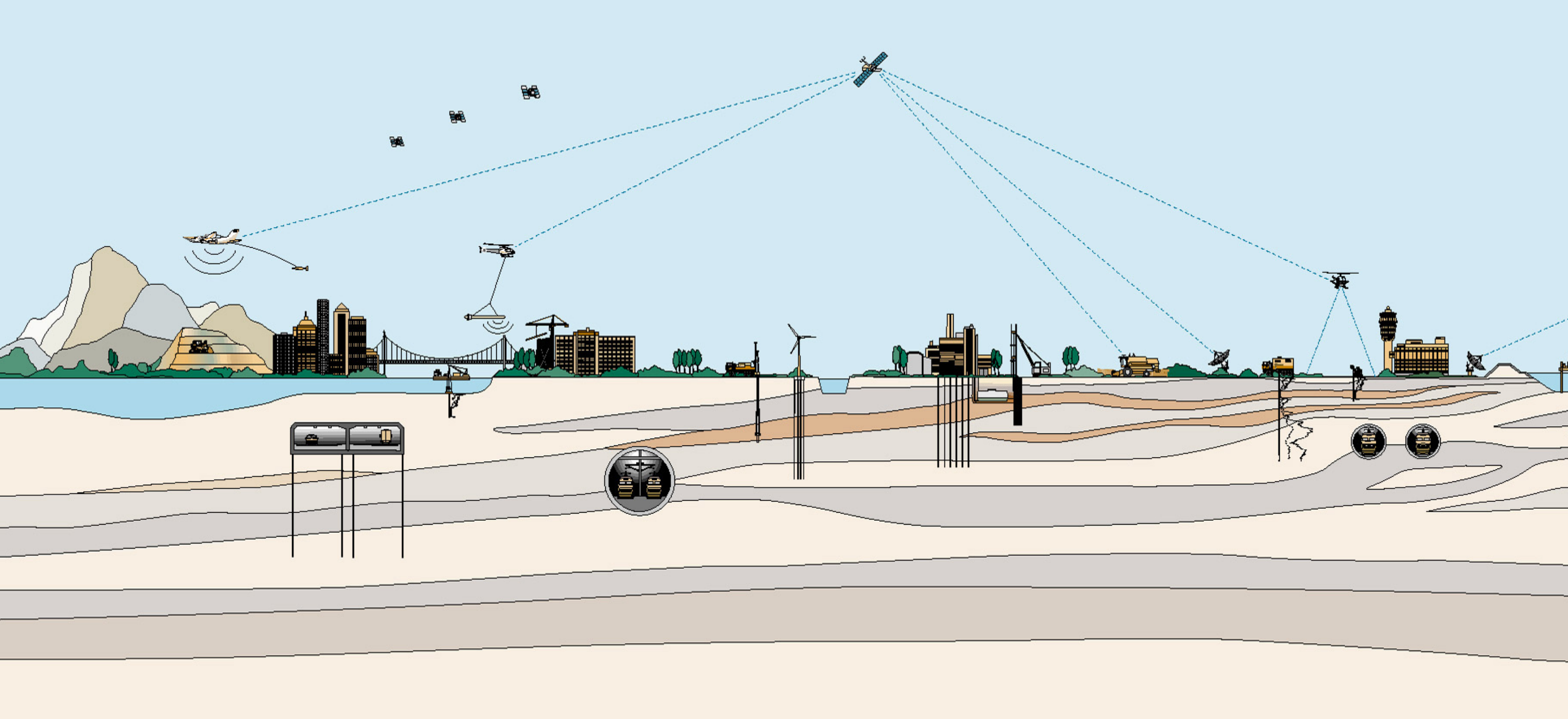
FUGRO USA LAND, INC.

GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY PROJECT
GILCHRIST, TEXAS

HNTB CORPORATION
AUSTIN, TEXAS

MAY 21, 2018



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Report No. 04.10170131
May 21, 2018

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Attention: Mr. Jarrod Choate, ENV SP, GGP, CPTED
Project Manager

Geotechnical Study
Lauderdale Recreational Facility Project
Gilchrist, Texas

Fugro USA Land, Inc. (Fugro) is pleased to present this report of our geotechnical study for the above-referenced project. Mr. Jarrod Choate, ENV SP, GGP, CPTED with HNTB Corporation (HNTB) requested our services on July 11, 2017. Our services were performed in general accordance with Fugro Proposal No. 04.10170131, dated November 8, 2017. Our services were authorized by issuance of Task Order No. 3 under HNTB Contract No. 57947, dated January 1, 2018.

A draft report was submitted to HNTB on April 26, 2018. Comments on the report were provided by HNTB via email on May 4, 2018. This report addresses these comments and supersedes all previously provided geotechnical information for the above-referenced project by Fugro. We appreciate the opportunity to be of service to HNTB. Please call us if you have any questions or comments concerning this report or when we may be of further assistance.

Sincerely,
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1.0 INTRODUCTION

1.1 Project Description

The Texas General Land Office (GLO) is planning to construct the proposed Lauderdale Recreational facility on the north side of Highway 87 in Gilchrist, Texas. A Vicinity Map of the project site is presented on Plate 1. We understand that HNTB is providing engineering and design services for the above mentioned project. The project involves the replacement of the existing dock and boat ramp located adjacent to Yacht Basin Road that extends north from Highway 87. Additionally, the proposed park improvements will include the construction of a shelter area, restroom/shower facility, parking lot, roadway improvements to Yacht Basin Road, and other utilities. Details of the exact dimensions and loading conditions for the above-mentioned structures were not available at the time of this report. However, we understand that most of the proposed structures will be supported on timber piles. HNTB requested Fugro to provide geotechnical recommendations to support the design of the proposed new facility.

1.2 Purpose and Scope

The purposes of our geotechnical study were to 1) explore and evaluate subsurface soil conditions and depth-to-water at the project site, and 2) develop geotechnical recommendations to guide others in the design and construction of the new recreational facility.

We accomplished these purposes by:

- Drilling 2 nearshore soil borings to a depth of 60 feet each at the location of the proposed dock to explore subsurface conditions and obtain samples for geotechnical laboratory testing.
- Drilling 7 onshore soil borings to depths varying between 20 ft and 30 ft to explore subsurface conditions and obtain samples for geotechnical laboratory testing.
- Performing laboratory tests on selected soil samples to assess pertinent engineering properties.
- Reviewing and analyzing the field and laboratory test data to develop appropriate geotechnical recommendations for the proposed structures.
- Preparing this report summarizing our findings and geotechnical recommendations.

1.3 Applicability of Report

The explorations and analyses for this study, as well as the conclusions and recommendations contained in this report, were selected or developed based on our understanding of the project as described previously and in later sections of this report. If there are differences in location or design features as we understand them, or if the locations or design features change, we should be contacted and authorized to review the changes and, if necessary, to modify our conclusions and recommendations.

We have prepared this report exclusively for HNTB as a guide for the geotechnical aspects of the design and construction of foundations for the aforementioned recreational facility. We have conducted this study using the standard level of care and diligence normally practiced by recognized engineering firms now performing similar services under similar circumstances at the same time and locality. We intend for this report, including all illustrations to be used in its entirety. This report should be made available to prospective contractors for information purposes only and not as a warranty of subsurface conditions. The observations, conclusions, and recommendations presented in this report may not apply to locations not explored by our borings or areas outside the project boundaries defined at the time of this report.

2.0 FIELD INVESTIGATION

This section provides information relating to our field exploration activities for this project. We have included discussions relating to drilling and sampling methods, water depth measurements, and borehole completion. This section is divided in two subsections: onshore field exploration and nearshore field exploration.

2.1 Onshore Field Exploration

We explored the subsurface conditions by drilling 7 onshore geotechnical borings. Our field exploration activities were performed between February 8 and February 9, 2018. The borings are designated Borings B-1 through B-7. The approximate locations of the borings are shown on the *Plan of Borings* presented on Plate 2. The boring locations and coordinates were provided to us by HNTB. The onshore borings were drilled by using our ATV-mounted drilling equipment.

2.1.1 Drilling and Sampling Methods

The onshore borings were drilled using a combination of dry-auger and wet-rotary techniques. Soil samples were taken continuously through any fill material up to a depth of 16-ft and at about 5-ft intervals to the completion depth of the borings, as indicated on the boring logs. Detailed descriptions of the soils encountered along with the boring coordinates are presented on the boring logs on Plates 3 through 9 (Borings B-8 and B-9 were nearshore borings, discussed later in this report, and are presented on Plates 10 and 11). A key identifying the terms and symbols used on the boring logs are presented on Plates 12a and 12b.

Undisturbed samples of cohesive soils were generally obtained by hydraulically pushing a 3-inch-diameter, thin-walled tube a distance of about 24 inches. Our field procedure for sampling undisturbed cohesive soils was conducted in general accordance with ASTM D1587, *Standard Practice for Thin-Walled Tube Sampling of Soils*. The soil samples were extruded in the field and visually classified by our field technician. We obtained field estimates of the undrained shear strength of the recovered samples using a calibrated hand-held penetrometer. The field estimates were modified for stiff to hard, over-consolidated natural cohesive soils, as described on Plate 12b. Portions of each recovered soil sample were placed into appropriate containers for transportation to our laboratory for additional geotechnical testing.

Granular soil samples were obtained using the Standard Penetration Test (SPT). Our field procedure for sampling disturbed cohesive soils and granular soils was conducted in general accordance with the ASTM D1586, *Standard Method for Penetration Test and Split-Barrel Sampling of Soils*. Our field technician recorded the hammer blows for each sampling interval. The SPT N-values are recorded on the boring logs. Soil samples obtained from the split-barrel sampler were visually classified, packaged by the technician, and transported to our laboratory for additional geotechnical testing.

2.1.2 Depth-to-Water Measurements

All onshore borings performed for this study were initially drilled using the dry-auger technique in an effort to identify the depth-to-water. Once water was encountered, drilling was temporarily halted and depth-to-water measurements in the open boreholes were recorded. Drilling was then resumed to the boring completion depths using wet-rotary drilling techniques. Depth-to-water measurements are noted on the boring logs on Plates 3 through 9. Further discussion on our depth-to-water observations is presented later in the *General Site Conditions* Section of this report.

2.1.3 Borehole Completion

After completing the field activities, each onshore boring was backfilled with cement-bentonite grout. Each borehole was grouted from the bottom up, using a tremie pipe. When the grout level reached approximately 4 to 6 inches of the ground surface, the tremie pipe was removed and the boreholes were topped-off by pouring grout from the surface.

2.2 Nearshore Field Exploration

We explored the nearshore subsurface conditions by drilling 2 nearshore geotechnical borings. Our field exploration activities were performed between February 13 and February 14, 2018. The borings are designated Borings B-8 and B-9. The approximate locations of the borings are shown on the *Plan of Borings* presented on Plate 2. The boring locations and coordinates were provided to us by HNTB. The nearshore borings were drilled using a pontoon-mounted drilling equipment and casing was set from the pontoon deck into the mudline. Water depth during drilling was observed to be at about 6 feet.

2.2.1 Drilling and Sampling Methods

The nearshore borings were drilled using wet-rotary techniques. Soil samples were taken continuously up to a depth of 16-ft and at about 5-ft intervals to the completion depth of the boring, as indicated on the boring logs. Detailed descriptions of the soils encountered along with the boring coordinates are presented on the boring logs on Plates 10 and 11. A key identifying the terms and symbols used on the boring logs are presented on Plates 12a and 12b.

Undisturbed samples of soils were generally obtained by hydraulically pushing a 3-inch-diameter, thin-walled tube a distance of about 24 inches. Our field procedure for sampling undisturbed cohesive soils was conducted in general accordance with ASTM D1587, *Standard Practice for Thin-Walled Tube Sampling of Soils*. The soil samples were extruded in the field and visually classified by our field technician. We obtained field estimates of the undrained shear strength of the recovered samples using a calibrated hand-held penetrometer. The field estimates were modified for stiff to hard, over-consolidated natural cohesive soils, as described on Plate 12b. Portions of each recovered soil sample were placed into appropriate containers for transportation to our laboratory for additional geotechnical testing.

2.2.2 Water Depth Measurements

Water depths at each nearshore boring were determined once the pontoon-mounted drill rig was positioned at each location. Water depths were measured by obtaining the distance from the floor of the pontoon to the mudline with an electrical bottom sensor and a piezometer tape. Depth of water measured at each boring location is noted on the boring logs on Plates 10 and 11.

2.2.3 Borehole Completion

No grouting was performed for the nearshore borings. The boreholes were filled with drill mud and cuttings.

3.0 LABORATORY TESTING

This section provides information relating to our laboratory-testing program for this project. The laboratory-testing program for this study was directed primarily toward evaluating the classification properties and undrained shear strength of the subsurface soils. Our laboratory tests were performed in general accordance with the appropriate standards as tabulated at the end of this section.

3.1 Classification Tests

The classification tests included tests for natural water content, liquid and plastic limits (collectively termed Atterberg limits), sieve analysis, and material finer than the No. 200 sieve (percent fines). These tests aid in classifying the soils and are used to correlate the results of other tests performed on samples taken from different borings and/or different depths. The results of the classification tests are recorded on the boring logs on Plates 3 through 11. Results from sieve analyses performed for this study are presented on Plate 13.

3.2 Undrained Shear Strength Tests

We measured the undrained shear strength from selected undisturbed samples of cohesive soils by performing unconfined compression (UC) and unconsolidated-undrained (UU) triaxial tests. The natural water content and dry unit weights were determined as routine parts of the shear strength tests. The results of the laboratory shear strength tests, along with the field estimates of shear strength, are presented on the boring logs on Plates 3 through 11.

3.3 Summary of the Laboratory Tests

Table 3-1 presents a summary of the type and number of laboratory tests performed for this study, as well as the applicable test standards.

Table 3-1. Laboratory Test Quantities and Testing Standards

Laboratory Test	Quantity	Testing Standard
Water Content	31	ASTM D2216
Atterberg Limits	14	ASTM D4318
Sieve Analysis	2	ASTM D6913
Percent Passing the No. 200 Sieve	23	ASTM D1140
Unit Dry Weight	16	ASTM D2166
Unconfined Compression	3	ASTM D2166
Unconsolidated-Undrained Triaxial Compression	12	ASTM D2850

4.0 GENERAL SITE CONDITIONS

The interpreted site and subsurface conditions based on our field exploration, laboratory testing, and our experience are discussed in this section. This section also includes a discussion on the depth-to-water conditions at the time of our study.

4.1 Site Location and Description

The project site is located along the northern coastal boundary of Bolivar Peninsula near Rollover Pass in Galveston County, Texas. A *Vicinity Map* of the site location is presented on Plate 1 and a layout of the site location is presented on the *Plan of Borings* on Plate 2. The site is generally flat and portions of the site are covered with stabilized material and asphalt parking. We understand there are no major structures currently located at the site. An overall site plan of the proposed development is presented in Appendix A.

4.2 Subsurface Soil Conditions

The subsurface soil conditions presented in this report are based on the field investigation and results from laboratory tests conducted as part of this study. The term existing grade as used herein refers to the grade at the time the borings were drilled. A brief description of the encountered subsurface soils is provided in the following sections.

4.2.1 Stratum I

Stratum I consists of about 1 to 6 ft of fill material. Fill material was only encountered in Borings B-1 through B-4 and B-7, and consisted of primarily clay and clayey sand. The percentage of material passing the No. 200 sieve within this stratum range from 24 to 72 percent. We recorded SPT N-values ranging from 2 to 35 blows per foot indicating very loose to dense relative density for the granular fill soils. Field estimates indicate that the undrained shear strength of the cohesive fill soils is approximately 500 psf (firm). Shell fragments were observed within the variable fill material.

4.2.2 Stratum II

This stratum consists of silty/clayey sand, clayey sand and sand extending from below Stratum I (where encountered) to a maximum depth of about 18.5 ft for the onshore borings (B-1 through B-7) and about 5 ft for the nearshore borings (B-8 and B-9). The percentage of material passing the No. 200 sieve ranged from 11 to 43 percent within this stratum. Moisture contents within this stratum range from 19 to 49 percent. We recorded SPT N-values ranging from WHO (weight of hammer) to 32 blows per foot indicating very loose to dense relative density for the granular soils. Low SPT N-values encountered B-5 and B-6 boring locations may be related to very loose dredge material placed behind the existing bulkhead.

4.2.3 Stratum III

Stratum III comprises of natural clay to sandy clay interlayered with clayey sand extending from below Stratum II to a depth of about 60 feet below the existing grade (the maximum depth explored for this study). Moisture contents in this stratum range from 21 to 68 percent. Measured liquid limits for the cohesive soils within this stratum range from 33 to 55 percent, with calculated plasticity indices ranging from 13 to 37 percent. Based on the field and laboratory measurements, undrained shear strengths in the cohesive soils within this stratum generally range from about 0.5 ksf (firm) to 2.6 ksf (very stiff). Recorded SPT N-values ranging from WOH to 22 blows per foot indicating very soft to very stiff materials. A 5-ft thick layer of clayey sand was encountered in Boring B-8 at a depth of 38 feet below the mudline.

Additional information about the soils encountered at each boring location is presented on the boring logs on Plates 3 through 11.

4.3 Depth-to-Water Conditions

As mentioned earlier, the onshore geotechnical borings for this study were drilled using a combination of dry-auger and wet-rotary techniques in an effort to identify the depth-to-water. Free water was first encountered at depths of about 4 to 8 feet below existing grade. Once free-water was encountered, drilling was temporarily halted and additional depth-to-water measurements were made. After a period of approximately 15 to 20 minutes, the water level in the onshore boreholes was observed at depths of about 2 to 3 feet below existing grade.

Please note that short-term depth-to-water observations recorded in open boreholes should not be considered to represent a long-term condition, especially in cohesive soils. The time associated with short-term observations may not be sufficient for the conditions in the open borehole to reach equilibrium. More accurate determinations of groundwater levels are usually made using long-term standpipe piezometer readings. Water levels at the project site will be controlled by the water level at Rollover Pass and the Gulf of Mexico. We recommend that the groundwater level at this site be assumed at the ground surface for design purposes.

4.4 Variations in Subsurface Conditions

Our interpretations of soil conditions, as described in this report, are based on data obtained from our visual observations, sample borings, laboratory tests, and our experience. Although we have allowed for minor variations in the subsurface conditions, our recommendations may not be appropriate for subsurface conditions other than those reported herein. It is possible that some undisclosed variations in soil or groundwater conditions might occur outside the boring locations. We recommend careful observations during construction to verify our interpretations. Should variations from our interpretations be found, we recommend that we be notified and authorized to evaluate what, if any, revisions should be made to our recommendations.

5.0 GRADE SUPPORTED SLAB RECOMMENDATIONS

We understand that grade-supported slabs are planned to support lightly loaded, settlement insensitive structures. The geotechnical investigation performed at the site showed up to 8 ft of fill composed primarily of very loose to dense clayey sand underlain by very loose to medium dense clayey sand and very soft clay down to a depth of about 18 ft below the existing grade. The soil stratigraphy is also very heterogeneous throughout the site. Based on the soil conditions encountered at the project site, we anticipate grade-supported slab foundations will likely experience settlements in the order of up to 4 inches, depending on the load applied. Therefore, we do not recommend using slab-on-grade foundations for the structures at this site, unless the proposed structures are able to tolerate such settlements. Hence, the Owner should be willing to accept risk of damage from total and differential settlements. The risks associated with the use of a conventional slab-on-grade can, however, be significantly reduced provided certain considerations are addressed during the design and construction of the foundation.

The following subsections provide recommendations for the design and construction of slab-on-grade foundations.

5.1 Site Preparation

Proposed slab-on-grade foundations should be placed over at least 24-inches of properly placed and compacted structural clay fill or chemically treated fill (lime-fly ash). The removal and replacement of at least 24 inches of soil below the structure footprint will remove a certain portion of loose and soft soils. The removal and replacement activities should extend to at least 5 feet beyond the edge of the building footprint. Following removal of the upper 24 inches of existing subgrade, and prior to the placement of properly compacted structural clay fill or chemically treated fill (lime-fly ash), the excavation bottom should be proof-rolled with a 20-ton rubber tired vehicle, vibratory roller or equivalent and observed by the Geotechnical Engineer-of-Record or their qualified representative to evaluate the conditions of the subgrade. Note that there might be limited space at some locations at the site, so the proposed equipment should be sized accordingly. In addition, the bottom of the excavation will be close to the existing groundwater level and, therefore, perched water may be expected during the site preparation activities. Weak, wet, and otherwise deleterious soils should be removed and replaced with properly compacted structural clay fill or chemically treated fill (lime-fly ash) clay. The objective will be to create a uniform, low permeable, relatively homogeneous foundation base of low plasticity chemically treated fill (lime-fly ash)/structural clay fill for the slab-on-grade. Recommendations for structural clay fill and chemically treated fill (lime-fly ash) are provided in later sections of the report. We recommend that concrete be placed soon after the construction of the building pad to protect the pad from drying or wetting.

We understand that the Client is considering the option of using crushed stone below the proposed slabs in lieu of structural clay fill or chemically treated fill (lime-fly ash). Crushed stone may be used below the slabs at the site. However, if crushed stone is used, a layer of geotextile (Mirafi

600x or equivalent) should be placed below the crushed stone over the underlying subgrade to act as a separator. Crushed stone should be in accordance with Texas Department of Transportation (TxDOT) Standard Specifications for Construction of Highways, Streets, and Bridges(1) Item 247 and should be compacted to 98 percent of the maximum dry density as determined by TxDOT Test Method Tex-113-E.

5.2 Slab-on-Grade Applied Pressure

Based on the information obtained from borings, fill material was encountered at the site extending to a depth of about 8 ft below the existing grade. We recommend limiting the applied pressure beneath the slab-on-grade foundations supported on structural clay fill, chemically treated fill (lime-fly ash), or crushed stone to 500 psf (allowable). Applied pressures greater than 500 psf will likely require the use of supplemental foundations. Reinforced concrete slabs should be proportioned so that the maximum contact pressure under the various load combinations does not exceed the appropriate allowable net bearing pressure given herein. Note that, due to the presence of fill at the site, slab-on-grade foundations will experience settlements in the order of up to 4 inches, depending on the load applied.

5.3 Settlement Estimates

It should be noted that conventional slab-on-grade foundations construction will still be susceptible to large total and differential settlements expected due to the underlying soft soils. Hence, the Owner should be willing to accept risk of damage if a slab-on-grade is used. Based on the information collected during this study, the recommendations presented in this report, and our experience with similar projects, we expect movements on the order several inches for slab-on-grade foundations at this site after removal and replacement activities.

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[1] Texas Department of Transportation, Standard Specifications for Construction of Highways, Streets, and Bridges, 1993

6.0 DEEP FOUNDATION RECOMMENDATIONS

We understand that current plans are to support the new dock and associated structures on deep foundations. This section presents our soil parameters for axial and lateral capacity and driven timber pile recommendations.

6.1 Soil Parameters

The soil parameters for axial capacity used to develop our deep foundation recommendations are based on the subsurface information obtained from our geotechnical borings for the onshore and nearshore locations. Design soil parameters for both onshore and nearshore locations are presented on Plates 14a and 14b. Plates 15a and 15b presents soil parameters for computing static axial and lateral capacity for the proposed piles. The selected soil parameters are in general accordance with ANSI/API (2011)[2] specifications.

6.2 Driven Timber Pile Recommendations

Based on our review of the information provided by Client, we understand that timber piles with butt and tip of 12 and 7 inches, respectively, are proposed to support the new dock as well as miscellaneous onshore structures. We understand that the top of the driven timber piles will be approximately 8 feet above mudline for the nearshore structures and at existing grade for the onshore structures. The following subsection presents our recommendations for timber piles including static axial capacity, axial group effects, lateral capacity, settlement considerations, and general scour considerations.

6.2.1 Static Axial Capacity

The ***ultimate*** axial capacity, in both compression and tension, of individual, isolated timber piles with butt and tip diameters of 12 and 7 inches, respectively, was computed using the static method of analysis. In this method, the ultimate compressive capacity of a pile is computed as the sum of skin friction acting along the pile surface and end bearing on the pile tip. The weight of the pile is neglected in the computations. We also neglected the top 5 feet of soil below existing grade to account for soil variability, construction disturbance, and scour of surficial granular soils for nearshore piles. The ultimate axial pile capacity curves were computed in general accordance with the API RP 2A (2011) method. Axial capacity values for 40- and 60-ft long timber piles for the nearshore locations are presented on Plates 16a and 16b. Additionally, axial capacity values for 40- and 60-ft long timber piles for the onshore locations are presented on Plates 16c and 16d.

We recommend a factor of safety of 2.0 be applied to the ultimate axial capacity of piles loaded in compression (transient and sustained) and transient tension. A factor of safety of 3.0 should be

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[2] American Petroleum Institute (2011), *Geotechnical and Foundation Design Considerations*, ANSI/ API Recommended Practice 2GEO, 1st Edition.

applied for sustained tension loads. The weight of the pile was neglected in the computation of ultimate tension capacity, but it may be included once the penetration is determined. The buoyant weight of the piles should be used. A factor of safety of 1.2 should be applied to the pile weight.

6.2.2 Axial Group Effects

The overall allowable axial load carrying capacity of a group of timber piles may, in some cases, be less than the sum of the individual allowable capacities. A reduction in the individual pile capacity, to allow for group effects, is usually not necessary for piles having a center-to-center spacing of 3 or more pile diameters. The reduction in individual capacity depends on many factors including the configuration of the group, number of piles in the group, pile size, the depth of installation, and the pile spacing. We recommend timber piles be spaced at least 3 pile diameters (center-to-center) to reduce substantial axial group effects. If piles are spaced closer, we would be pleased to review the design and comment on axial group effects.

6.2.3 Lateral Capacity Analysis

We performed our lateral capacity analyses using the computer software program *LPILE* 2017 developed by Ensoft, Inc. This program uses finite difference numerical techniques to compute lateral deflections and bending moments induced in a pile due to lateral and axial loads applied at the top of the pile. The pile-soil system is modeled as a series of finite segments that represent the pile and the soil. Based on the subsurface soils encountered during our study, we developed the soil parameters used in *LPILE*. Soil resistance is calculated using p-y data developed from a distribution of input soil unit weights and strength parameters.

We evaluated the lateral resistance for a timber pile with butt and tip diameters of 12 and 7 inches, respectively. We analyzed free- and fixed-head conditions with lateral loads that would yield limiting deflections of ¼ inch, ½ inch and 1 inch at the top of the pile for the onshore and nearshore locations. The top of pile is assumed at the existing grade for onshore piles and no more than 8 feet above the mudline for nearshore piles. We used a Young’s Modulus (E) of 1,000 ksi for the driven timber piles based on information provided by Client.

Individual, isolated piles are representative of a free-head condition. A group of piles structurally tied together with a rigid concrete cap is more closely representative of a fixed-head condition. On the basis of the final foundation design, the Structural Engineer-of-Record should determine the representative condition.

Tables 6-1 and 6-2 present the lateral capacity analysis results for isolated, individual timber piles with butt and tip diameters of 12 and 7 inches, respectively. The pile length used in our lateral capacity analysis is 32 feet. Axial and lateral loads refer to loads applied at the pile top. Top deflection refers to pile deflection at the pile top (at existing grade for onshore piles and 8-feet above mudline for nearshore piles).

Table 6-1. Lateral Capacity Analysis Results – Nearshore Locations (8-ft Pile Stick-up)

Pile Size and Type	Pile Top Fixity	Lateral Deflection(1) (inch)	Shear Force at Pile Top (kips)	Maximum Bending Moment (kips-inch)	Depth of Maximum Bending Moment(2) (ft)
Tapered Timber Pile					
12-inch to 7-inch Timber Pile	Free Head	¼	0.08	15	13.0
		½	0.16	25	13.5
		1	0.29	50	13.5
	Fixed Head	¼	0.37	40	0
		½	0.69	75	0
		1	1.26	135	0

Notes:

(1) Lateral deflection at the top of the pile (8-feet above mudline).
(2) The depth of maximum bending moment is referenced from the top of pile.

Table 6-2. Lateral Capacity Analysis Results – Onshore Locations (No Pile Stick-up)

Pile Size and Type	Pile Top Fixity	Lateral Deflection(1) (inch)	Shear Force at Pile Top (kips)	Maximum Bending Moment (kips-inch)	Depth of Maximum Bending Moment(2) (ft)
Tapered Timber Pile					
12-inch to 7-inch Timber Pile	Free Head	¼	1.5	60	4.5
		½	2.2	95	5.0
		1	3.4	150	5.0
	Fixed Head	¼	3.8	160	0
		½	5.7	265	0
		1	8.6	430	0

Notes:

(1) Lateral deflection at the top of the pile (at existing grade).
(2) The depth of maximum bending moment is referenced from the top of pile.

We recommend that the piles be installed to a depth sufficient enough to: 1) develop the axial capacity required to support the proposed structures with a factor of safety as recommended in previous sections, or 2) have a minimum depth to provide sufficient lateral support, whatever length is greater. The minimum penetrations required for lateral capacity of timber piles with butt and tip diameters of 12 and 7 inches, respectively, in nearshore and onshore locations are provided below.

Table 6-3. Minimum Pile Penetrations Based On Lateral Pile Capacity

Pile Type	Location	Pile Diameter (in)	Penetration Below Pile Top (ft)
Tapered Timber Pile	Onshore	12 to 7	18
Tapered Timber Pile	Nearshore	12 to 7	21

6.2.4 Settlement Considerations

A detailed settlement analysis for timber piles was beyond the scope of this study. However, based upon the soil conditions at the site and our experience, we expect consolidation settlements for the driven timber piles designed according to the axial capacity curves on Plates 16a through 16d to be less than about ½ inch. If this estimate of settlement is beyond tolerable limits, we would be pleased to perform a detailed settlement analysis to evaluate settlement movements on a case-by-case basis once pile details have been finalized.

Groups of piles will likely settle more than individual piles subjected to the same load per pile. The increase in settlement from individual piles to groups is generally negligible for groups of piles than are less than about 3-by-3. The settlement of groups is dependent on several variables including: dimension of the group, the pile length, the sustained structural load, and the compressibility characteristics of the foundation soils. If requested, we would be pleased to perform a detailed group settlement analysis on a case-by-case basis under separate cover.

6.2.5 General Scour Considerations

The design of deep foundations to support structures over water should consider the long-term effects scour action around the foundation elements. Accordingly, the proposed driven timber piles to support the new dock over water should be sized to account for potential soil loss associated with scour action over the anticipated life-span of the dock. As mentioned earlier, the nearshore borings (B-8 and B-9) showed approximately 5 feet of clayey sand underlaid by natural cohesive soils.

Based on historical data of hydraulic activity along the Texas Gulf shoreline in Galveston County and the foundation soils encountered during our study, we assume in our analyses that the potential magnitude of soil loss from scour at the site will on the order of 5 feet below the mudline. We recommend that a Hydraulic Engineer be consulted to evaluate and provide final recommendations for the design depth of scour at this site. If it is determined that the design scour depth is greater than 5 feet, we should be notified to modify our recommendation presented in this report.

7.0 SOIL PARAMETERS FOR BULKHEAD STRUCTURE

Based on conversations with the Client, we understand that current plans will include the construction of a bulkhead structure east of the new concrete boat ramp. A detailed design for a steel sheet pile bulkhead was outside our scope of work. The soil parameters for sheet pile wall analysis and design are provided in the following table. These parameters were developed based on field and laboratory test results for Borings B-7 and B-8, drilled along the location of the proposed sheet pile.

Table 7-1. Design Soil Parameters for Sheet Pile Wall (Borings B-7 and B-8)

Soil Description	Depth(1) (ft)	Total Unit Weight (pcf)	Submerged Unit Weight (pcf)	Short Term (Undrained) Condition		Long-Term (Drained) Condition	
				Cohesion (psf)	Friction Angle (degree)	Cohesion (psf)	Friction Angle (degree)
Granular Backfill(2)	Varies	120	57.6	0	30	0	30
Existing Fill: Clayey Sand	up to 8	115	52.6	0	19	0	19
Clayey Sand	8 to 12	115	52.6	0	20	0	20
Clay	12 to 20	122	59.6	700	0	30	15
Clay	20 to 30	120	57.6	900	0	50	16

Notes:

(1) Depth referenced from the top of the steel sheet pile bulkhead.
(2) Granular backfill to be used behind the proposed sheet pile wall, where engineered and designed.
(3) Angle of wall friction (steel pile) can be computed as 0.5 x friction angle for sand. Angle of wall friction should be neglected in clays.

Earth pressure coefficients for the soils encountered at the site should be calculated with the following equations.

$$K_o = 1 - \sin\phi\,; \qquad K_a = \tan^2[45 - (\phi/2)]; \qquad K_p = \tan^2[45 + (\phi/2)]$$

where, K_0 = coefficient of earth pressure at-rest
K_a = coefficient of active earth pressure
K_p = coefficient of passive earth pressure
ϕ = friction angle, degrees

Surcharge loads should be included in the design of the bulkhead. We recommend that the retention system be designed based on the critical design case. Soils and hydrostatic water pressures behind the walls will impose a triangular stress distribution on the walls below-grade, while surcharge loads will impose a rectangular stress distribution. Water should be assumed to be at final grade or the slope crest to account for possible high groundwater levels. We understand that current plans include the construction of a French drain immediately behind the sheet pile wall, weeping into the boat basin. If sufficient drainage is provided, groundwater level may be assumed to be 2 feet above the elevation of the drain pipes. Hydrostatic water pressures should be computed using a unit weight of 62.4 pcf. Lateral earth pressures resulting from surcharge loads may be calculated using a coefficient of lateral earth pressure of 0.7.

The cohesion of the soil can be considered in computing the earth pressure of soil. The cohesion of soils should be ignored if the resulting net pressure has a negative value. However, we recommend neglecting the cohesion in computing the earth pressure because the earth pressure theories are not able to consider the actual behavior of the cohesive soils, including creep, expansive nature, potential tension cracks, poor drainage, etc. An appropriate factor of safety for the lateral earth pressures should be applied depending on the application and analysis used.

When determining sheet pile wall penetrations, we recommend using a factor of safety of 2.0 for passive pressures under undrained (short-term) loading conditions, and a factor of safety of 1.5 for passive pressures under drained (long-term) loading conditions. For determining maximum bending moments and anchor loads, a factor of safety of 1 may be used for the drained loading condition. We expect the Structural Engineer will apply adequate factors of safety to the structural design of the wall. We also recommend that, when determining loads imparted on the sheet pile wall, appropriate surcharges are considered behind the wall.

8.0 PARKING LOT RECOMMENDATIONS

We understand that a parking lot will be constructed as a part of the project. Portland cement concrete and asphaltic concrete pavements are commonly used and either pavement type may be used for automobile parking areas. However, we do not recommend asphaltic concrete pavements in truck and heavy traffic areas because of the potential for shoving and rutting, particularly during hot summer weather conditions.

8.1 Subgrade Preparation

The performance of pavement/ parking lot ultimately depends on the underlying subgrade. Subgrade preparation for pavements should include clearing and stripping all organic material, debris, and other deleterious materials within the footprint of the pavements. The subgrade preparation should extend laterally at least 5 feet beyond the edges of the pavements. After removing deleterious materials and stripping, the exposed subgrade should be proof-rolled with a fully loaded dump truck or other heavy (20-ton), rubber-tired vehicle and observed by the Geotechnical Engineer-of-Record or their qualified representative to evaluate the condition of the subgrade. Depending on the condition of the existing pavement parking area, it may need to be removed and replaced according with the recommendations provided in this section.

Areas of the subgrade that are observed to be soft, wet, weak, or contain deleterious materials should be over-excavated to expose competent soils. We recommend that the prepared subgrade directly below the pavement section consist of at least 12 inches of chemically treated fill (lime-fly ash). Areas of the subgrade in which pumping or significant deflections are observed should also be over-excavated to expose competent soils. Over-excavated areas should be backfilled with properly placed and compacted chemically treated fill (lime-fly ash). Recommendations for chemically treated fill (lime-fly ash) are presented in following sections of this report.

8.2 Pavement/ Parking Lot Section

A detailed pavement design was beyond the scope of this investigation since design traffic loads have not been developed at this time. The following table presents a typical section for parking area based on our experience with similar subsurface conditions. This section is not based on a specific loading condition (e.g., equivalent single axle load) or pavement life expectancy. If desired, we can perform a detailed pavement design based on the type and frequency of vehicles anticipated.

Table 8-1. Typical Pavement Section

Type and Recommended Use	Material	Thickness	Reference or Specification
Flexible Pavement (parking areas for cars and light duty trucks)	Hot-Mix Asphaltic Concrete over	3 inches	TxDOT Item 340 Type C or D
	Crushed Limestone Base over	8 inches	TxDOT Item 247 Type A, Grade 1
	Stabilized Subgrade	12 inches	See Text

8.2.1 Stabilized Subgrade

The existing subgrade soils should be chemically treated with a combination of both lime and flyash. The chemically treated fill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D698. Additional details for lime-fly ash stabilization is presented in Section 9.0 *Construction Considerations*.

8.2.2 Crushed Limestone

The crushed limestone base should be in accordance with Texas Department of Transportation (TxDOT) Standard Specifications for Construction of Highways, Streets, and Bridges(3) Item 247. Crushed limestone should be compacted to 98 percent of the maximum dry density as determined by TxDOT Test Method Tex-113-E.

8.2.3 Asphalt

Hot mix asphaltic concrete (HMAC) should be placed in accordance with TxDOT Item 340. The HMAC should be either a Type D or Type C surface course mix. The asphaltic concrete should be compacted to between 91 and 95 percent of the theoretical density as described by TxDOT Item 340 Specifications.

8.3 Drainage

The importance of drainage to the proper operation and function of any pavement cannot be overemphasized. The pavement and subgrade surface should be raised above adjacent grade and properly sloped into drainage inlets or lateral ditches. Water should not be allowed to pond on or adjacent to the pavement whereby the subgrade may become saturated. If the pavement sublayers do become saturated, the bearing capacity will be greatly reduced and the useful life of the pavement will be decreased. Periodic inspections and repair of cracks in pavement sections should be performed as part of routine facility maintenance.

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[3] Texas Department of Transportation, Standard Specifications for Construction of Highways, Streets, and Bridges, 1993

9.0 CONSTRUCTION CONSIDERATIONS

Recommendations for subgrade preparation, fill selection and placement, shallow open-cut excavations, fill selection and placement, pile installation considerations, groundwater control, and construction monitoring are included in this section.

9.1 Subgrade Preparation

We understand that current plans do not involve changes to the existing grade at the site. The degree of site preparation to be completed will depend on the general contractor's requirements. General site preparation should include clearing and stripping of all significant vegetation, organic materials, debris, and other deleterious materials. Soft spots encountered should be removed and replaced with properly compacted structural (select) clay fill or chemically treated fill (lime-fly ash).

Positive drainage should be provided away from structures and pavements during and after construction activities. We believe that it is essential to establish and maintain adequate site drainage. This should reduce access problems and delays, as well as help other earthwork-related activities. Construction traffic, including proofrolling, should be avoided during extended periods of wet weather.

We recommend that proofrolling be performed using a fully loaded dump truck or water truck with a weight of at least 20 tons and a tire pressure of at least 70 psi. We do not recommend using off-road earth moving equipment (*i.e.* loaders and scrapers), compactors, or track-mounted vehicles (i.e. bull dozers and front-end loaders) for proofrolling. Proofrolling specifications should provide for rut depths less than 1 inch and no visual evidence of pumping. Unsuitable soils should be removed and replaced with properly placed and compacted structural clay fill or chemically treated fill (lime-fly ash). We recommend scheduling these activities during a relatively dry period. We do not recommend that subgrade preparation activities begin immediately after or during a significant rain event. It may be necessary to wait for the site to dry prior to restarting subgrade preparation activities depending on the effectiveness of onsite drainage.

9.2 Shallow Open-Cut Excavations

We expect that shallow excavations will be required during construction. Foundation soils exposed by the excavations should be protected from disturbance due to construction activities. We recommend that positive surface drainage away from the excavation should be established to prevent surface runoff from either flooding the excavation or ponding within the excavation.

Excavations should be designed in accordance with all applicable local, state, and federal trenching regulations including the Federal Occupational Safety and Health Administration (OSHA) requirements for excavations presented in 29 CFR Part 1926, Subpart P, Excavations. Based on the results of field and laboratory tests, and our interpretation of OSHA regulations, the surficial clayey sand soils encountered in our borings at the time of our field exploration are classified as Type C soils. Federal OSHA regulations do not generally require shallow excavations to depths of

5 feet or less to be sloped back or braced. However, if sloughing and caving is experienced, we recommend the slopes should be cut back to a stable condition. Excavations deeper than 5 feet should be braced or sloped back. Sloped excavations should be no steeper than 1.5-horizontal on 1-vertical in Type C soils. Flatter slopes should be used if sloughing or raveling is observed. Excavations in structural fill should also conform to Type C requirements.

9.3 Groundwater Control

Groundwater or perched water was encountered at very shallow depths (within the top few feet) at the project site. The groundwater level also fluctuates with rainfall and water level in the adjacent water bodies. Therefore, surface water and groundwater control should be established early at the site and maintained throughout construction and operation.

The Contractor should be prepared to provide groundwater control particularly if excavations extend below the groundwater level. For foundation excavations in fill material and/or granular soils, seepage flow into the excavations may be significant. Sumps and pumps or a well point system will be required to lower groundwater levels to a point where excavations can be performed safely, and in the dry. We recommend that prior to excavation the groundwater level be measured. The groundwater level should be lowered and maintained at a depth of at least 3 ft below the bottom of the excavation. Groundwater level should be maintained at this level until the exposed subgrade is brought to the design grade.

We understand that current plans are to construct a new concrete boat ramp as part of the proposed Lauderdale recreational facility. In order to allow for the construction of the new boat ramp, the bay area comprising of the exiting rock jetty to the west and the rip-rap covered bank to the east, will require dewatering. Based on information provided by Client, the water level will need to be lowered by approximately 4 feet in this area. A dewatering system will likely include a temporary sheet pile system or sandbag walls to prevent water from the canal to enter the construction area. If a sheet pile system is used, sheet piles should fully penetrate the surficial clayey sand of Stratum II (about 5-feet thick) and extend at least 2 feet into the cohesive soils of Stratum III. We anticipate a sheet pile penetration of at least about 7 feet will be needed. Information of the composition of the existing rip rap jetty was not available at the time of this report. It is likely that the materials used to build the jetty are highly permeable and, therefore, the sheet pile system or sandbag wall would likely need to extend along the jetty to prevent water from entering the construction area from the canal through the rip rap jetty. We recommend a detailed dewatering plan be developed including seepage and stability analyses. We can provide further recommendations for groundwater control, if needed.

9.4 Fill Material and Placement

The following sub-sections discuss our recommendations for fill materials, including placement and compaction, to be used for this project. (1) Structural (Select) Clay Fill, (2) Lime-Fly Ash Stabilization, (3) Crushed Stone Fill, (4) Granular Fill, and (5) General Fill.

9.4.1 Structural (Select) Clay Fill.

We recommend using low plasticity cohesive soils for structural clay fill. Structural clay fill should have a liquid limit of less than 45 (preferably less than 35), a plasticity index between 8 and 20, and at least 60 percent of the material finer than the No. 200 Sieve. Structural clay fill should be free of deleterious matter and should have an effective clod diameter less than 3 inches.

Prior to the start of structural clay fill placement, representative samples should be collected and characterized in terms of Atterberg Limits, gradation, and specific gravity. Structural clay fill should be placed in 6- to 8-inch-thick loose lifts and uniformly compacted at a moisture content of optimum to 4 percent “wet” of optimum and to at least 95 percent of the maximum dry density as determined from a field moisture-density curve that is specific to the site compactor and soil combination, as verified with field moisture-density data. Structural clay fill should be compacted by equipment that provides a “kneading” compaction with an approved tamping or padfoot roller with weight and horsepower equivalent to or greater than a CAT 563 class compactor.

Adjacent to foundations, piping, utilities, or other structural features and confined areas, structural clay fill should be placed in 4-inch thick loose lifts and compacted using hand-operated compaction equipment at a moisture content of optimum to 4 percent “wet” of optimum and to at least 95 percent of the maximum dry density representative of ASTM D698 (Standard Proctor), normalized to represent the laboratory line-of-optimums for structural clay fill.

If wet weather or extended dry periods deteriorate the exposed surface whereby a good bond cannot be formed between successive lifts, the Contractor should prepare the surface as necessary. This preparation may include removing or scarifying the top couple of inches of the underlying material before placing the next lift.

9.4.2 Lime-Fly Ash Stabilization

The near-surface subgrade soils at the project site primarily include sandy lean clay and clayey sand soils. In some areas, based on our study, these soils have a plasticity (PI) of less than 25 and less than 60 percent of the material finer than the No. 200 Sieve. Consequently, we recommend using a combination of lime and fly ash to stabilize these soils. Laboratory tests should be performed at the time of construction to determine the optimum lime-fly ash content and the ratio to be applied to the subgrade soils. We estimate about 3 percent lime and 6 percent fly ash by dry weight may be required to stabilize the onsite sandy lean clay and clayey sand soils. Generally, the recommended ratio of lime to fly ash is between 1 to 2 and 1 to 3. The actual amount of lime-fly ash required will vary depending on the type of fly ash available, the gradation of the fill soils, and the plasticity of the sandy lean clay and clayey sand soils. Fly ash should conform to the requirements of ASTM C 618 and meet the following requirements: 1) have a minimum CaO content of 20 percent, 2) loss on ignition should not exceed 3 percent, and 3) contain no lignite.

The lime-fly ash stabilized soil subgrade should be thoroughly mixed and then recompacted to 95 percent of standard Proctor maximum dry density (ASTM D 698). Mixing should be performed using an approved single-pass or multiple-pass rotary speed mixer to obtain a homogenous mixture. The moisture-density relationship should be established based on a material sample obtained from the on-site silty soils after stabilization with lime-fly ash has taken place. The contractor should conduct operations to reduce the elapsed time between mixing and compacting within 6 hours after adding and mixing the lime-fly ash into the soil. Compaction should be performed with approved heavy pneumatic or vibrating rollers, or a combination of tamping rollers and light pneumatic rollers.

9.4.3 Crushed Stone Fill

Crushed stone fill may be used to backfill below the proposed slab foundations, given that an appropriate barrier, such a layer of geotextile (Mirafi 600x or equivalent), is placed between the crushed stone and over the underlying bedding subgrade. Crushed stone fill should be placed in loose lifts not more than 6- to 8-inches and uniformly compacted to 98 percent of the maximum dry density at a moisture content within 2 percent “dry” to 2 percent “wet” of the optimum moisture content as determined by TxDOT Test Method TX-113-E. The crushed stone base should generally have the grain size characteristics as presented in Table 9.1.

Table 9-1. Grain Size Characteristics of Crushed Stone

U.S. Sieve Standard Size	Percent Passing
2 1/2 inch	100
1 3/4 inch	90 to 100
7/8 inch	65 to 90
3/8 inch	35 to 70
No. 4	25 to 55
No. 40	10 to 35

9.4.4 Granular Fill

Granular Fill can consist of crushed stone or sand with a maximum particle size of 4 inches and no more than 10 percent passing the No. 200 sieve. The material shall consist of sound particles, which are angular and not rounded. Numerous gradations will be applicable, however the material should be well graded and have sufficient fines to fill voids between larger particles. The gradation of the selected granular fill should be approved by the Geotechnical Engineer. Granular fill should be placed in lifts no greater than 8-inches, and at a moisture content at a workable level. Granular fill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1577 using a vibratory roller.

9.4.5 General Fill

General fill may be used for areas outside the footprints of structures/equipment, foundations, and pavements. General fill consists of cohesive fill soils not conforming to the compacted clay fill specifications. General fill soils must still be compacted to at least 95 percent of ASTM D698. Cohesive soils used as general fill should have a plasticity index less than 60. A higher Liquid Limit and Plasticity Index will inherently make compaction of general fill to 95 percent of ASTM D698 difficult, specifically relative to meeting moisture requirements. Proper fill preparation (disking, drying, and placement) will be critical for some very wet soils to achieve the required compaction for general fill.

9.5 Pile Installation Considerations

Recommendations for driven timber piles are provided in this section. We have included pile drivability, pile driving specifications, pile driving equipment, installation, pile driving records, and pile load tests.

9.5.1 Pile Drivability

Previous experience driving piles at the site should be considered in selection of the pile driving hammer and equipment. We also recommend that consideration be given to performing a *Wave Equation* analyses to select the proper hammer and cushioning. We are available to provide such geotechnical consultation once the final design has been developed. We also recommend that consideration be given to using fixed leads during pile driving operations.

9.5.2 Pile Driving Specification

The design engineer, in conjunction with the Geotechnical Engineer-of-Record, or their qualified representative, should prepare detailed pile driving specifications. The specifications should cover the project requirements for furnishing and installing the piles including the scope of services, necessary submittals, piling details, equipment requirements, installation requirements and tolerances, capacity evaluation, and construction records. The specification should require the contractor to submit a complete package detailing the proposed piling equipment and installation procedures for approval prior to mobilization to the site. As noted above, the complete package should also include the results of a wave equation analysis to evaluate the proposed pile-driving hammer and cushion system prior to approval for mobilization to the site.

We recommend that the specification establish a pile-driving criterion to clearly define the required pile capacities, pile penetrations, and/or final driving resistance for acceptance. The results of the wave equation analysis should be used to establish the pile-driving criterion. Requirements for pile load tests and capacity evaluation should be stated. The specification should require the contractor to notify the Geotechnical Engineer-of-Record, or their qualified representative, of any changes to the pile driving equipment and methods so that the pile-driving criterion can be adjusted, if necessary. Remedial measures should be presented to address piles not achieving the specified criterion, out of tolerance piles, or piles with questionable driving records.

9.5.3 Pile Driving Equipment

Production piles should be driven using a hammer of adequate size in as nearly a continuous operation as feasible, without interruption, if possible. Pile driving hammers may be diesel, steam, or air operated. The use of a drop hammer, with a light ram and a large stroke, is discouraged since this type of hammer can produce exceedingly high and damaging stresses.

As previously stated, we recommend that the contractor perform a wave equation analysis to evaluate the proposed pile driving hammer and cushion system prior to approval for mobilization to the site. The results of the wave equation analysis should be used to demonstrate that the proposed hammer has sufficient energy to install the piles to the required capacity and/or penetration, and that the hammer is properly cushioned to avoid structurally damaging the piles. To avoid damaging the pile and/or pile driving equipment, refusal criteria should be determined and agreed upon by all parties involved prior to the start of actual pile driving.

9.5.4 Driven Pile Installation

Production piles should be installed to a penetration criterion. The penetration criteria should be in conjunction with the pile driving criteria, to avoid pile damage. The selection of a particular length and particular criteria depends on the pile size, available length, and capacity requirements, in addition to the soil properties. We recommend retapping selected production piles periodically to determine if the driving resistance and pile capacity increase or decrease with time.

9.5.5 Pile Driving Records

An *independent* inspector should keep an accurate and detailed driving log during production driving operations. The log should provide a complete record of hammer blows per foot of penetration from the initial to the final blow for each pile installed. The record for each pile should also include the driving date, pile information, hammer information, cushion information, hammer and compressor operation information, ground and pile tip elevations, records of pre-drilling and/or retapping, and notes on installation delays, problems, or unusual occurrences.

9.5.6 Pile Load Tests

We recommend that design pile capacities be verified during installation by dynamic methods utilizing a Pile Driving Analysis (PDA). PDA testing can verify hammer performance, driving stresses, hammer-to-pile alignment, pile damage, and pile capacity during driving. It should be performed at the end of driving and after soil set-up is allowed to occur. The Geotechnical Engineer-of-Record or their qualified representative should be consulted to develop a PDA testing program.

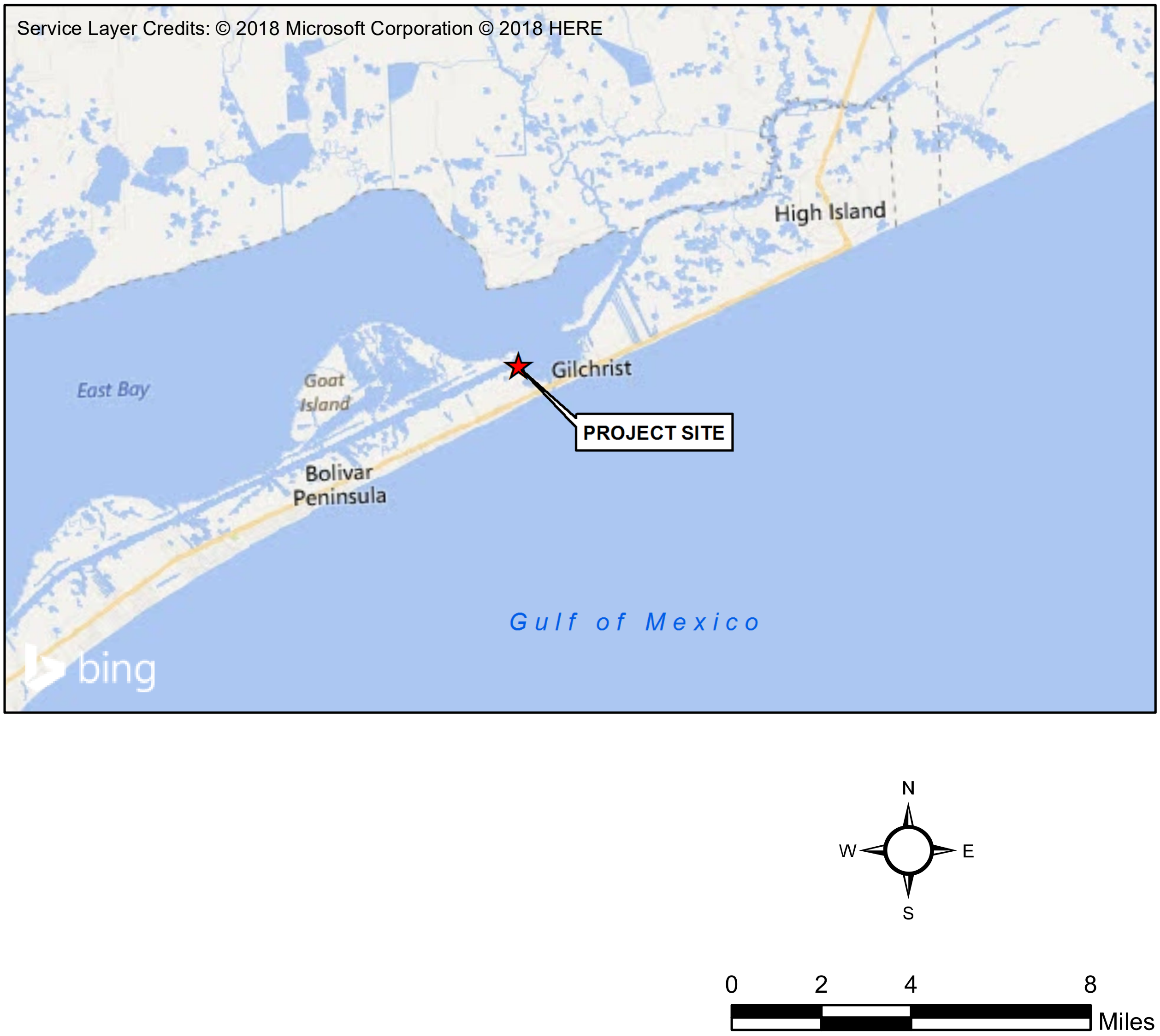
9.6 Construction Monitoring

We recommend that the Geotechnical Engineer-of-Record, or their qualified representative, be present on site during construction to observe and monitor construction activities. Construction monitoring performed by qualified personnel *independent* of the Contractor is recommended because the performance of foundations constructed during this project will be directly related to the Contractor's adherence to the recommendations presented in this report and to the specifications prepared by the Designer. Additionally, unanticipated soil and/or groundwater conditions may be encountered during construction. Qualified geotechnical personnel observing construction on-site can monitor construction activities and may aid in recognizing unanticipated subsurface conditions and reconciling these conditions with design recommendations.

Construction monitoring should be performed during the installation of foundations to verify the suitability of the subgrade soil for foundation support and to observe foundation installation. During the foundation installation and construction phases, we can provide material testing and surveillance to: 1) observe compliance with the design concepts, specifications, and recommendations; 2) observe subsurface conditions during construction; and 3) perform quality control tests including performing PDA services.

* * *

ILLUSTRATIONS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VICINITY MAP

GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

R:\04100\2017 Projects\0100-0199\04.10170131 - Lauderdale Dock - HNTB\00_GIS\MXD\Plate-01_Vicinity Map.mxd, 3/1/2018, t.serrano


Service Layer Credits: © 2018 Microsoft Corporation © 2018 DigitalGlobe ©CNES (2018) Distribution Airbus DS © 2018 HERE
Coordinate System: NAD 1983 2011 StatePlane Texas South Central FIPS 4204 FtUS
Projection: Lambert Conformal Conic
Datum: NAD 1983 2011
False Easting: 1,968,500.0000
False Northing: 13,123,333.3333
Central Meridian: -99.0000
Standard Parallel 1: 28.3833
Standard Parallel 2: 30.2833
Latitude Of Origin: 27.8333
Units: Foot US

LEGEND

● GEOTECHNICAL BORING

NOTE:
1. BORING LOCATIONS ARE APPROXIMATE.

PLAN OF BORINGS

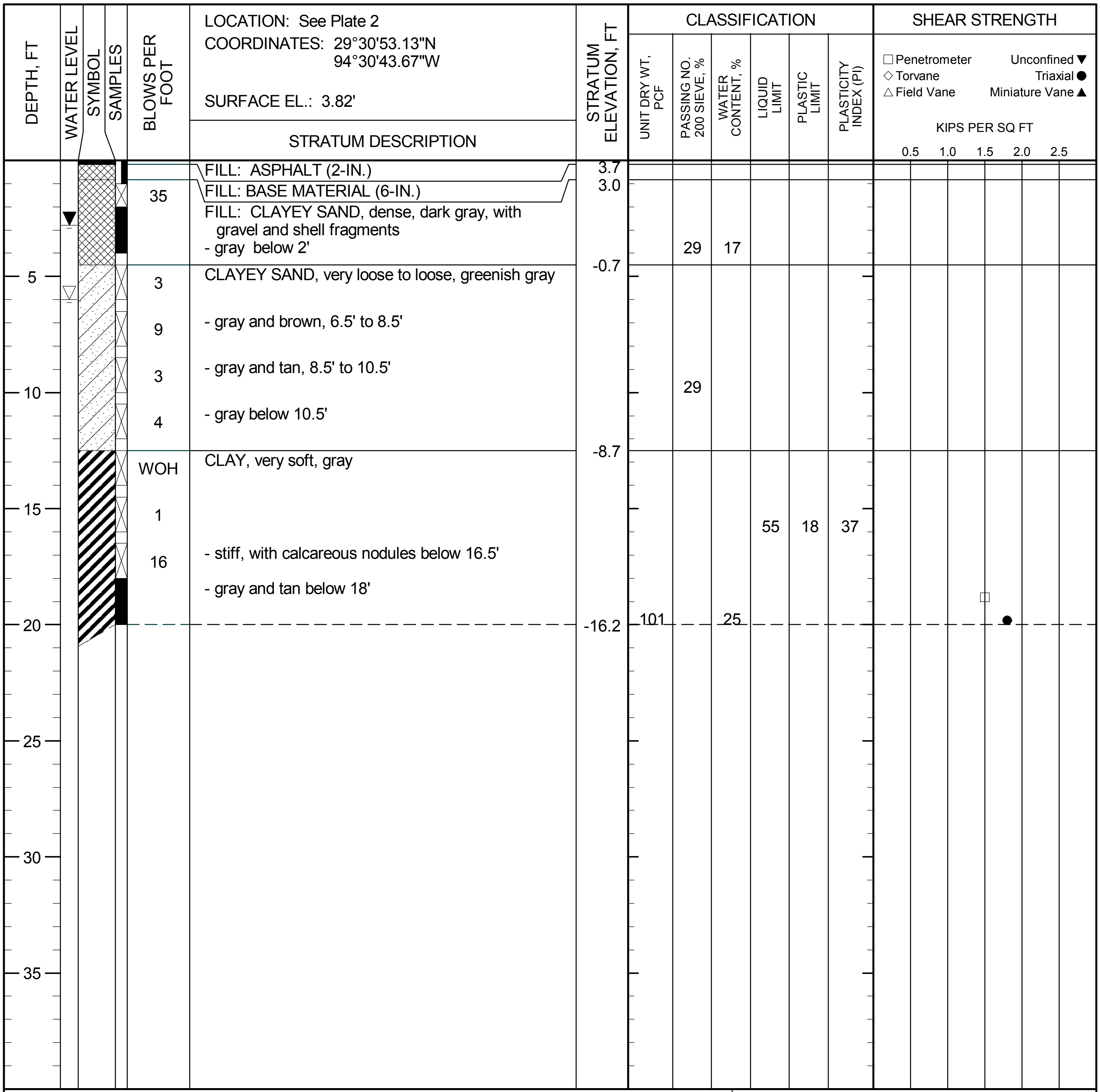
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

PLATE 2

R:\04100\2017 Projects\0100-0199\04.10170131 - Lauderdale Dock - HNTB\00_GIS\MXD\Plate-02_Plan of Borings.mxd, 5/18/2018, t.serrano

LOCATION: See Plate 2
COORDINATES: 29°30'53.13"N
94°30'43.67"W

SURFACE EL.: 3.82'



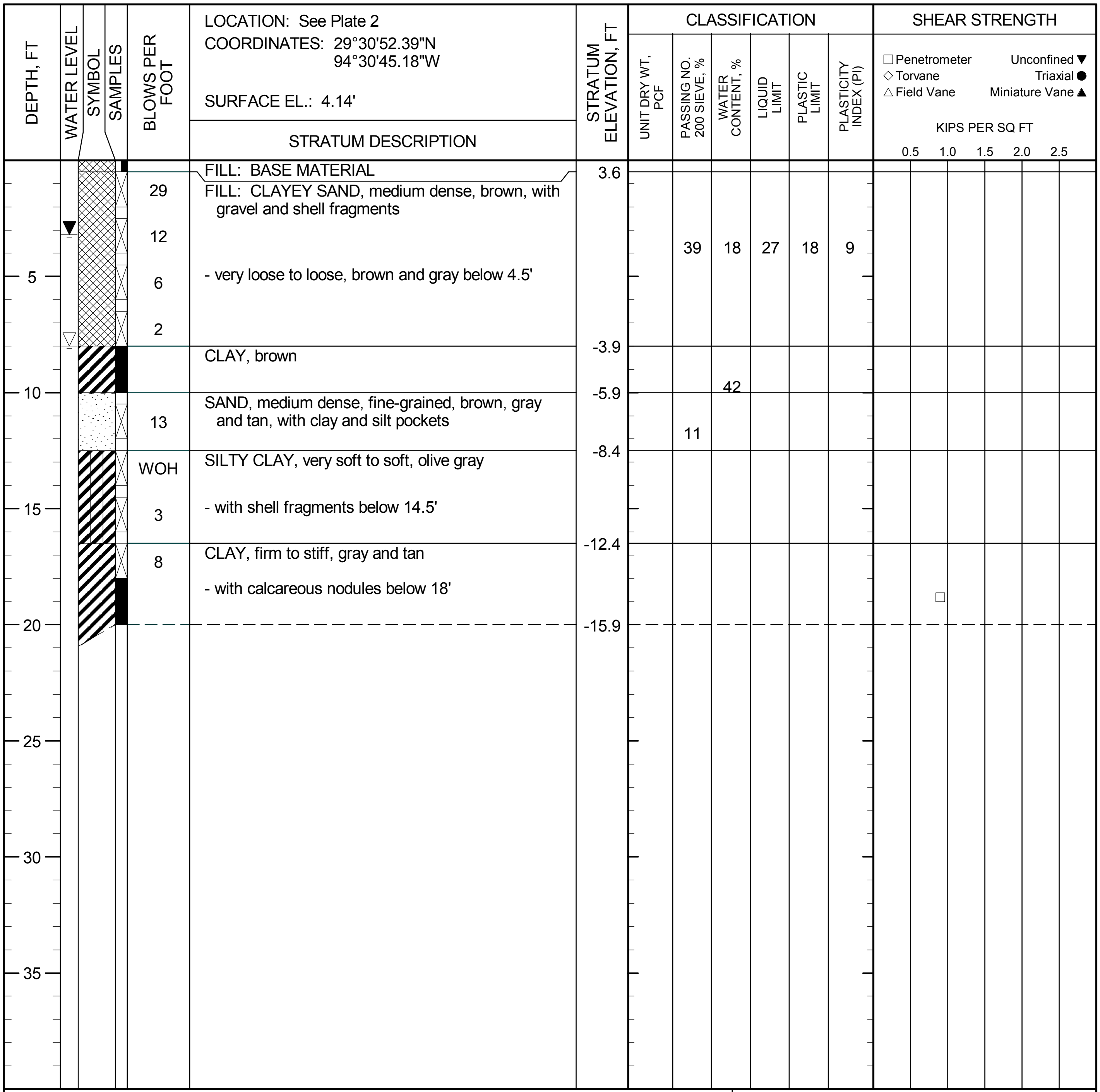
BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
	FILL: ASPHALT (2-IN.)	3.7						
35	FILL: BASE MATERIAL (6-IN.)	3.0						
	FILL: CLAYEY SAND, dense, dark gray, with gravel and shell fragments - gray below 2'	-0.7		29	17			
3	CLAYEY SAND, very loose to loose, greenish gray							
9	- gray and brown, 6.5' to 8.5'							
3	- gray and tan, 8.5' to 10.5'			29				
4	- gray below 10.5'	-8.7						
WOH	CLAY, very soft, gray							
1						55	18	37
16	- stiff, with calcareous nodules below 16.5'							
	- gray and tan below 18'	-16.2	101		25			

NOTES:
1. ▽: Water First Noticed. ▼: Depth To Water after 20 minutes.
2. Boring coordinates were obtained with a hand-held GPS device.
3. WOH - Split-spoon sampler advanced due to the weight of the hammer.
4. Terms and symbols defined on Plates 12a and 12b.

DATE: February 9, 2018
TOTAL DEPTH: 20'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 8'
WET ROTARY: 8' to 20'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

R:\04100\2017 PROJECTS\0100-0199\04.10170131 - LAUDERDALE DOCK - HNTB\00_GIS\GINT\04.10170131.GPJ 04.10170047 BORING LOG 5/9/2018

LOG OF BORING NO. B-1
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS



LOCATION: See Plate 2
COORDINATES: 29°30'52.39"N 94°30'45.18"W
SURFACE EL.: 4.14'

DEPTH, FT	WATER LEVEL	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH, KIPS PER SQ FT
			FILL: BASE MATERIAL	3.6							
		29	FILL: CLAYEY SAND, medium dense, brown, with gravel and shell fragments								
	▼	12				39	18	27	18	9	
5		6	- very loose to loose, brown and gray below 4.5'								
	▽	2									
			CLAY, brown	-3.9			42				
10		13	SAND, medium dense, fine-grained, brown, gray and tan, with clay and silt pockets	-5.9		11					
		WOH	SILTY CLAY, very soft to soft, olive gray	-8.4							
15		3	- with shell fragments below 14.5'								
		8	CLAY, firm to stiff, gray and tan	-12.4							
			- with calcareous nodules below 18'								☐
20				-15.9							
25											
30											
35											

SHEAR STRENGTH legend:
☐ Penetrometer — Unconfined ▼
◇ Torvane — Triaxial ●
△ Field Vane — Miniature Vane ▲

KIPS PER SQ FT: 0.5, 1.0, 1.5, 2.0, 2.5

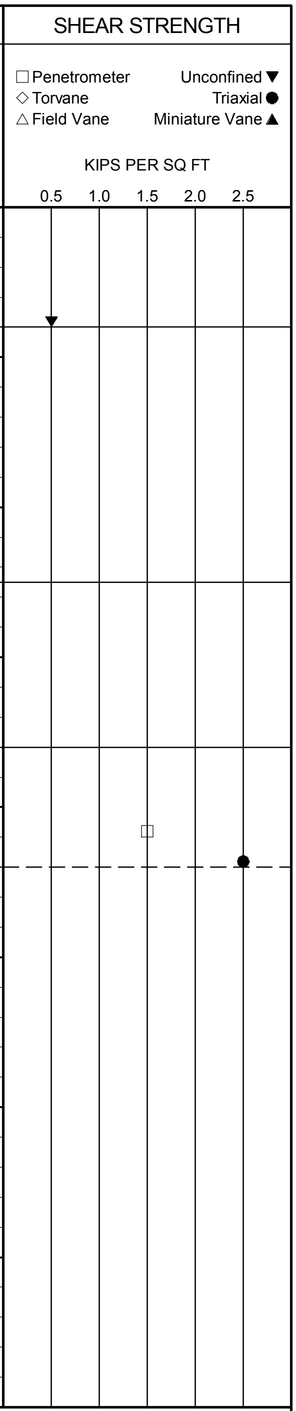
NOTES:
1. ▽: Water First Noticed. ▼: Depth To Water after 15 minutes.
2. Boring coordinates were obtained with a hand-held GPS device.
3. WOH - Split-spoon sampler advanced due to the weight of the hammer.
4. Terms and symbols defined on Plates 12a and 12b.

DATE: February 9, 2018
TOTAL DEPTH: 20'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 10'
WET ROTARY: 10' to 20'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-2

GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

DEPTH, FT	WATER LEVEL	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: 29°30'53.43"N 94°30'46.03"W SURFACE EL.: 4.12' STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
		8	FILL: CLAY, firm, brown, with sand and roots								
	▼		- dark brown and gray, with shell fragments below 2'		86	72	35	25	18	7	▼
5	▽	14	CLAYEY SAND, loose to medium dense, brown	0.1							
		6	- dark gray and black, with silt pockets below 6.5'								
10		3	- very loose at 9'			35	32	26	19	7	
		12									
		WOH	SILTY CLAY, very soft, dark gray, with shell fragments	-8.4							
15		WOH									
		2	- gray below 16.5'								
		10	CLAY, stiff, gray and tan	-13.9							
20			- very stiff below 21'	-17.9	107		21				□ ●
25											
30											
35											



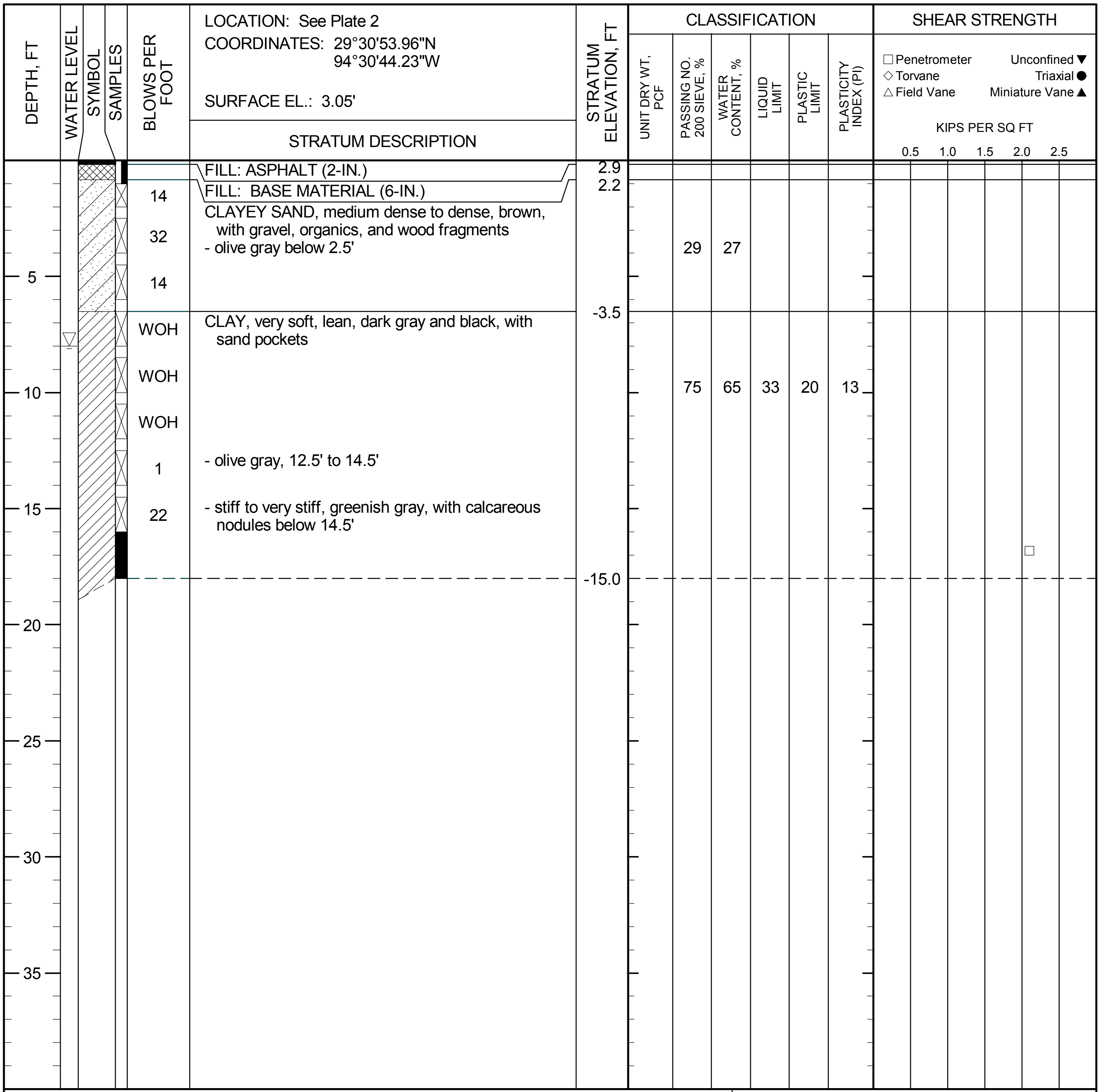
NOTES:

1. ▽: Water First Noticed. ▼: Depth To Water after 20 minutes.
2. Boring coordinates were obtained with a hand-held GPS device.
3. WOH - Split-spoon sampler advanced due to the weight of the hammer.
4. Terms and symbols defined on Plates 12a and 12b.

DATE: February 9, 2018
TOTAL DEPTH: 22'
CAVED DEPTH: Not Available
DRY AUGER: Surface to 8'
WET ROTARY: 8' to 22'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-3
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°30'53.96"N
94°30'44.23"W
SURFACE EL.: 3.05'



DEPTH, FT	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		FILL: ASPHALT (2-IN.)	2.9						
	14	FILL: BASE MATERIAL (6-IN.)	2.2						
	32	CLAYEY SAND, medium dense to dense, brown, with gravel, organics, and wood fragments - olive gray below 2.5'			29	27			
5	14								
	WOH	CLAY, very soft, lean, dark gray and black, with sand pockets	-3.5						
10	WOH				75	65	33	20	13
	WOH								
	1	- olive gray, 12.5' to 14.5'							
15	22	- stiff to very stiff, greenish gray, with calcareous nodules below 14.5'							
			-15.0						

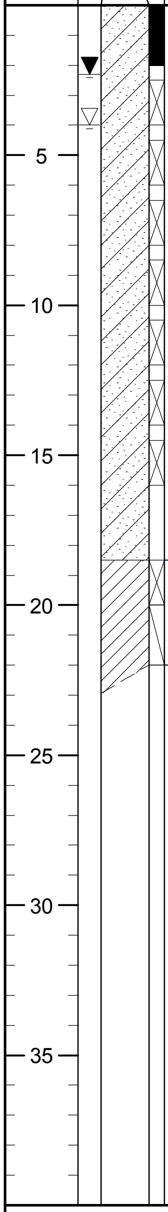
NOTES:

1. ▽: Water First Noticed.
2. Boring coordinates were obtained with a hand-held GPS device.
3. WOH - Split-spoon sampler advanced due to the weight of the hammer.
4. Terms and symbols defined on Plates 12a and 12b.

DATE: February 9, 2018
TOTAL DEPTH: 18'
CAVED DEPTH: 3.9'
DRY AUGER: Surface to 10'
WET ROTARY: 10' to 18'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-4
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°30'54.33"N
94°30'46.51"W
SURFACE EL.: 4.10'



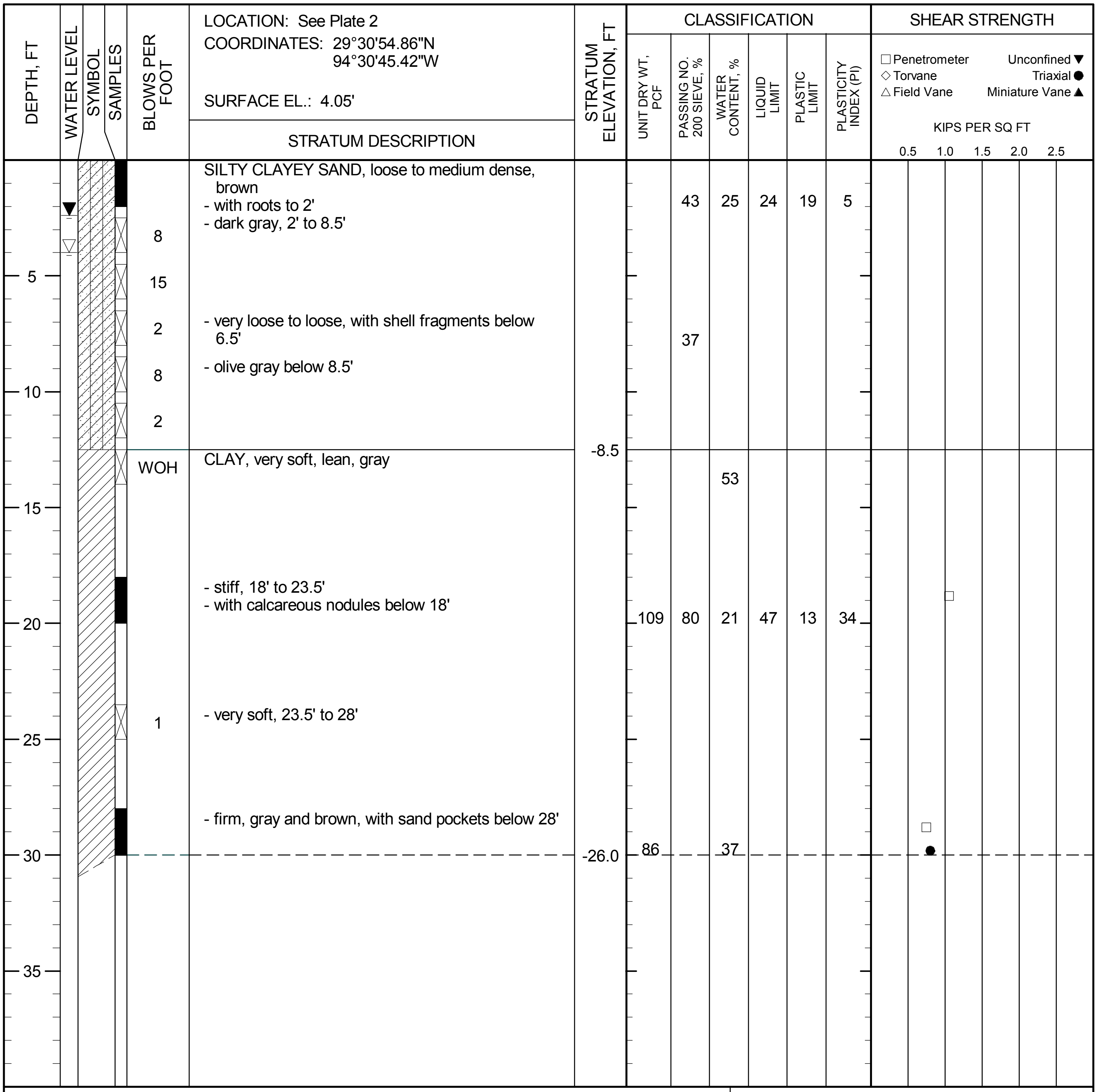
BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
	CLAYEY SAND, medium dense, fine-grained, brown - with roots and gravel to 1'			21	19				
12									
2	- very loose below 4' - gray, 4' to 10.5'			30	29				
1									
WOH	- with wood and shell fragments below 8.5'								
WOH	- dark gray below 10.5'			32					
3									
3				36					
5	CLAY, firm, lean, dark gray, with shell fragments and sand pockets	-14.4		81	36	47	15	32	
	(Refusal at 22')	-17.9							

SHEAR STRENGTH legend: ☐ Penetrometer, ◇ Torvane, △ Field Vane, Unconfined ▼, Triaxial ●, Miniature Vane ▲; KIPS PER SQ FT 0.5 1.0 1.5 2.0 2.5

NOTES:
1. ▽: Water First Noticed. ▼: Depth To Water after 20 minutes.
2. Boring coordinates were obtained with a hand-held GPS device.
3. WOH - Split-spoon sampler advanced due to the weight of the hammer.
4. Terms and symbols defined on Plates 12a and 12b.

DATE: February 8, 2018
TOTAL DEPTH: 22'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 6'
WET ROTARY: 6' to 22'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-5
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS



LOCATION: See Plate 2
COORDINATES: 29°30'54.86"N
94°30'45.42"W
SURFACE EL.: 4.05'

STRATUM DESCRIPTION	BLOWS PER FOOT	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
SILTY CLAYEY SAND, loose to medium dense, brown - with roots to 2' - dark gray, 2' to 8.5'				43	25	24	19	5
	8							
	15							
- very loose to loose, with shell fragments below 6.5'	2			37				
- olive gray below 8.5'	8							
	2	-8.5						
CLAY, very soft, lean, gray	WOH				53			
- stiff, 18' to 23.5' - with calcareous nodules below 18'			109	80	21	47	13	34
- very soft, 23.5' to 28'	1							
- firm, gray and brown, with sand pockets below 28'		-26.0	86		37			

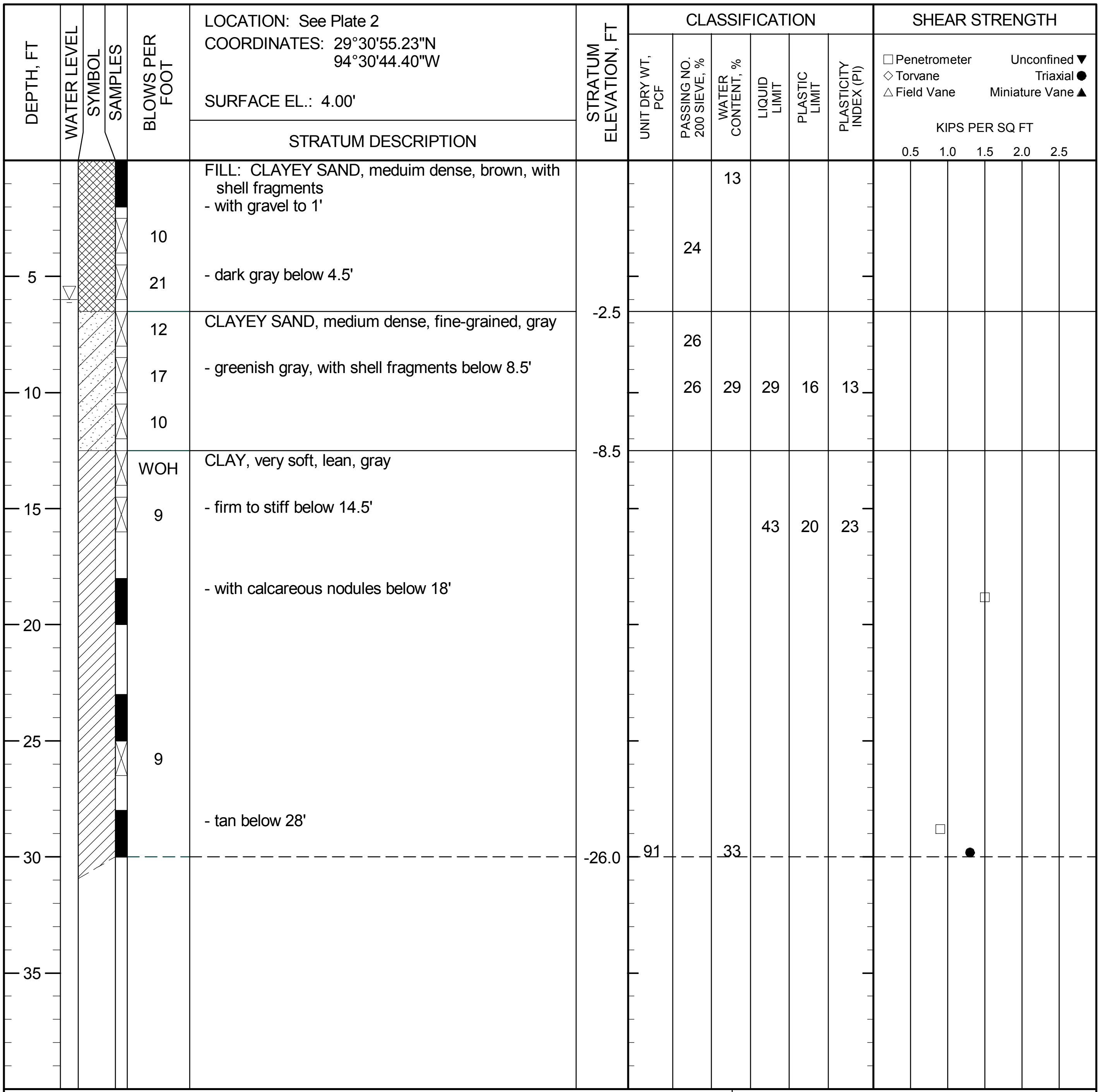
NOTES:

1. ▽: Water First Noticed. ▼: Depth To Water after 20 minutes.
2. Boring coordinates were obtained with a hand-held GPS device.
3. WOH - Split-spoon sampler advanced due to the weight of the hammer.
4. Terms and symbols defined on Plates 12a and 12b.

DATE: February 8, 2018
TOTAL DEPTH: 30'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 6'
WET ROTARY: 6' to 30'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-6
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°30'55.23"N
94°30'44.40"W
SURFACE EL.: 4.00'



BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
10 21	FILL: CLAYEY SAND, meduim dense, brown, with shell fragments - with gravel to 1' - dark gray below 4.5'	-2.5		24	13			
12 17 10	CLAYEY SAND, medium dense, fine-grained, gray - greenish gray, with shell fragments below 8.5'	-8.5		26 26	29	29	16	13
WOH 9 9	CLAY, very soft, lean, gray - firm to stiff below 14.5' - with calcareous nodules below 18' - tan below 28'	-26.0	91		33	43	20	23

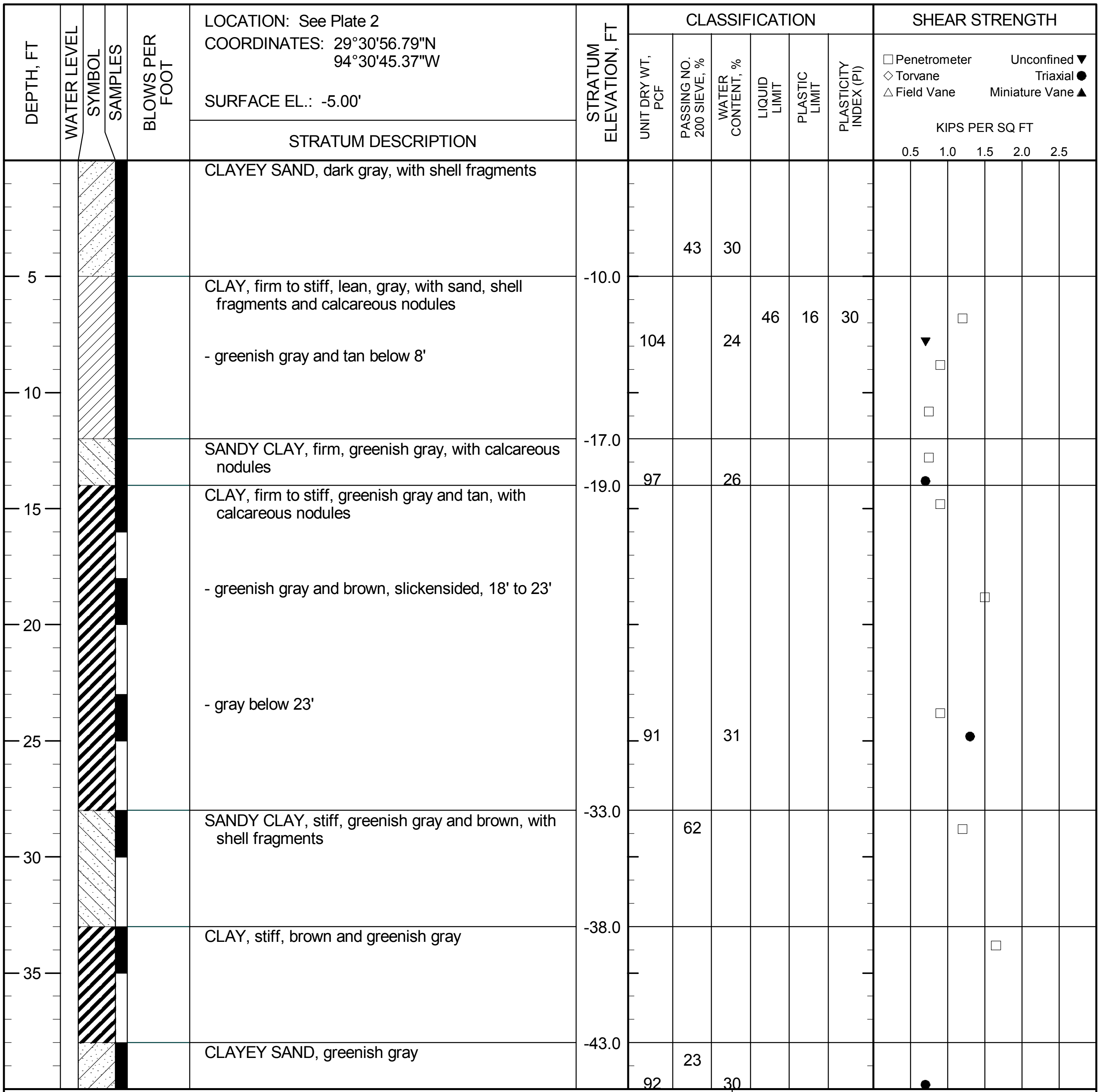
NOTES:

1. ▽: Water First Noticed.
2. Boring coordinates were obtained with a hand-held GPS device.
3. WOH - Split-spoon sampler advanced due to the weight of the hammer.
4. Terms and symbols defined on Plates 12a and 12b.

DATE: February 8, 2018
TOTAL DEPTH: 30'
CAVED DEPTH: 5.1'
DRY AUGER: Surface to 8'
WET ROTARY: 8' to 30'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-7

GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS



LOCATION: See Plate 2
COORDINATES: 29°30'56.79"N
94°30'45.37"W

SURFACE EL.: -5.00'

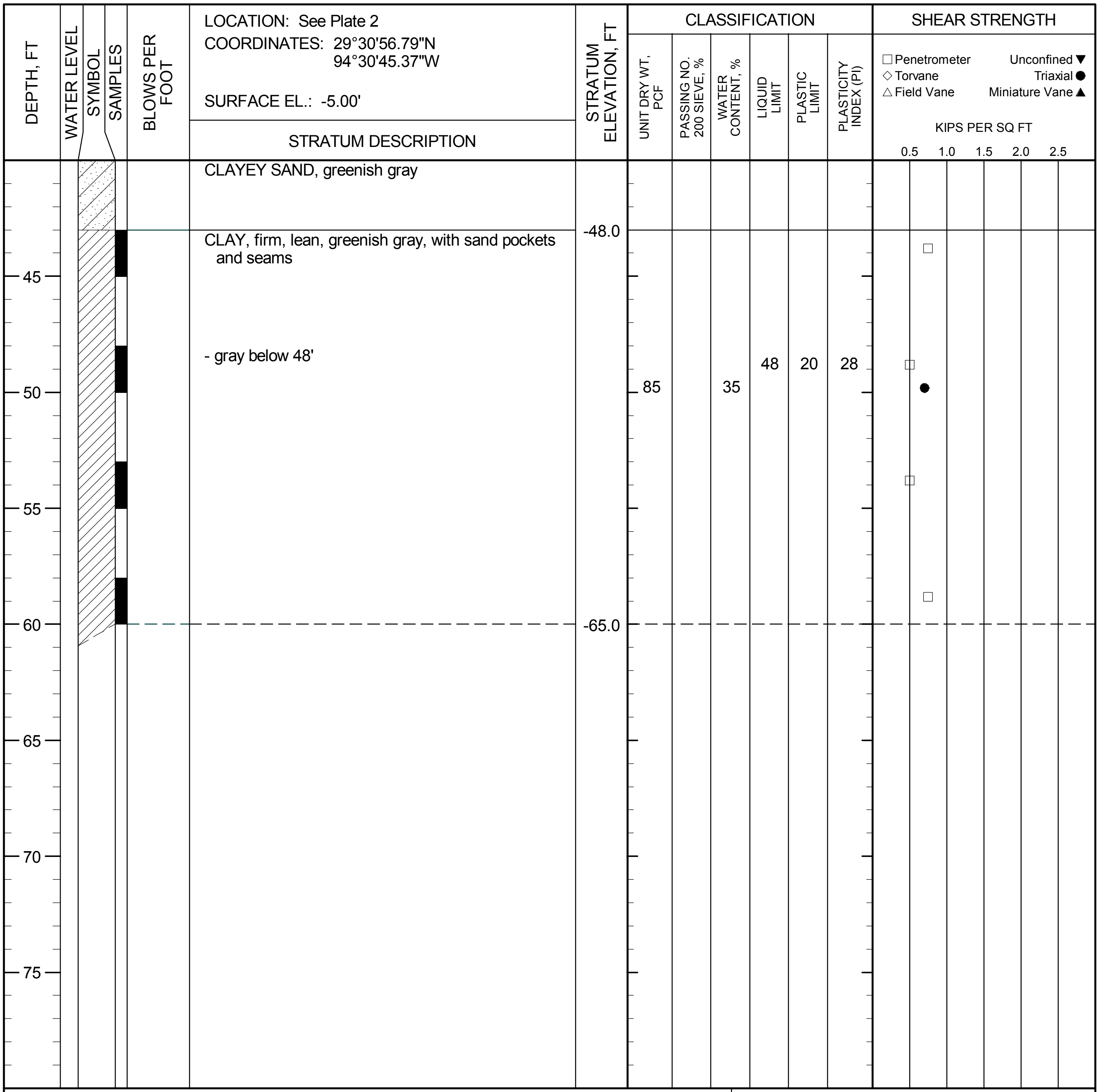
STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
CLAYEY SAND, dark gray, with shell fragments			43	30			
CLAY, firm to stiff, lean, gray, with sand, shell fragments and calcareous nodules - greenish gray and tan below 8'	-10.0	104		24	46	16	30
SANDY CLAY, firm, greenish gray, with calcareous nodules	-17.0	97		26			
CLAY, firm to stiff, greenish gray and tan, with calcareous nodules - greenish gray and brown, slickensided, 18' to 23' - gray below 23'	-19.0	91		31			
SANDY CLAY, stiff, greenish gray and brown, with shell fragments	-33.0		62				
CLAY, stiff, brown and greenish gray	-38.0						
CLAYEY SAND, greenish gray	-43.0	92	23	30			

NOTES:
1. Water depth was approximately 6 feet above the mudline.
2. Boring coordinates were obtained with a hand-held GPS device.
3. Terms and symbols defined on Plates 12a and 12b.

DATE: February 13, 2018
TOTAL DEPTH: 60'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-8
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°30'56.79"N
94°30'45.37"W
SURFACE EL.: -5.00'

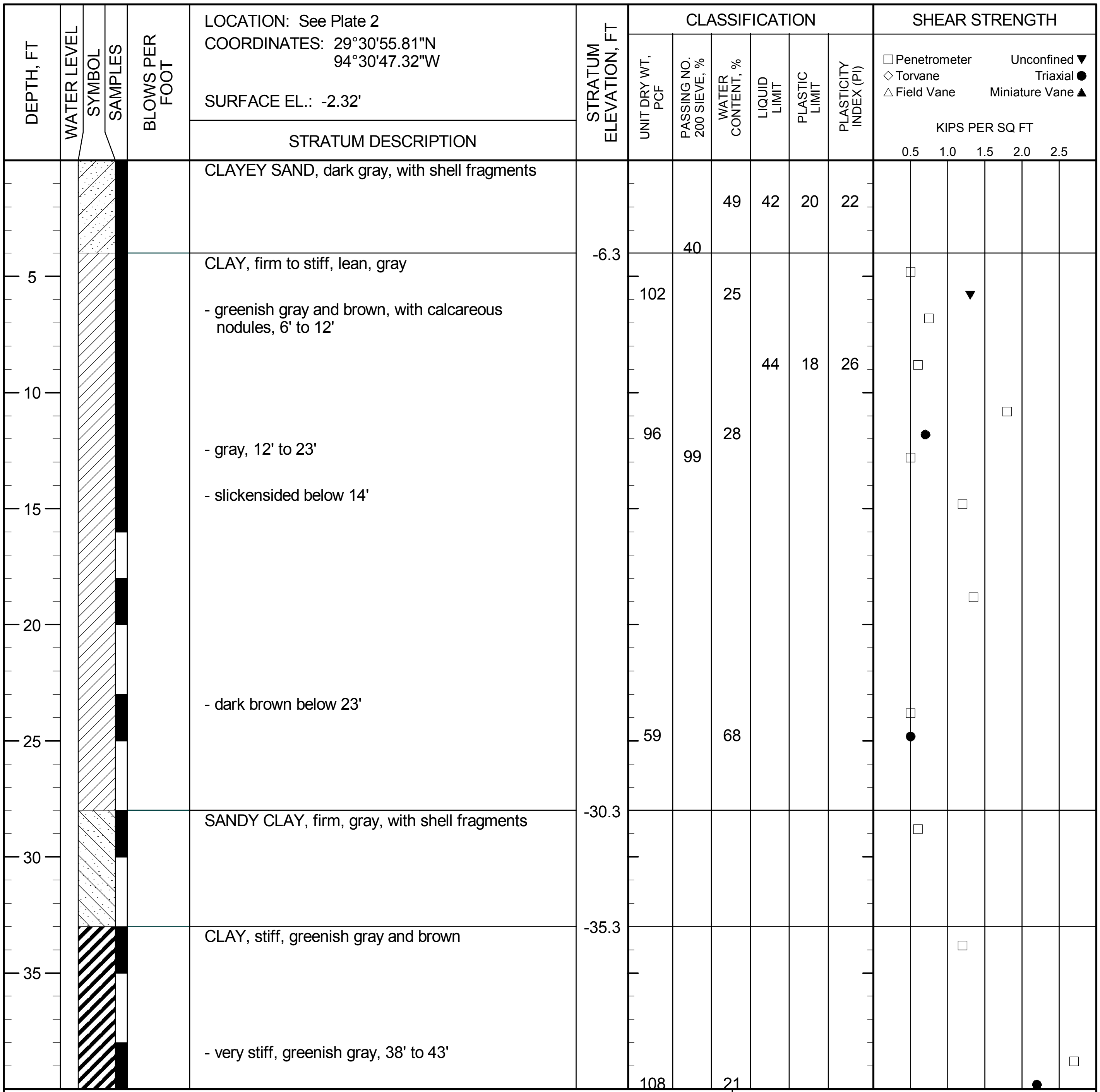


STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
CLAYEY SAND, greenish gray	-48.0						
CLAY, firm, lean, greenish gray, with sand pockets and seams - gray below 48'		85		35	48	20	28
	-65.0						

NOTES:
1. Water depth was approximately 6 feet above the mudline.
2. Boring coordinates were obtained with a hand-held GPS device.
3. Terms and symbols defined on Plates 12a and 12b.

DATE: February 13, 2018
TOTAL DEPTH: 60'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-8
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS



LOCATION: See Plate 2
COORDINATES: 29°30'55.81"N
94°30'47.32"W

SURFACE EL.: -2.32'

STRATUM DESCRIPTION	STRATUM ELEVATION, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
CLAYEY SAND, dark gray, with shell fragments				49	42	20	22
			40				
CLAY, firm to stiff, lean, gray	-6.3	102		25			
- greenish gray and brown, with calcareous nodules, 6' to 12'							
					44	18	26
		96		28			
- gray, 12' to 23'			99				
- slickensided below 14'							
- dark brown below 23'							
		59		68			
SANDY CLAY, firm, gray, with shell fragments	-30.3						
CLAY, stiff, greenish gray and brown	-35.3						
- very stiff, greenish gray, 38' to 43'							
		108		21			

NOTES:
1. Water depth was approximately 6 feet above the mudline.
2. Boring coordinates were obtained with a hand-held GPS device.
3. Terms and symbols defined on Plates 12a and 12b.

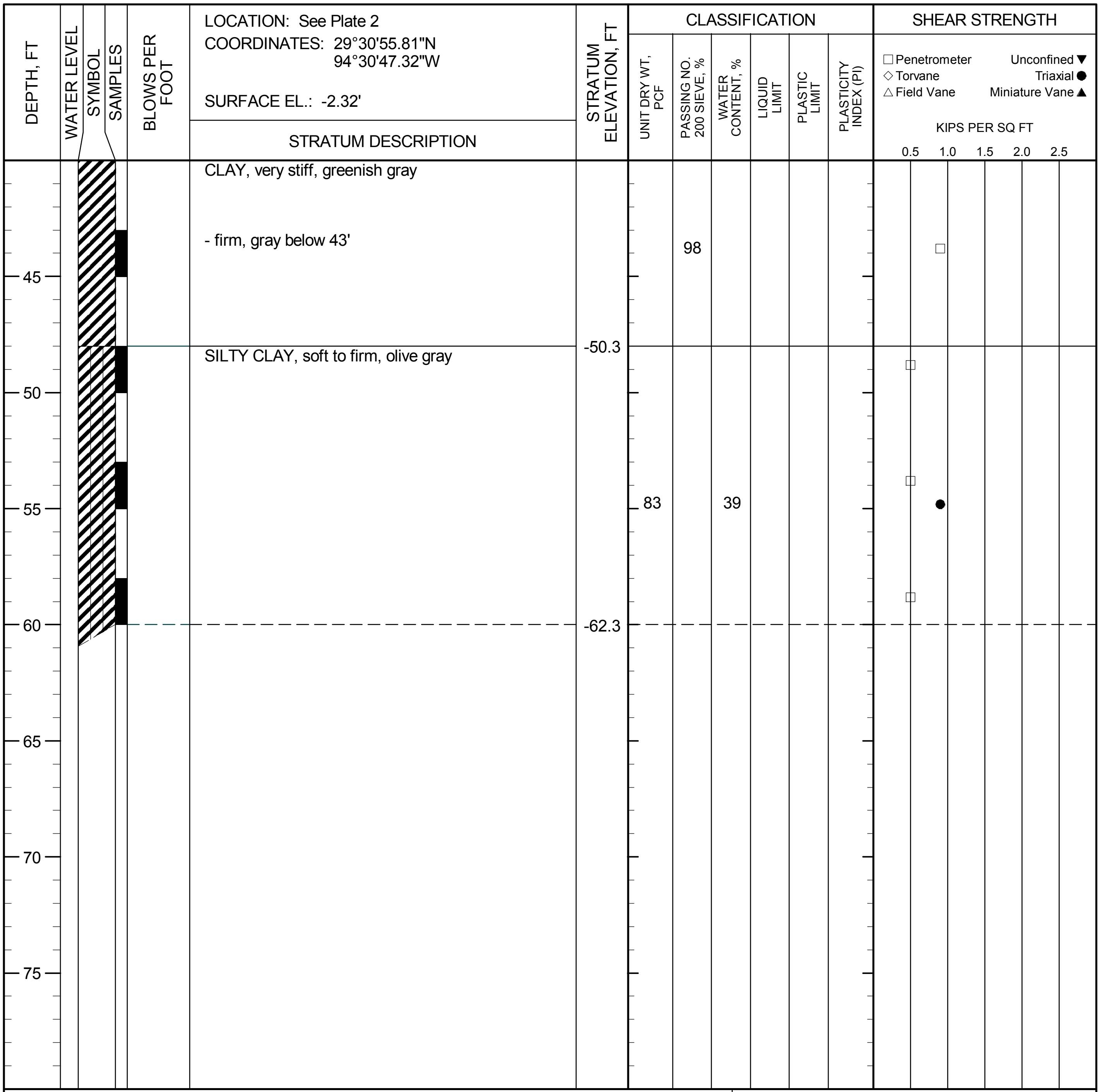
DATE: February 14, 2018
TOTAL DEPTH: 60'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

LOG OF BORING NO. B-9
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

LOCATION: See Plate 2

COORDINATES: 29°30'55.81"N
94°30'47.32"W

SURFACE EL.: -2.32'



CLAY, very stiff, greenish gray

- firm, gray below 43'

SILTY CLAY, soft to firm, olive gray

NOTES:

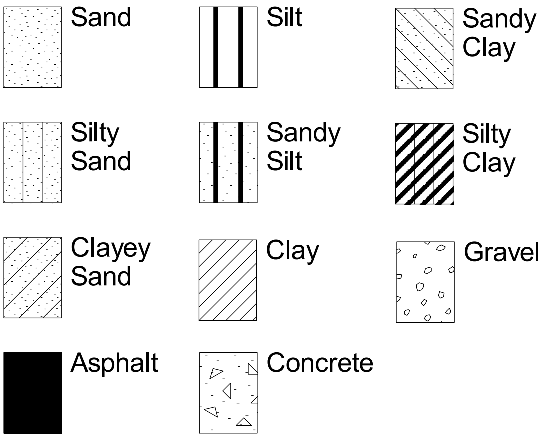
1. Water depth was approximately 6 feet above the mudline.
2. Boring coordinates were obtained with a hand-held GPS device.
3. Terms and symbols defined on Plates 12a and 12b.

DATE: February 14, 2018
TOTAL DEPTH: 60'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: M. Davidson

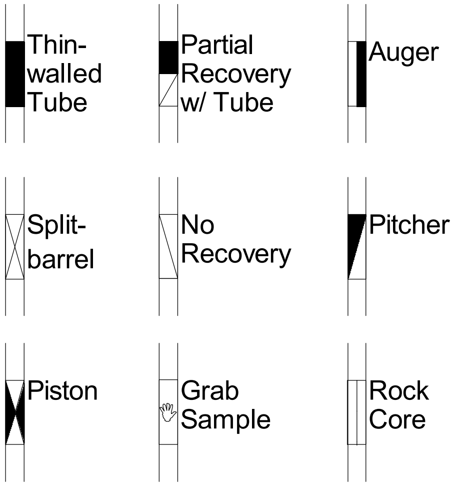
LOG OF BORING NO. B-9

GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

SOIL TYPES



SAMPLER TYPES



SOIL GRAIN SIZE

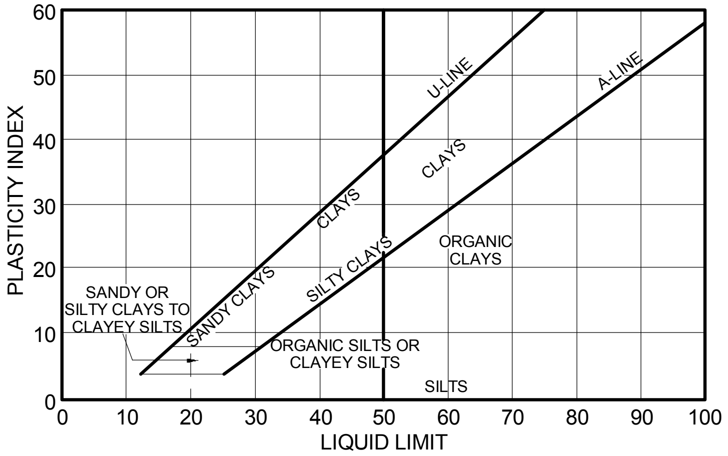
U.S. Standard Sieve

6" 3" 3/4" 4 10 40 200

Boulders	Cobbles	Gravel		Sand			Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine		

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002 (mm)

PLASTICITY CHART



SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

TERMS AND SYMBOLS USED ON BORING LOGS

SOIL CLASSIFICATION (1 of 2)

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

DENSITY OF GRANULAR SOILS

Descriptive Term	*Relative Density, %	**Blows Per Foot (SPT)
Very Loose	< 15	0 to 4
Loose	15 to 35	5 to 10
Medium Dense	35 to 65	11 to 30
Dense	65 to 85	31 to 50
Very Dense	> 85	> 50

*Estimated from sampler driving record.

**Requires correction for depth, groundwater level, and grain size.

STRENGTH OF COHESIVE SOILS

Term	Undrained Shear Strength, ksf	Blows Per Foot (SPT) (approximate)
Very Soft	< 0.25	0 to 2
Soft	0.25 to 0.50	2 to 4
Firm	0.50 to 1.00	4 to 8
Stiff	1.00 to 2.00	8 to 16
Very Stiff	2.00 to 4.00	16 to 32
Hard	> 4.00	> 32

SHEAR STRENGTH TEST METHOD

U - Unconfined Q = Unconsolidated - Undrained Triaxial

P = Pocket Penetrometer T = Torvane V = Miniature Vane F = Field Vane

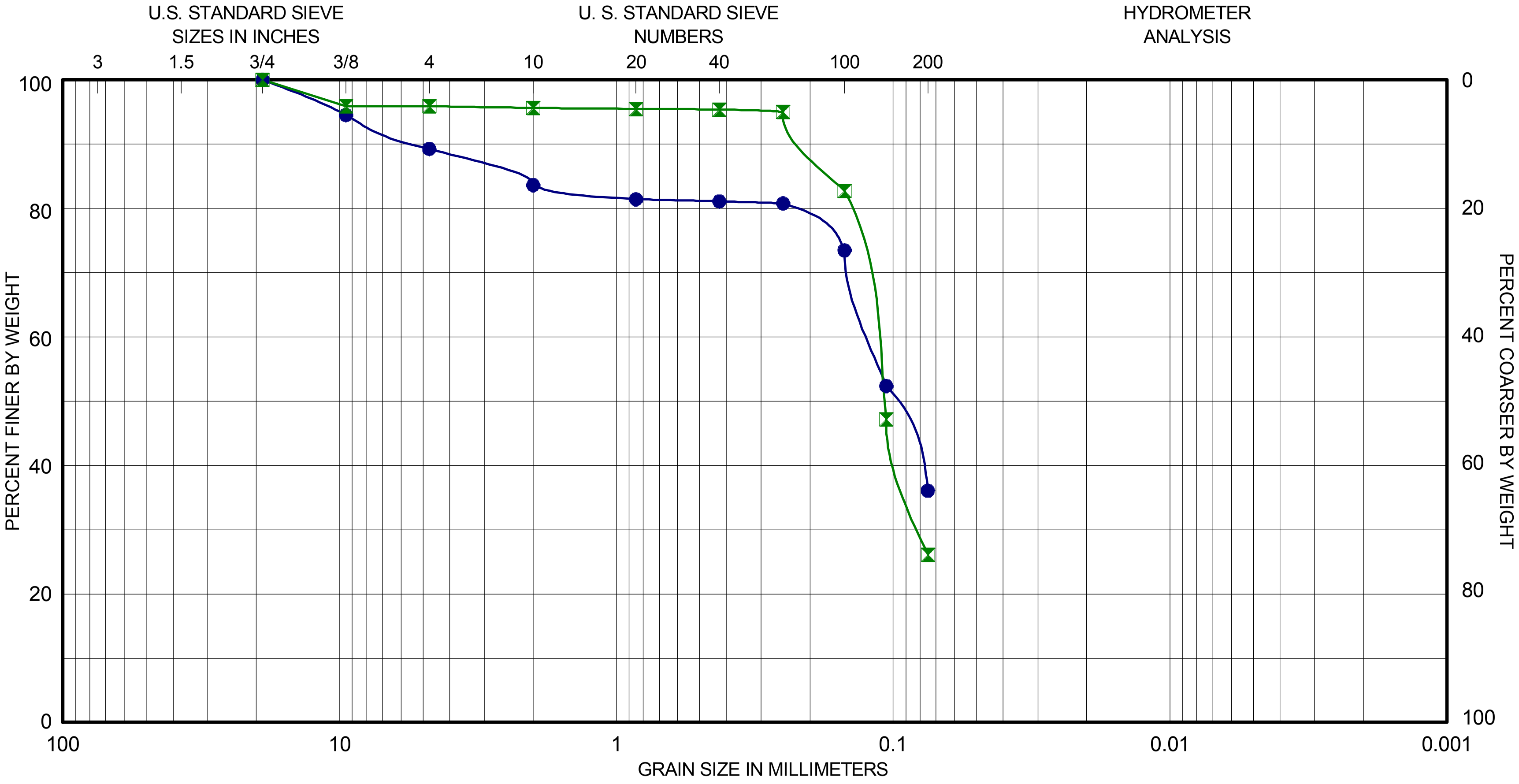
HAND PENETROMETER CORRECTION

Our experience has shown that the hand penetrometer generally overestimates the in-situ undrained shear strength of over consolidated Pleistocene Gulf Coast clays. These strengths are partially controlled by the presence of macroscopic soil defects such as slickensides, which generally do not influence smaller scale tests like the hand penetrometer. Based on our experience, we have adjusted these field estimates of the undrained shear strength of natural, overconsolidated Pleistocene Gulf Coast soils by multiplying the measured penetrometer reading by a factor of 0.6. These adjusted strength estimates are recorded in the "Shear Strength" column on the boring logs. Except as described in the text, we have not adjusted estimates of the undrained shear strength for projects located outside of the Pleistocene Gulf Coast formations.

Information on each boring log is a compilation of subsurface conditions and soil or rock classifications obtained from the field as well as from laboratory testing of samples. Strata have been interpreted by commonly accepted procedures. The stratum lines on the logs may be transitional and approximate in nature. Water level measurements refer only to those observed at the time and places indicated, and can vary with time, geologic condition, or construction activity.

TERMS AND SYMBOLS USED ON BORING LOGS

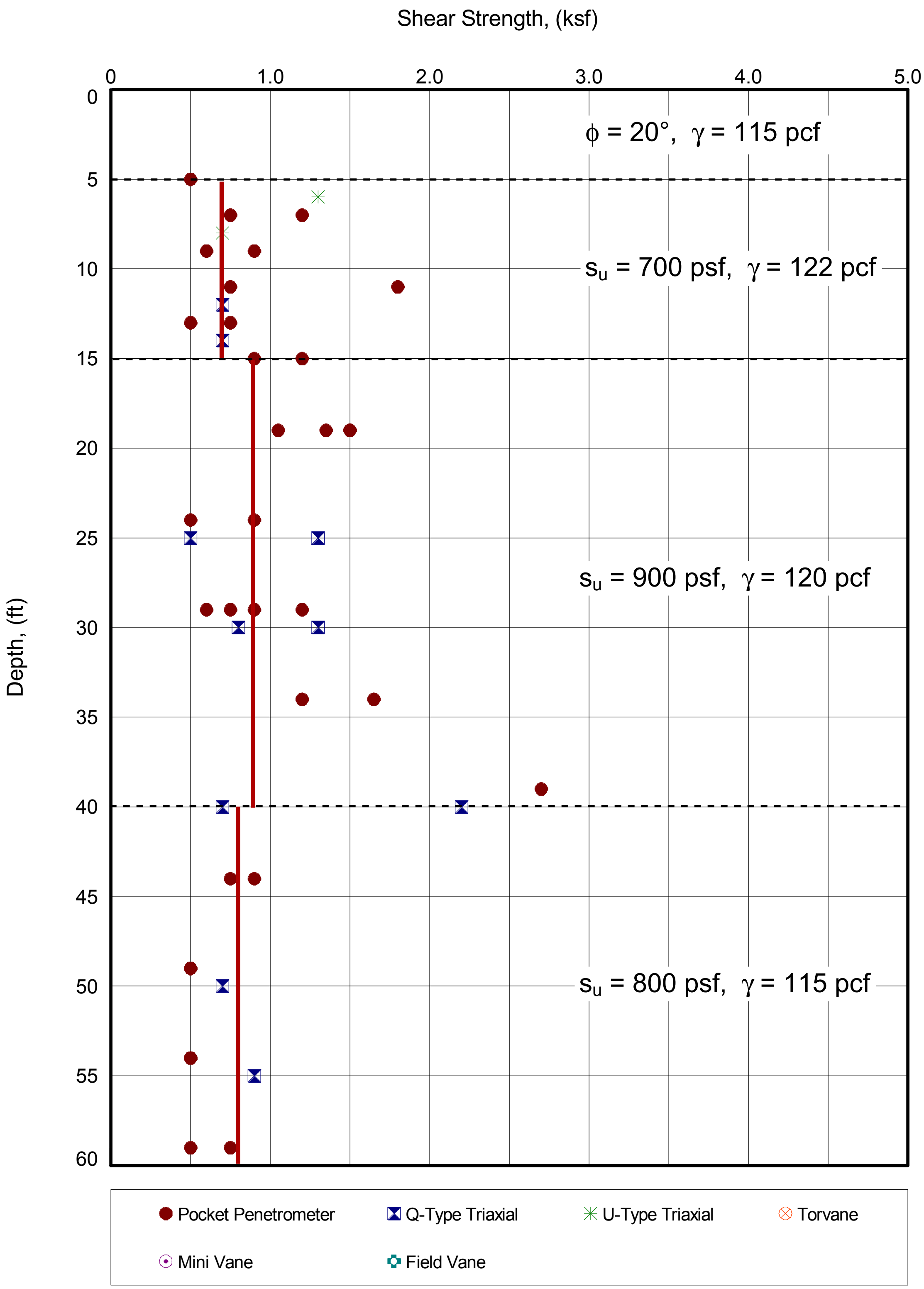
SOIL CLASSIFICATION (2 of 2)



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

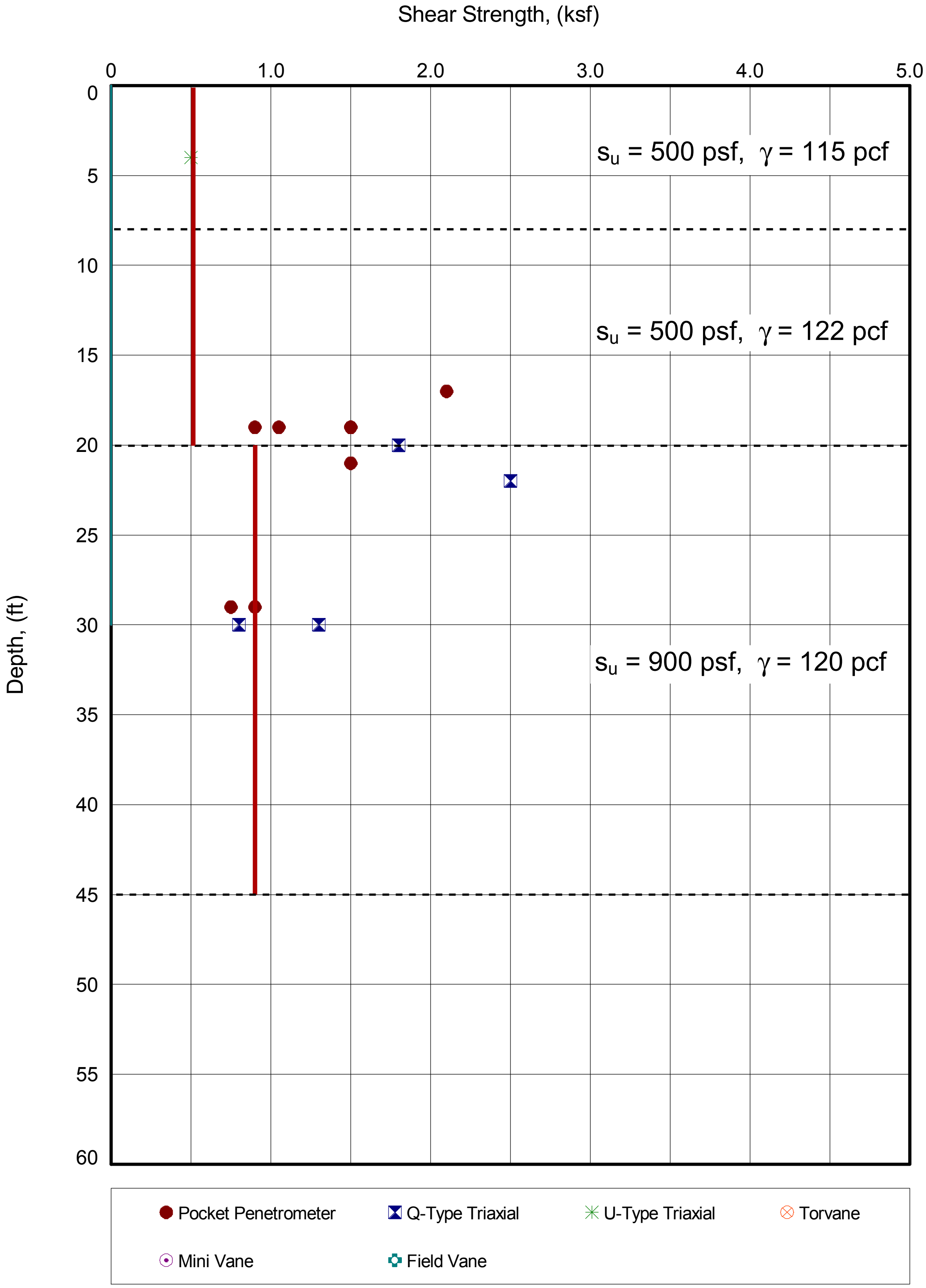
SYMBOL	BORING	DEPTH, FT	C_c	C_u	D_{10}	D_{30}	D_{60}	CLASSIFICATION
●	B-5	16					0.12	CLAYEY SAND, fine-grained, greenish gray and olive gray
⧗	B-7	8				0.08	0.12	CLAYEY SAND, fine-grained, greenish gray, with shell fragments

GRAIN SIZE CURVES
GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS



DESIGN SOIL PARAMETERS - NEARSHORE LOCATIONS (BORINGS B-5 THROUGH B-9)

GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS



DESIGN SOIL PARAMETERS - ONSHORE LOCATIONS (BORINGS B-1 THROUGH B-7)

GEOTECHNICAL STUDY
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

TABLE: PILE CAPACITY DESIGN PARAMETERS – NEARSHORE LOCATIONS

Depth Below Mudline (ft)	Soil Description	Total Unit Weight(4) (pcf)	Axial Capacity(3)					Lateral Capacity				
			Cohesion (psf)	Friction Angle, ϕ (degrees)	Limiting Skin Friction (ksf)	N_q	Limiting End Bearing (ksf)	Soil Type	Cohesion(3) (psf)	Friction Angle, ϕ(3) (degrees)	Subgrade Modulus (pcf)	ε_{50}(6) (in/in)
0 to 5(2)	Clayey Sand	115	Neglected to account for soil scour.(5)					Clayey Sand	Neglected to account for soil scour.(5)			
5 to 15	Firm Clay	122	700	-	-	-	-	Firm Clay	700	-	-	0.01
15 to 40	Firm Clay	120	900	-	-	-	-	Firm Clay	900	-	-	0.01
40 to 60	Firm Clay	115	800	-	-	-	-	Firm Clay	800	-	-	0.01

NOTES:

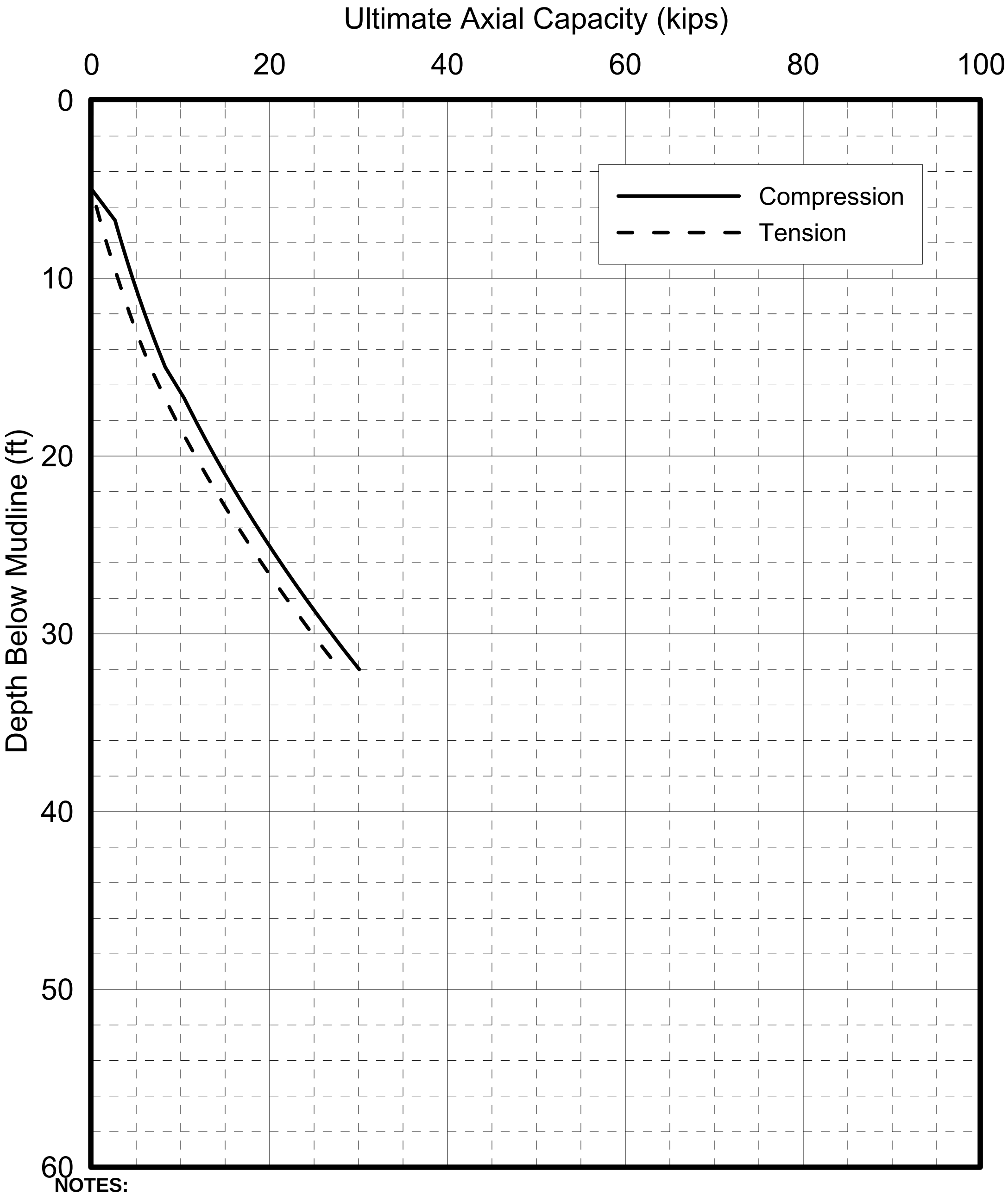
(1) This table presents design parameters used for axial and lateral capacity analyses for piles.
(2) Mudline elevation was not available at the time of this report.
(3) Selection of undrained shear strength, friction angle, and other engineering parameters are based on the laboratory test results, CPTs, SPT (N) values, pocket penetrometer, and torvane values from pertinent boring logs.
(4) Water table was assumed above mudline.
(5) For axial capacity and lateral capacity, we neglected the soil strength in the upper 5 feet below mudline to account for scour due to the presence of granular soils at mudline.
(6) Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength. The strains at 50% of maximum stress are same for both static and cyclic loading conditions in clays.
(7) API recommends using coefficients of lateral earth pressure for compression (k_c) and tension (k_t) equal to 1.0 and 0.7 respectively, for timber piles.

TABLE: PILE CAPACITY DESIGN PARAMETERS – ONSHORE LOCATIONS

Depth (ft)	Soil Description	Total Unit Weight(4) (pcf)	Axial Capacity(3)					Lateral Capacity				
			Cohesion (psf)	Friction Angle, ϕ (degrees)	Limiting Skin Friction (ksf)	N_q	Limiting End Bearing (ksf)	Soil Type	Cohesion(3) (psf)	Friction Angle, ϕ(3) (degrees)	Subgrade Modulus (pcf)	ε_{50}(6) (in/in)
0 to 2(2)	Fill (Soft to Firm Clay)	115	Neglected to account for disturbance(5)					Fill (Soft to Firm Clay)	Neglected to account for disturbance(5)			
2 to 5	Fill (Soft to Firm Clay)	115						Fill (Soft to Firm Clay)	500	-	-	0.02
5 to 8	Fill (Soft to Firm Clay)	115	500	-	-	-	-	Fill (Soft to Firm Clay)	500	-	-	0.02
8 to 20	Firm Clay	122	500	-	-	-	-	Firm Clay	500	-	-	0.02
20 to 45	Firm Clay	120	900	-	-	-	-	Firm Clay	900	-	-	0.01
45 to 60	Firm Clay	115	800	-	-	-	-	Firm Clay	800	-	-	0.01

NOTES:

(1) This table presents design parameters used for axial and lateral capacity analyses for piles.
(2) Final grade was assumed at existing grade elevation
(3) Selection of undrained shear strength, friction angle, and other engineering parameters are based on the laboratory test results, CPTs, SPT (N) values, pocket penetrometer, and torvane values from pertinent borings.
(4) Groundwater table was assumed at existing grade.
(5) For axial capacity, we neglected the soil strength in the upper 5 feet below existing grade to account for construction disturbance. For lateral capacity, we neglected the soil strength in the upper 2 feet below existing grade to account for construction disturbance.
(6) Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2017), and are correlated with the estimated undrained shear strength. The strains at 50% of maximum stress are same for both static and cyclic loading conditions in clays.
(7) API recommends using coefficients of lateral earth pressure for compression (k_c) and tension (k_t) equal to 1.0 and 0.7 respectively, for timber piles.



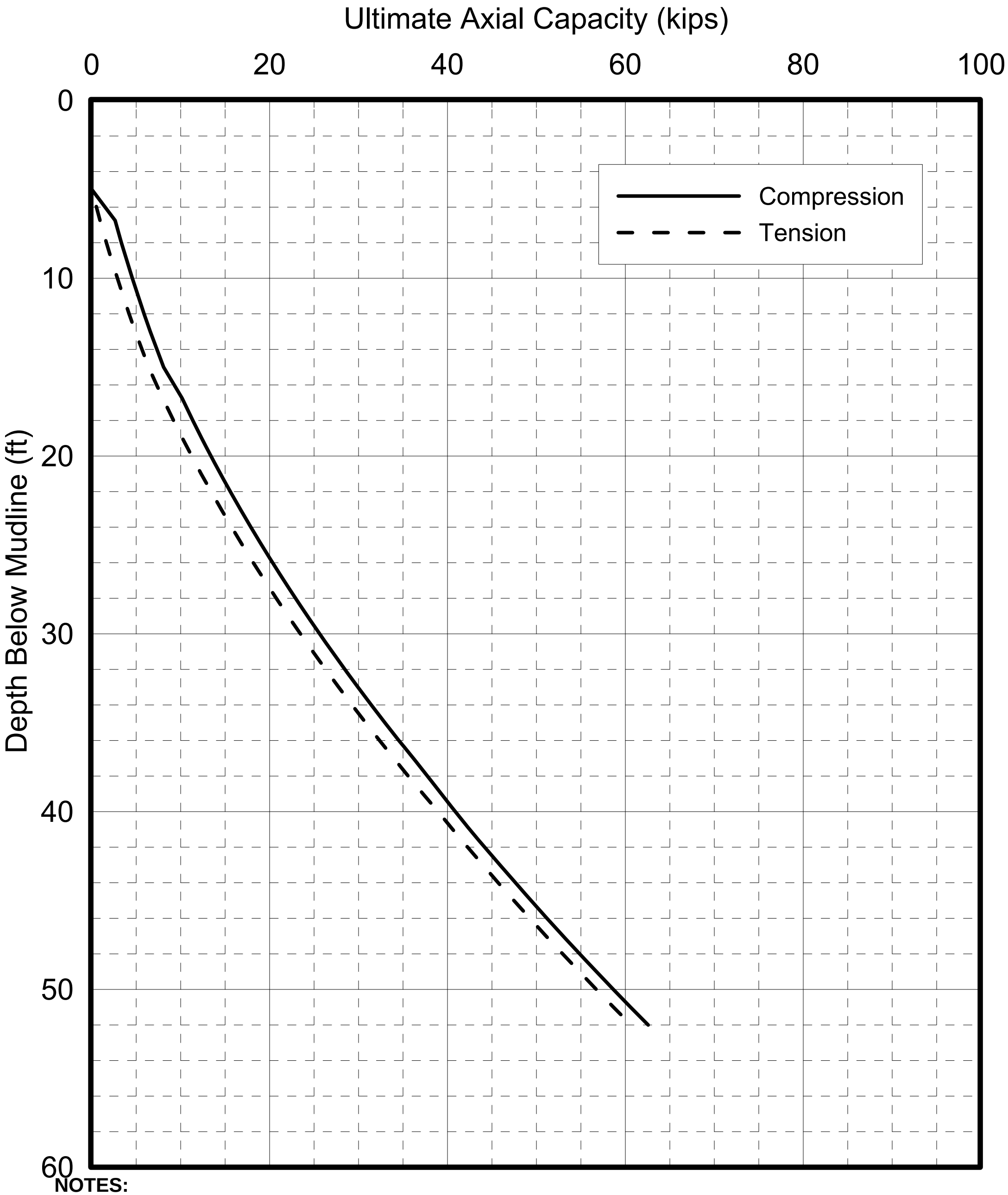
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained compressive loads or transient tensile loads and a safety factor of 3.0 should be applied for sustained tensile loads.
2. These curves are for a single isolated pile. Group effects are discussed in the text.
3. Depth referenced from mudline. Pile top was assumed to be at 8 ft above mudline.
4. Weight of the pile was not included in the computations.
5. Pile capacities were calculated using the API RP-2A 2000 method.

GEOTECHNICAL ULTIMATE AXIAL PILE CAPACITY - NEARSHORE LOCATIONS

12-INCH TO 7-INCH TAPERED TIMBER PILE (40-FT LONG)
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

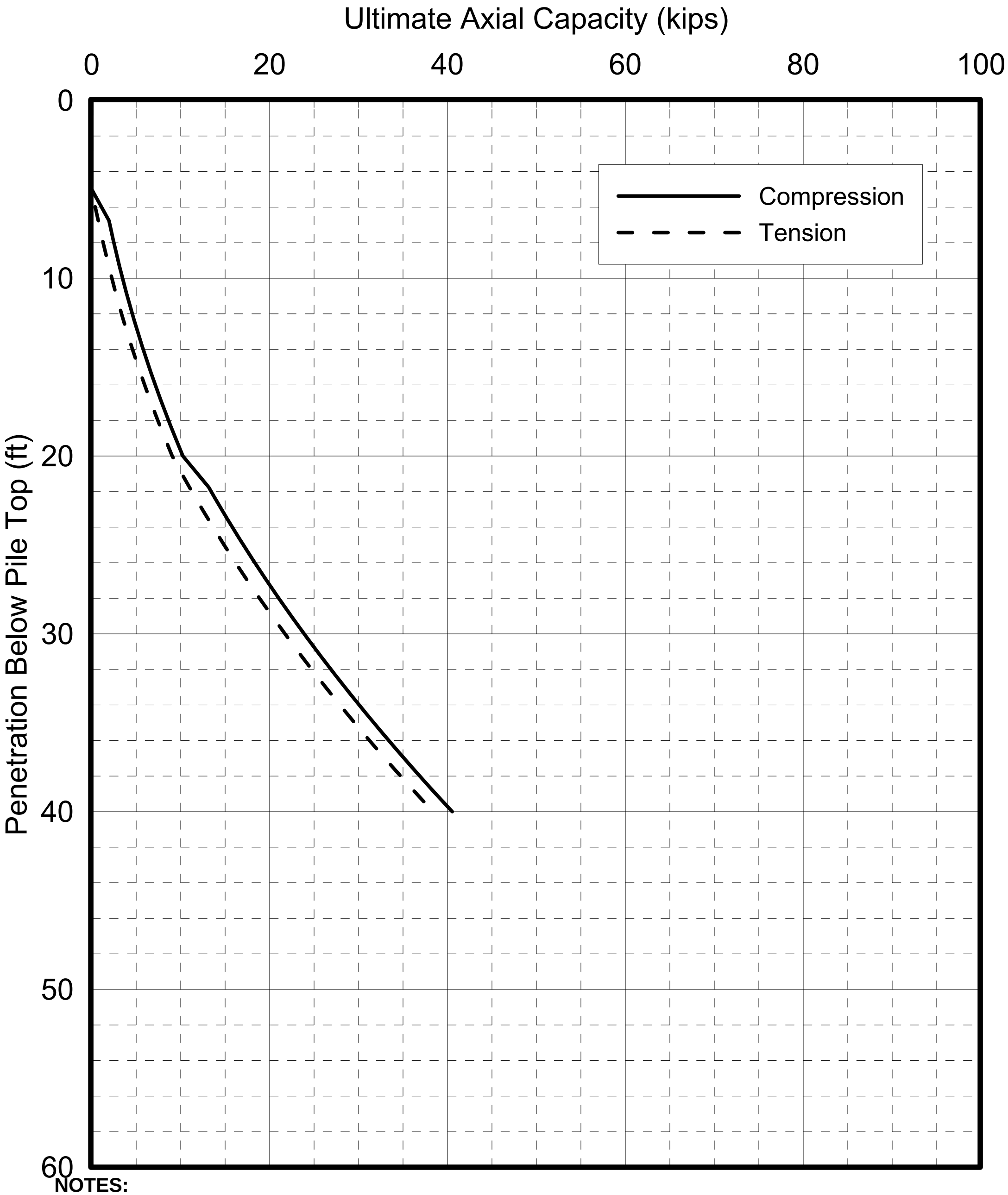
PLATE 16a



NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained compressive loads or transient tensile loads and a safety factor of 3.0 should be applied for sustained tensile loads.
2. These curves are for a single isolated pile. Group effects are discussed in the text.
3. Depth referenced from mudline. Pile top was assumed to be at 8 ft above mudline.
4. Weight of the pile was not included in the computations.
5. Pile capacities were calculated using the API RP-2A 2000 method.

GEOTECHNICAL ULTIMATE AXIAL PILE CAPACITY - NEARSHORE LOCATIONS
12-INCH TO 7-INCH TAPERED TIMBER PILE (60-ft LONG)
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

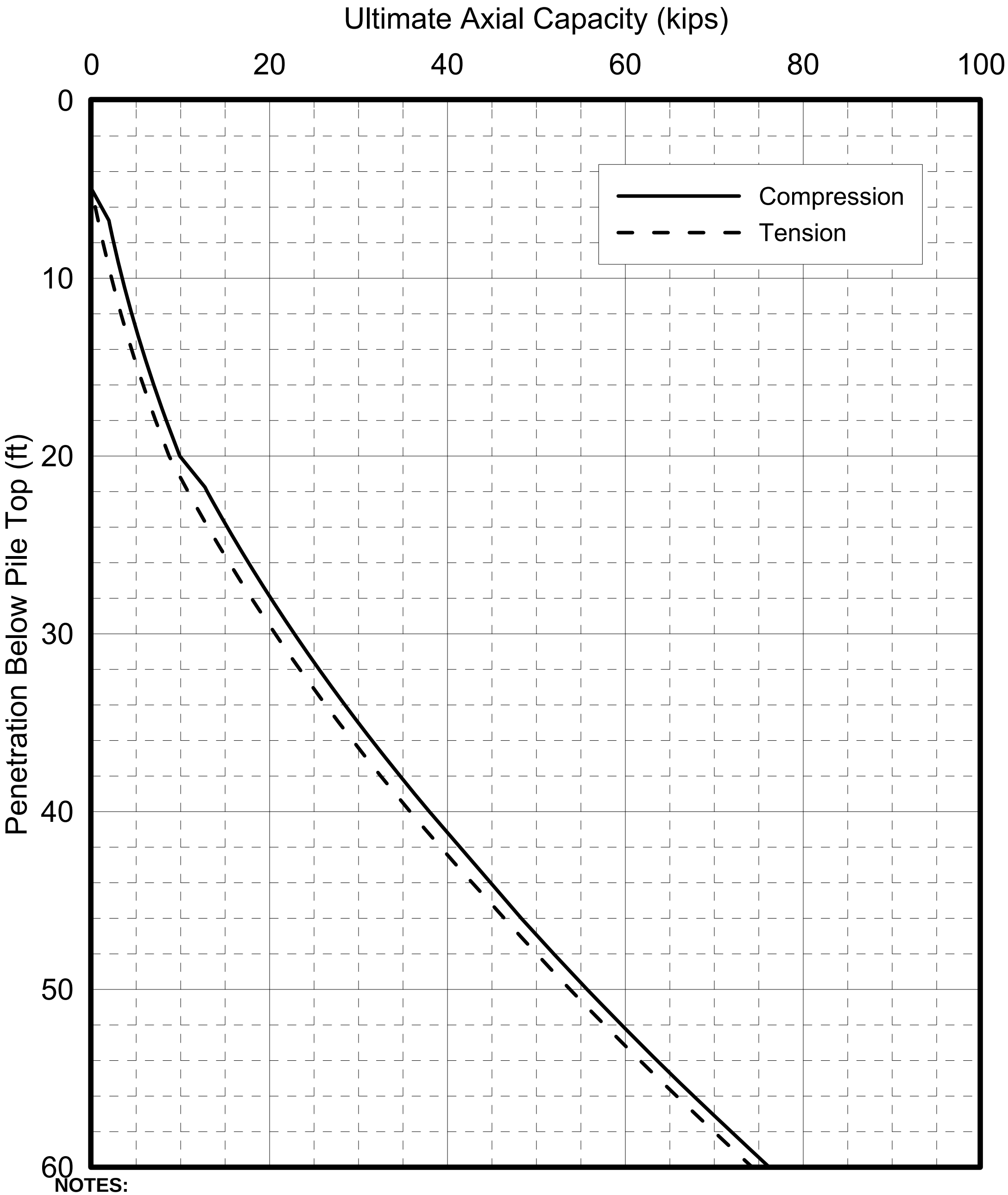


NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained compressive loads or transient tensile loads and a safety factor of 3.0 should be applied for sustained tensile loads.
2. These curves are for a single isolated pile. Group effects are discussed in the text.
3. Pile top and groundwater elevation was assumed at existing grade.
4. Weight of the pile was not included in the computations.
5. Pile capacities were calculated using the API RP-2A 2000 method.

GEOTECHNICAL ULTIMATE AXIAL PILE CAPACITY - ONSHORE LOCATIONS

12-INCH TO 7-INCH TAPERED TIMBER PILE (40-FT LONG)
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS



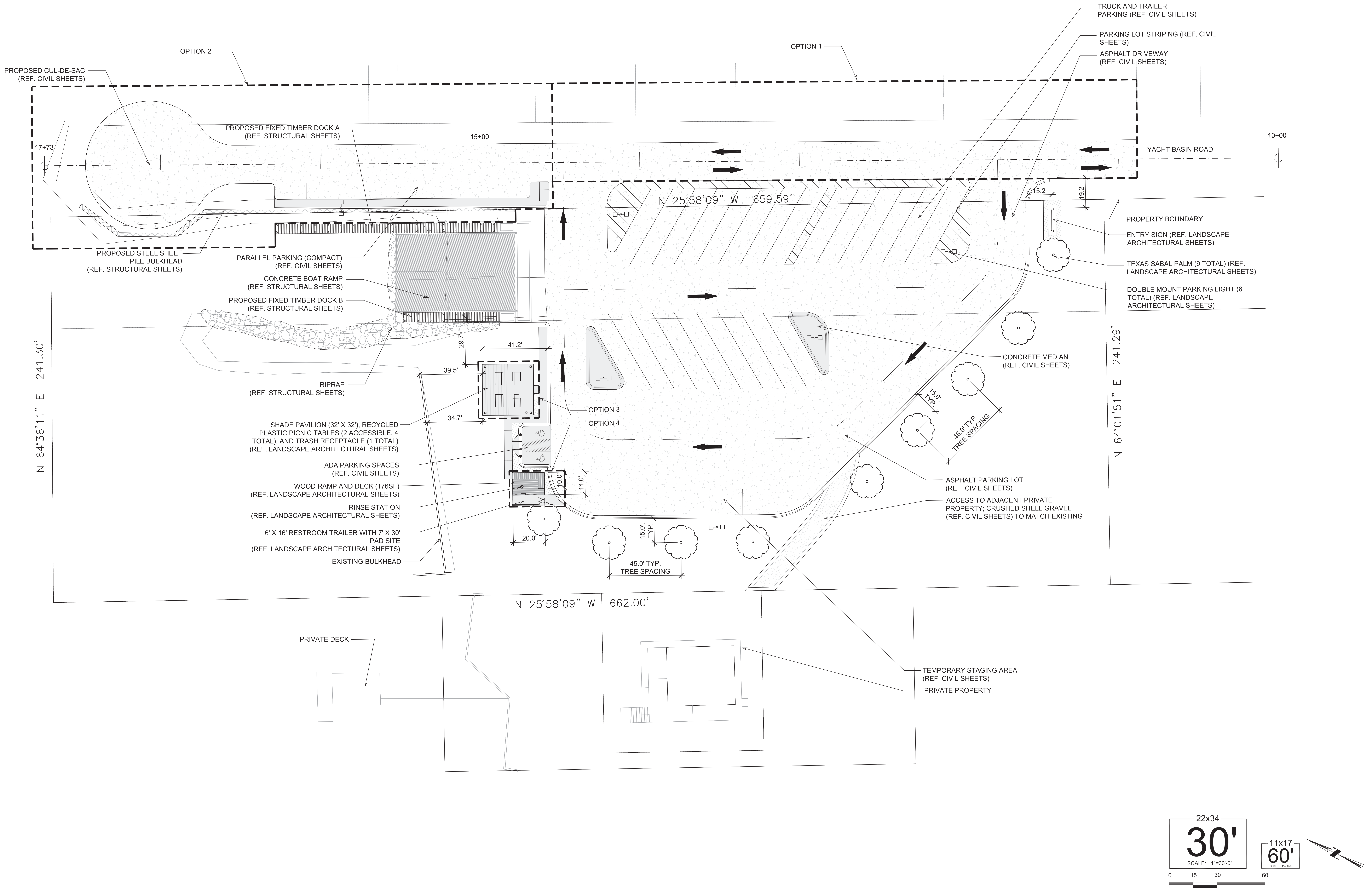
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained compressive loads or transient tensile loads and a safety factor of 3.0 should be applied for sustained tensile loads.
2. These curves are for a single isolated pile. Group effects are discussed in the text.
3. Pile top and groundwater elevation was assumed at existing grade.
4. Weight of the pile was not included in the computations.
5. Pile capacities were calculated using the API RP-2A 2000 method.

GEOTECHNICAL ULTIMATE AXIAL PILE CAPACITY - ONSHORE LOCATIONS
12-INCH TO 7-INCH TAPERED TIMBER PILE (60-FT LONG)
LAUDERDALE RECREATIONAL FACILITY
GILCHRIST, TEXAS

APPENDIX A

PROPOSED DEVELOPMENT



NO.	REVISION	BY	DATE

DRAWN BY: L. URIE
DESIGNED BY: S. LAWTON
CHECKED BY: S. CALFEE
APPROVED BY: S. LAWTON
SCALE: 1"=30'
DATE: 3/29/2018

THESE DOCUMENTS ARE FOR INTERIM REVIEW AND NOT FOR CONSTRUCTION, BIDDING OR PERMIT PURPOSES.

SUMMER B. LAWTON, P.L.A. NO 2444
29–Mar–18
TBPE FIRM REGISTRATION No. 420

HNTB CORPORATION
TBPE FIRM REGISTRATION NO. 420

GALVESTON COUNTY

LAUDERDALE BOAT DOCK

OVERALL SITE PLAN

SP-01
SHEET 4
OF 20

\\ausw00\Jobs\57947_GLO_DR\Techprod\Lauderdale Boat Dock\Techprod\UDP\CAD\Sheets\30%   Mar 29, 2018 - 7:43pm

Lauderdale Recreational Facility Project

Write a description for your map.