notes:
Borehole observed to be open and dry upon completion

GPS Coordinates: 31.077225, -96.178489
Drilled By: Lewis Drilling
Logger: James Werbiski

Boring Log Descriptive Terminology

Key to Soil Symbols and Terms

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	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.
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	●	GM	Silty gravels, gravel-sand-silt mixtures.
			▨	GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	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.
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	▒	SM	Silty sands, sand-silt mixtures.
			▒	SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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 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

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 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.

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

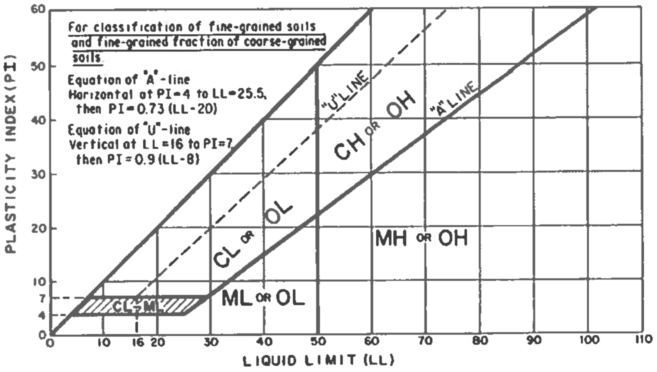
Moisture Condition

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.



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.

APPENNDIX B

Laboratory Testing Reports

ETTL Engineers & Consultants

GEOTECHNICAL★MATERIALS★ENVIRONMENTAL★DRILLING★LANDFILLS

LABORATORY TEST DATA SUMMARY SHEET

PROJECT: Hilltop Lakes - Radio Tower
ETTL JOB NUMBER: G6669-259
PROJECT LOCATION: Hilltop, Texas
CLIENT: Leon County
PROJECT MANAGER.: James Werbiski

START DATE: 4/2/2026
FINISH DATE: 4/7/2026
TECHNICIAN(S): JC
DATE SAMPLED: 3/31/2026

Boring No.	Depth (ft.)		Sample No.	Description of Sample	USCS Classification		Atterberg Limits			Moisture Content (%)	(%) Passing No. 200 Sieve	(%) Retained No. 40 Sieve	(%) Retained No. 4 Sieve	Unit Weight / Compression Tests					Consol. / Swell Tests Results			
	Top	Bott					LL	PL	PI					Total Unit Weight (pcf)	Moisture Content (%)	Compressive Strength (tsf)	Failure Strain (%)	Confining Pressure (psi)	Total Unit Weight (pcf)	Moisture Content (%)	Free Swell (%)	Restraining Pressure (ksf)
B-1	2.5	4.0		Lt. Brown	SM	Silty Sand	NP	NP	NP	6	7	1	0									
B-1	13.5	15.0		Lt. Brown	SM	Silty Sand	NP	NP	NP	4	10	0	0									
B-1	18.5	20.0		Reddish Brown w/ Gray	SM	Silty Sand	NP	NP	NP	8	18	0	0									
B-1	23.5	25.0		Tan w/ Lt. Gray	SC	Clayey sand	29	20	9	6	45	0	0									
B-1	28.5	30.0		Brownish Gr w/ Redd Br	CH	Fat Clay	62	17	45	17	91	0	0									

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

APPENDIX C

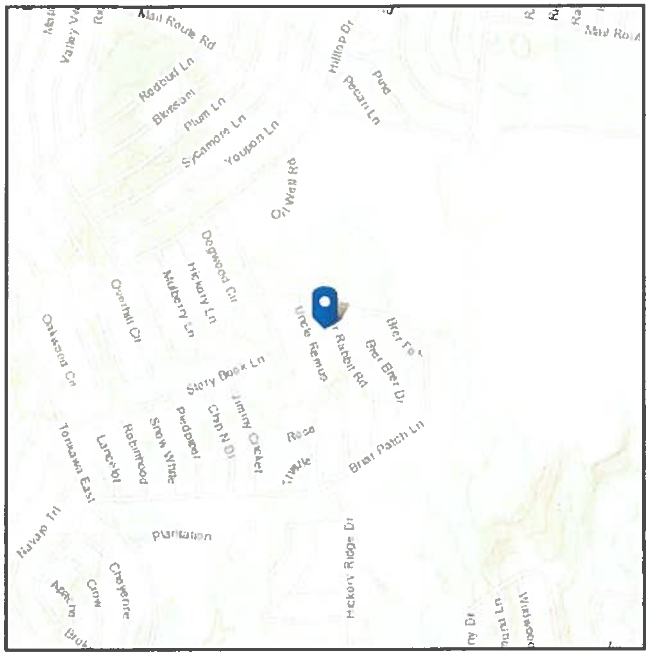
USGS Seismic Design Report

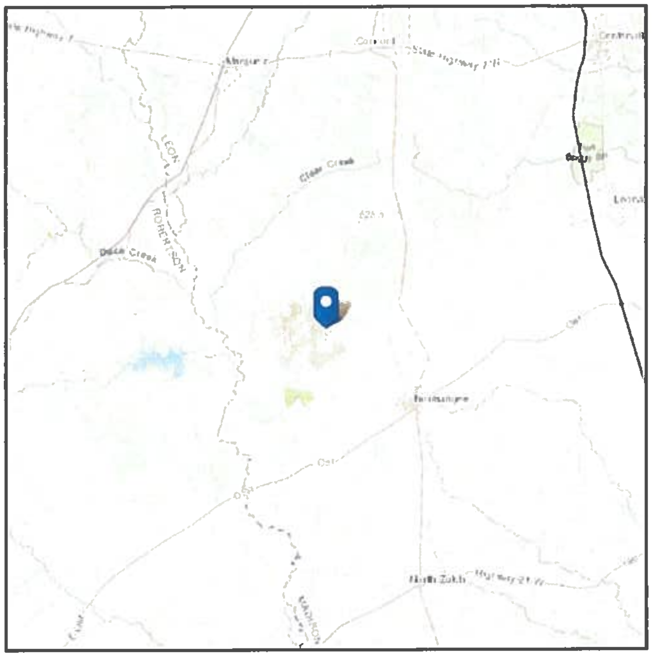
ASCE Hazards Report

Address:
No Address at This Location

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

Latitude: 31.077346
Longitude: -96.180448
Elevation: 470.17782319201535 ft (NAVD 88)





ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

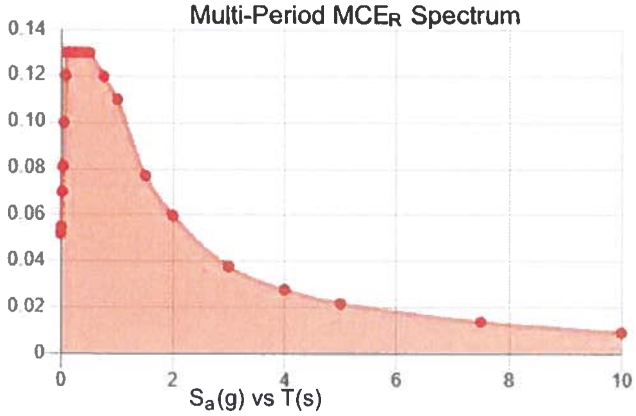
Seismic

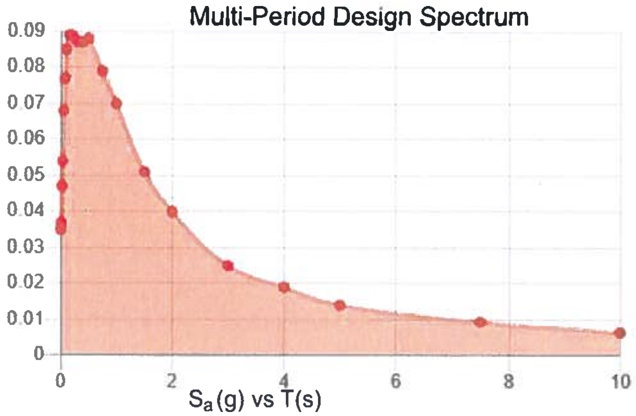
Site Soil Class: D - Stiff Soil

Results:

PGA_M :	0.047	T_L :	12
S_{MS} :	0.12	S_S :	0.09
S_{M1} :	0.11	S_1 :	0.049
S_{DS} :	0.08	V_{S30} :	260
S_{D1} :	0.072		

Seismic Design Category: B


Multi-Period MCE_R Spectrum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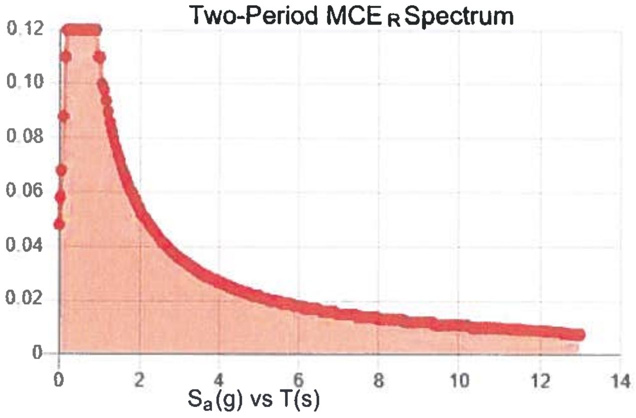
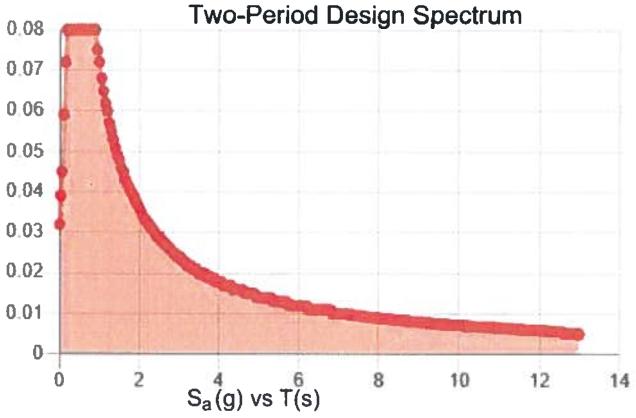

Two-Period MCE_R Spectrum


Two-Period Design Spectrum

MCE_R Vertical Response Spectrum

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

Design Vertical Response Spectrum

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

Data Accessed: **Wed Apr 08 2026**

Date Source:

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

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

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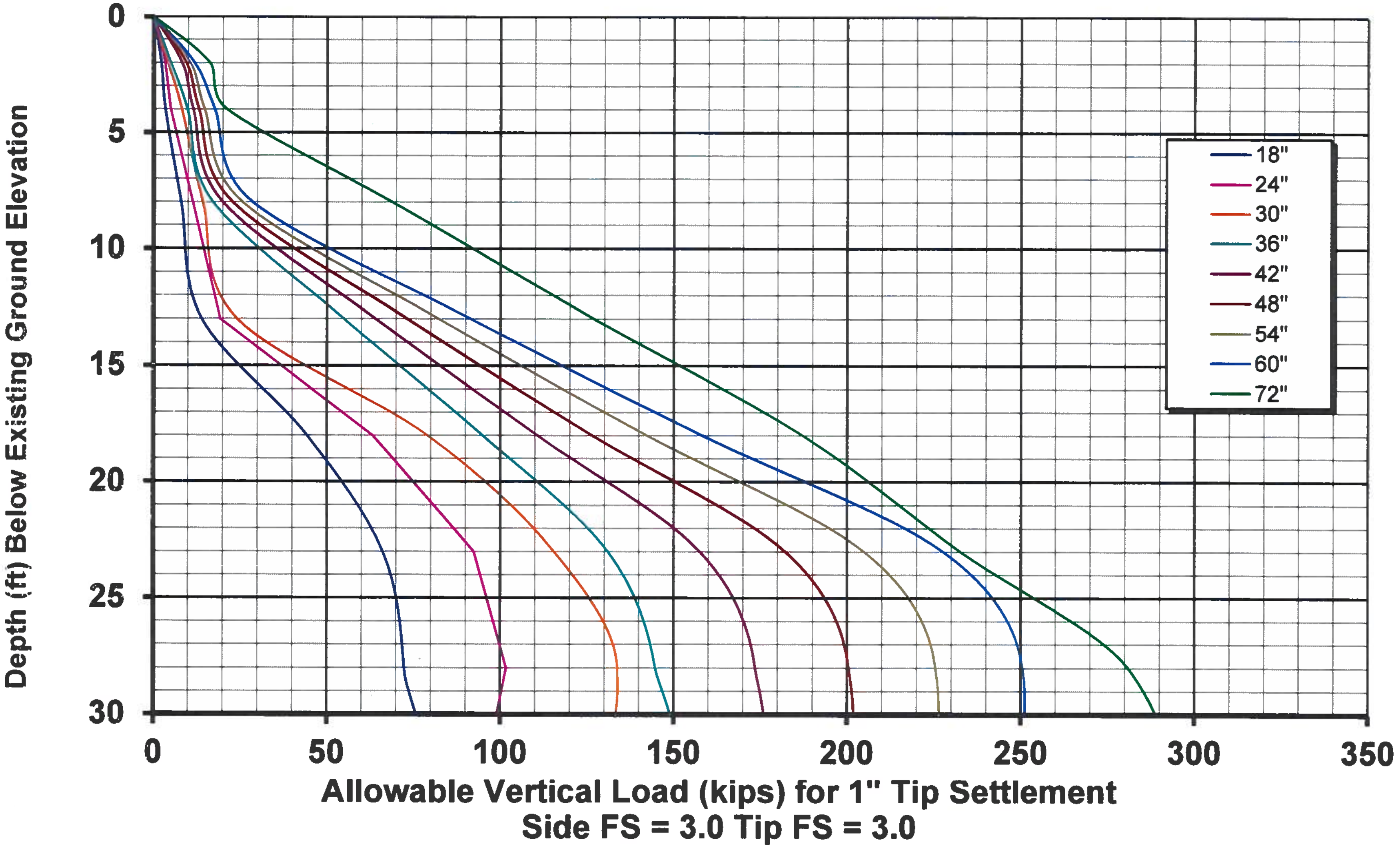
APPENDIX D

Drilled Shaft Capacity Report

ETTL Engineers & Consultants Inc.

GEOTECHNICAL ✶ MATERIALS ✶ ENVIRONMENTAL ✶ DRILLING ✶ LANDFILLS

DRILLED SHAFT CAPACITIES



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Hilltop, Texas

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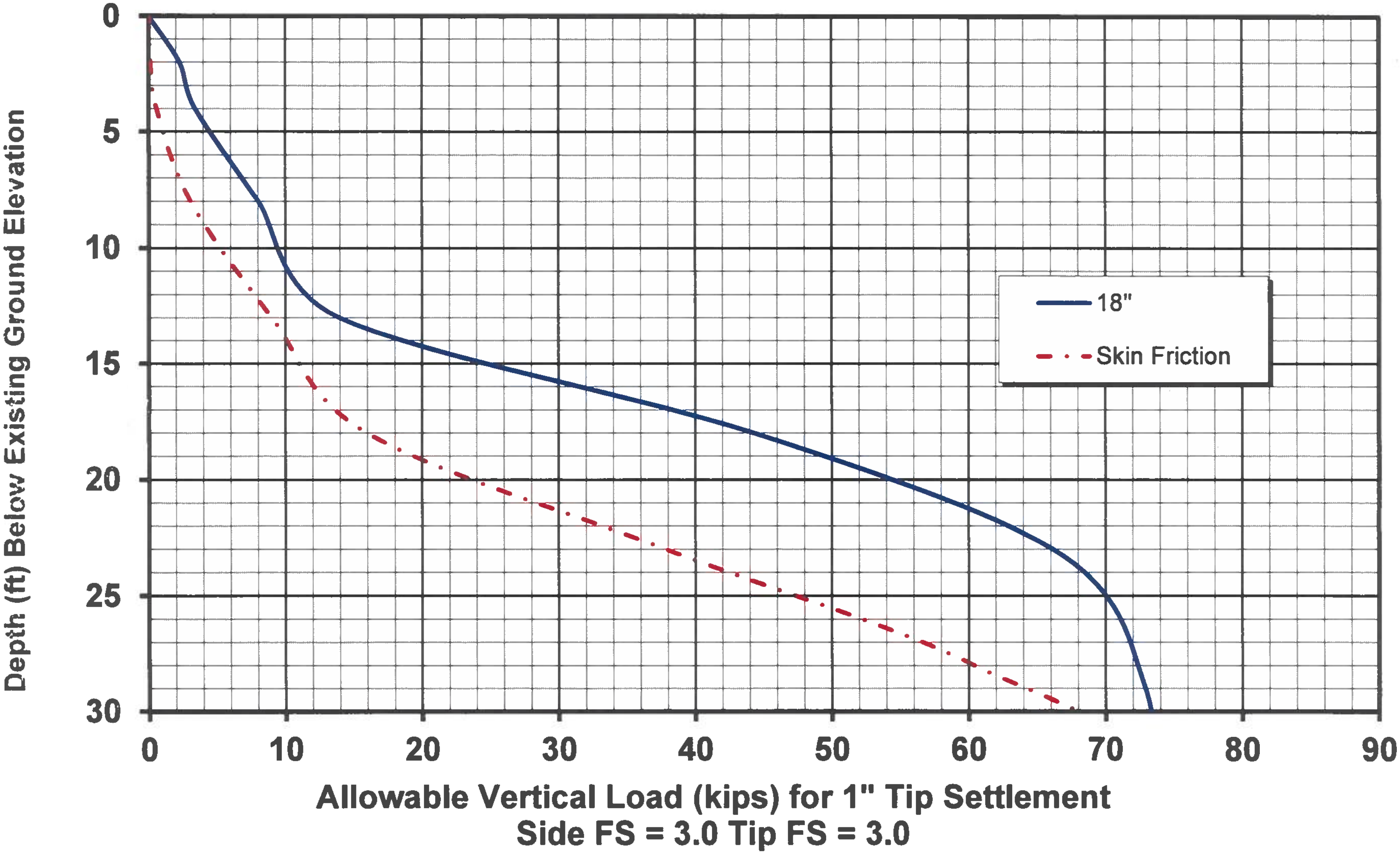
DRILLED SHAFT CAPACITIES



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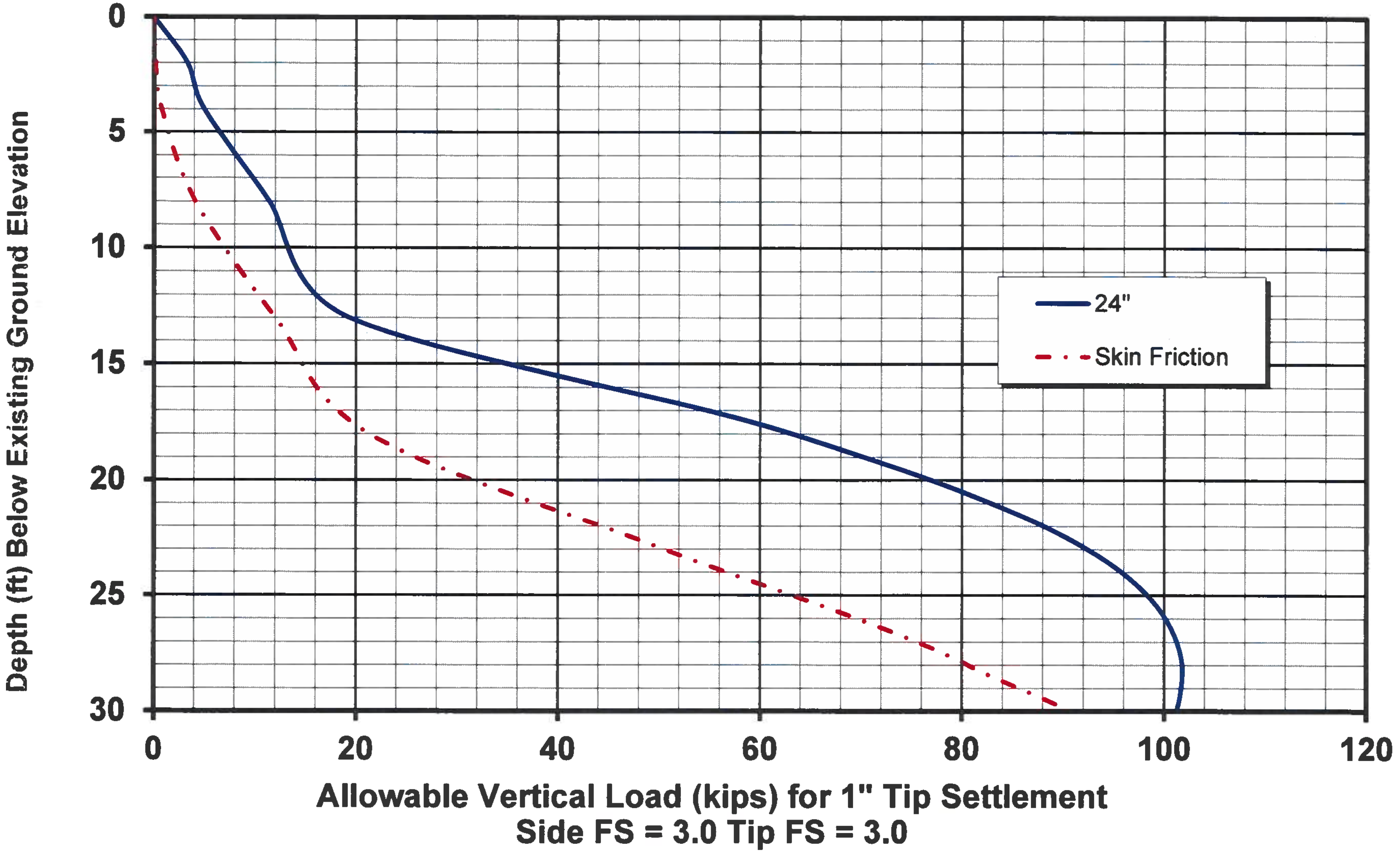
GEOTECHNICAL ✶ MATERIALS ✶ ENVIRONMENTAL ✶ DRILLING ✶ LANDFILLS

DRILLED SHAFT CAPACITIES



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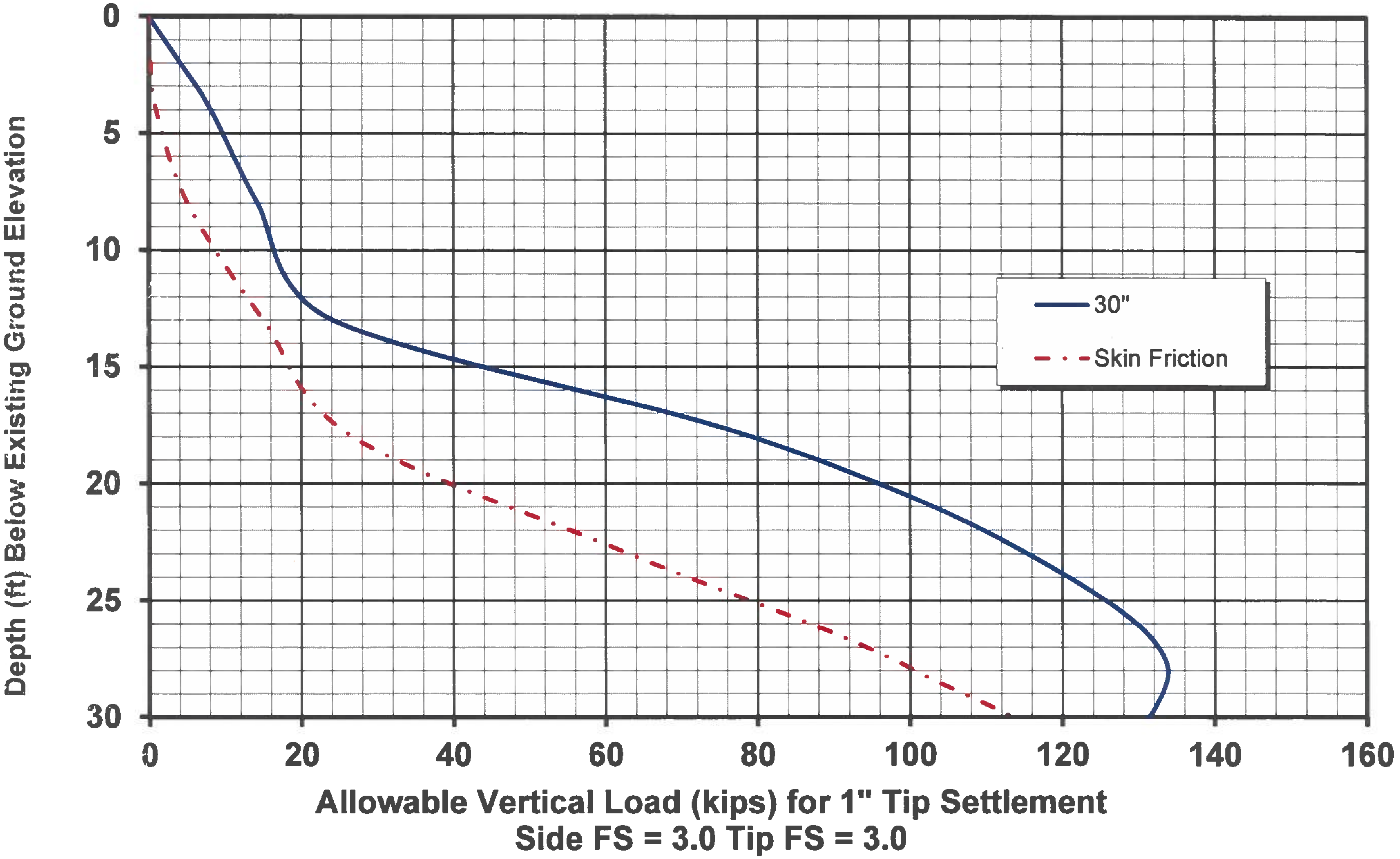
GEOTECHNICAL ✶ MATERIALS ✶ ENVIRONMENTAL ✶ DRILLING ✶ LANDFILLS

DRILLED SHAFT CAPACITIES



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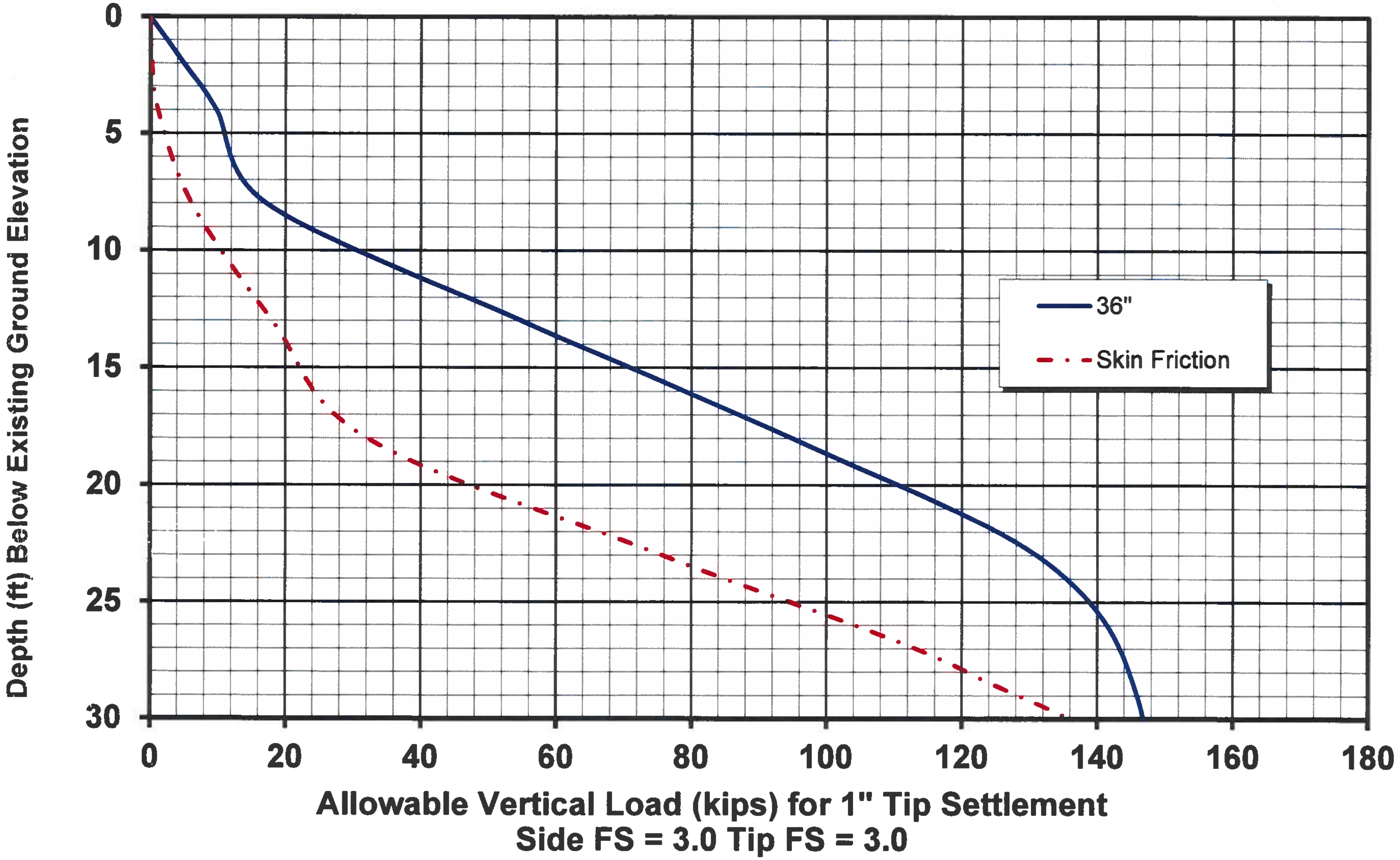

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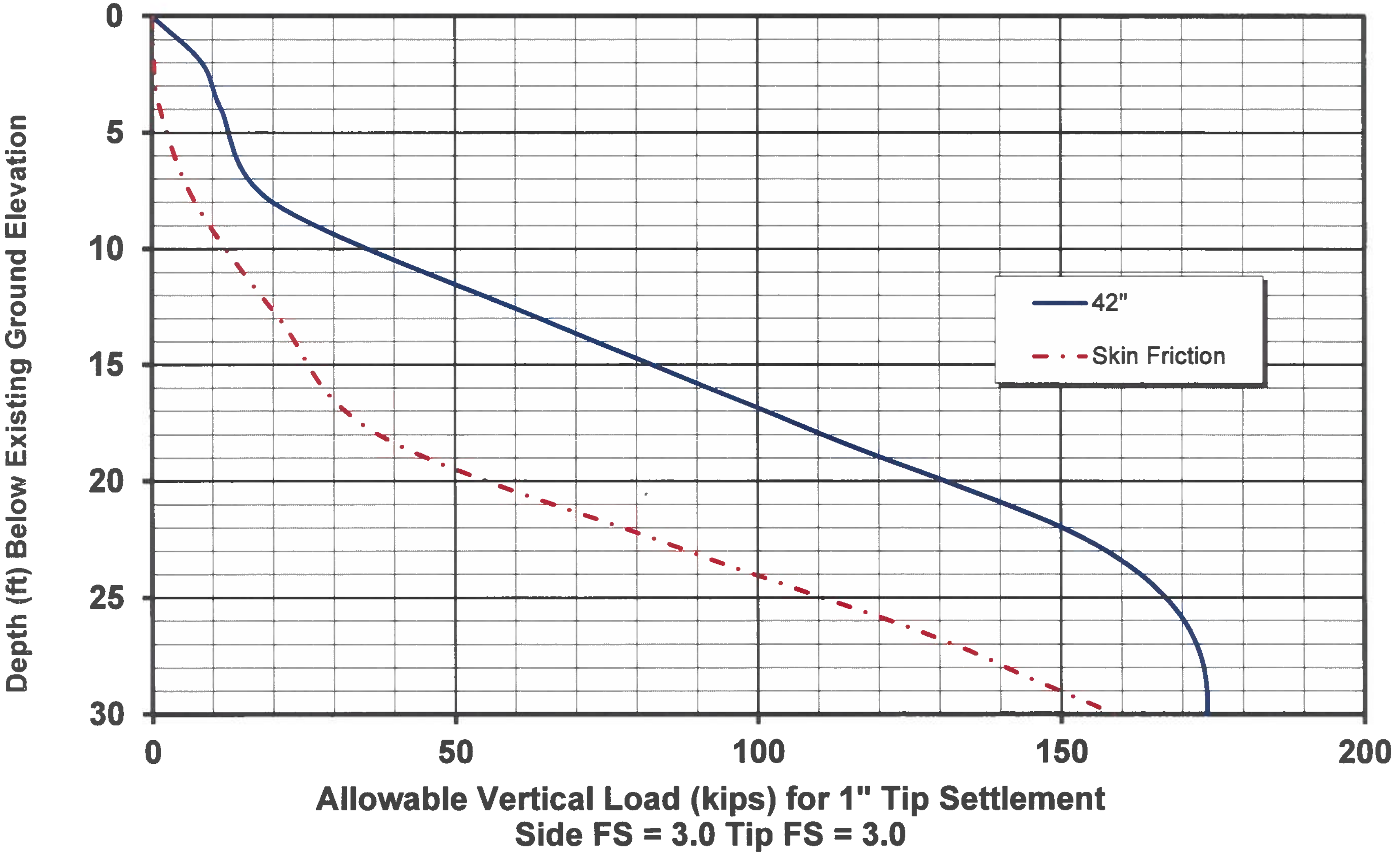

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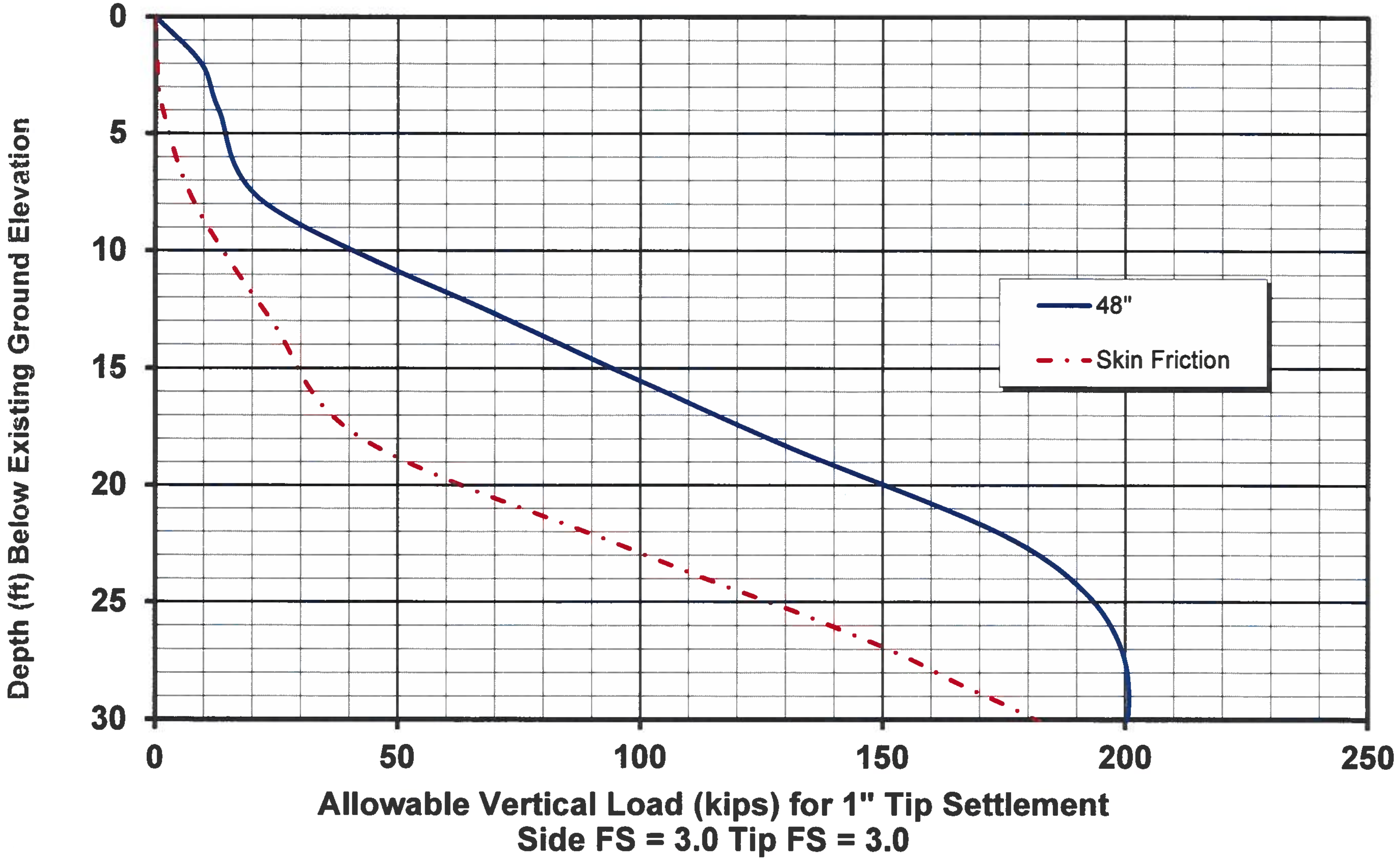

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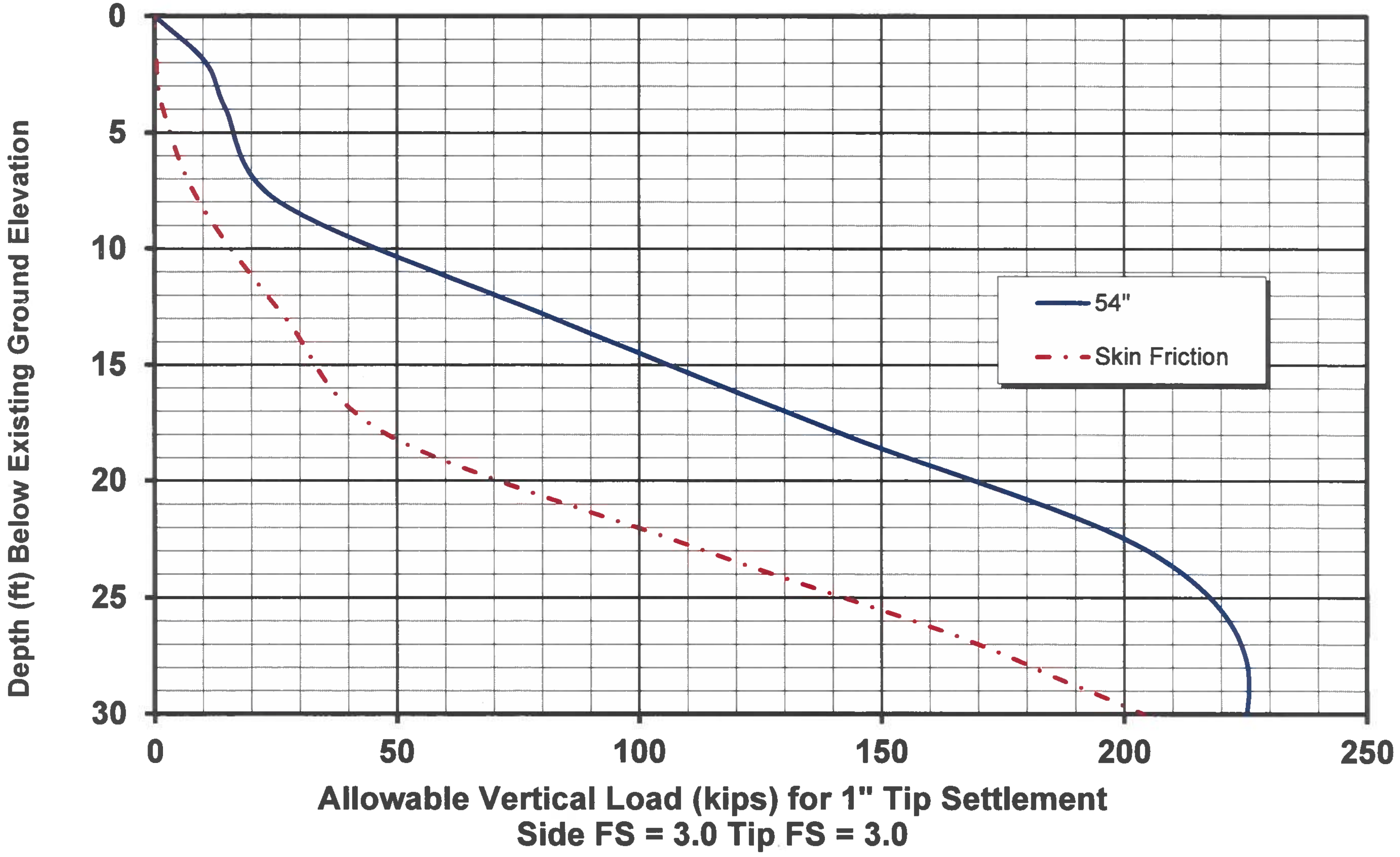

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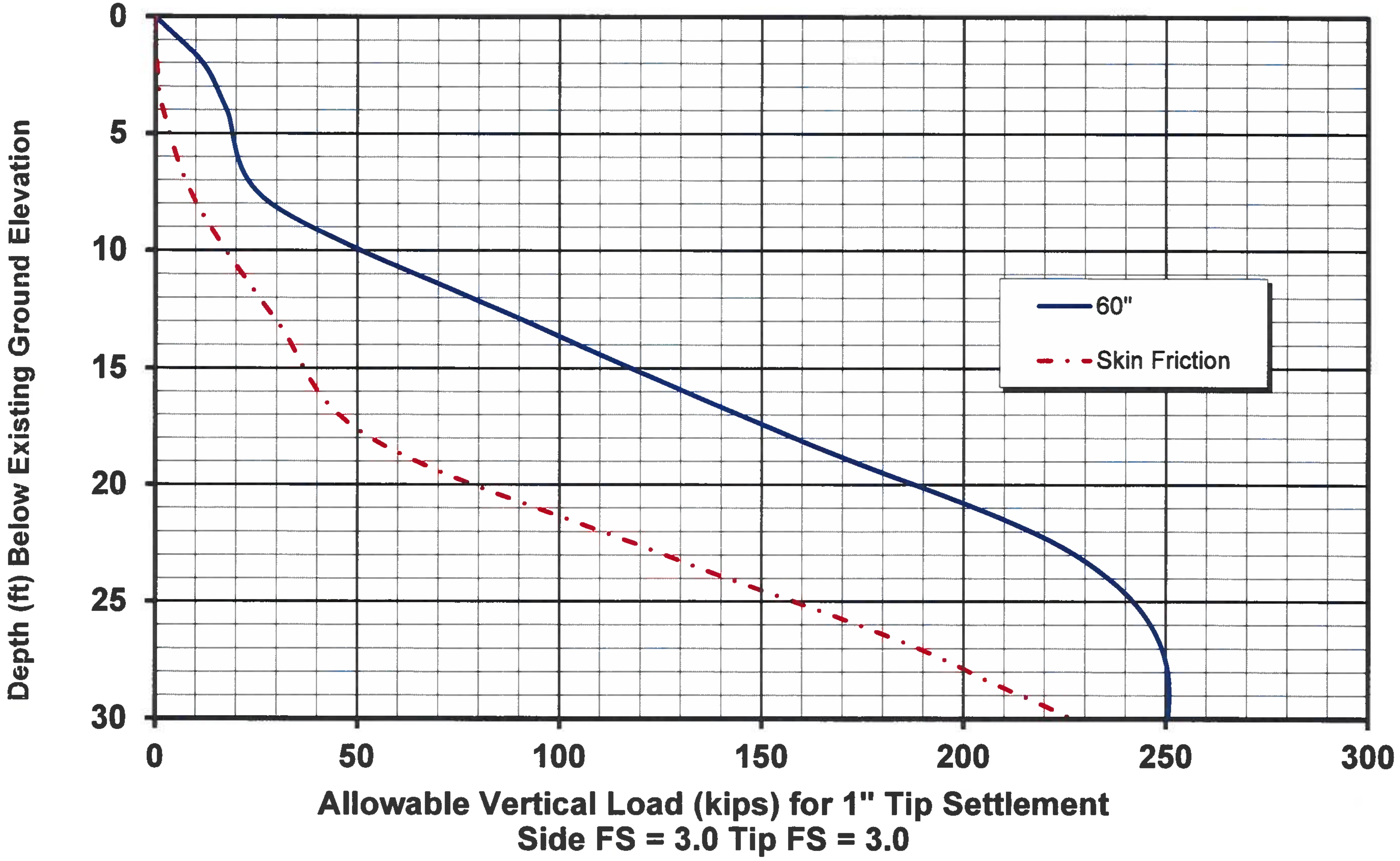

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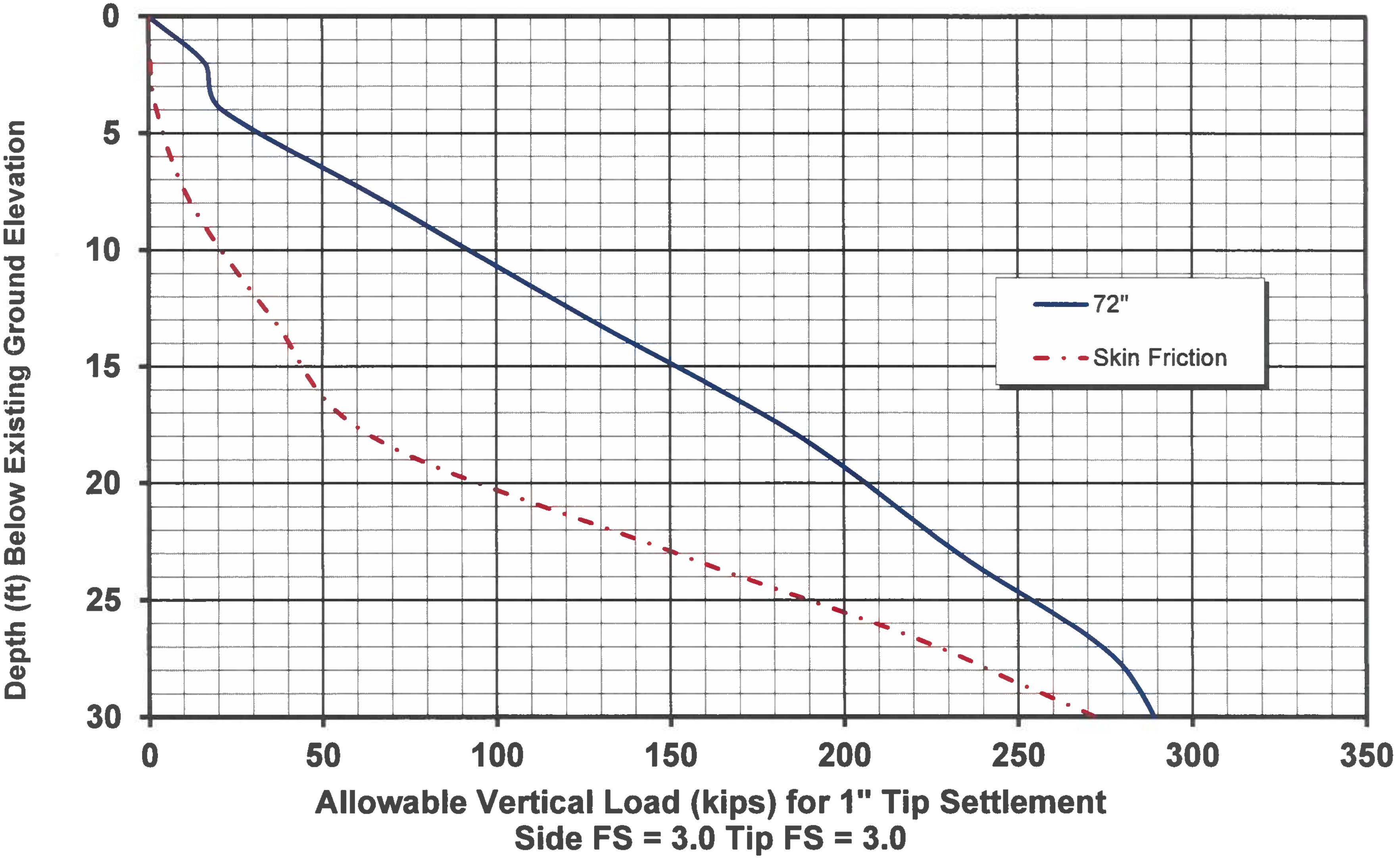

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ETTL Engineers and Consultants, Inc.
Tyler, Texas

GPD GROUP

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

Phone 330.572.2100
www.gpdgroup.com

January 19, 2026

Statement of Compliance

GPD Group Inc., (GPD) has performed a NEPA Report dated January 19, 2026, for the site- Leon County Texas- Leon County Hilltop Lakes.

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
Environmental Scientist

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

Aeronautical Study No.
2025-ASW-13200-OE

Issued Date: 01/13/2026

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

** DETERMINATION OF NO HAZARD TO AIR NAVIGATION **

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

Structure:	Antenna Tower Hilltop Lakes
County, State:	Leon, Texas

Collected Point(s):

Label	Latitude	Longitude	SE	DET AGL	AMSL
Tower	31-4-38.01N	96-10-42.56W	470 Ft	400 Ft	870 Ft

This aeronautical study revealed that the structure would have no substantial adverse effect on the safe and efficient utilization of the navigable airspace by aircraft or on the operation of air navigation facilities. Therefore, pursuant to the authority delegated to me, it is hereby determined that the structure would not be a hazard to air navigation provided the following condition(s) is(are) met:

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

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

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

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

See attachment for additional condition(s) or information.

This determination expires on 07/13/2027 unless: