Runway 18 Approach Lighting Project

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Fugro USA Land, Inc. (Fugro) is pleased to present our geotechnical study report for the Runway 18 Approach Lighting project in Houma, Louisiana. We performed our services in accordance with Fugro Proposal dated August 14, 2023. Our services were authorized on February 27, 2024, through the issuance of Consulting Services Task Order No. 148049283.

We appreciate the opportunity to be of service to Jacobs and look forward to working together in the future. Please feel free to 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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Abbreviations

FOS	factor of safety
SPT	standard penetration test
HDD	horizontal directional drilling

Executive Summary

This report presents the results of a geotechnical engineering study performed for the Runway 18 Approach Lighting project located at the Houma-Terrebonne Airport (HUM) in Houma, Louisiana.

1. Our geotechnical investigation consisted of drilling five (5) soil borings to depths between 12 and 60 feet below existing site grades and performing one (1) concrete coring within the existing blast pad.

2. Subsurface conditions at the site vary along the project alignment. Soils encountered in the borings include soft to stiff lean and fat clays (CL, CH), very soft organic clay (OH), silt (ML), and loose to dense silty sand, poorly graded sand with silt, and clayey sand. Depth to groundwater in the borings, after a 15-minute waiting period, ranged between 3 and 5 feet below existing grades at the time of our field investigation.

3. Per the 2021 IBC and ASCE 7-16, Seismic Site Class of 'E' is considered appropriate for the design of foundations.

4. Site grade will be raised between approximately 6 and 8 feet at the planned MALSR equipment shelter location. We anticipate that a 6-month settlement waiting period following completion of grade raise will be incorporated into the construction schedule. Temporary surcharge loads and/or ground improvement (e.g., wick drains) to accelerate settlements are not anticipated. Settlement monitoring should be performed during and after fill placement.

5. Lightly loaded equipment and structures may be supported on spread and strip footings. Recommended allowable bearing pressures (FS = 3.0 for sustained loading conditions) for square spread footings and strip footings are 1,000 pounds per square foot (psf) and 850 psf, respectively. Footings should bear a minimum of 1 foot into native soils and at least 2 feet below lowest adjacent finish grade.

6. The slab-on-grade at the MALSR equipment shelter location should bear on structural sand fill. Elsewhere on the project, slabs-on-grade should be underlain by at least 12 inches of structural clay fill. The applied pressures beneath slabs-on-grade should be limited to 500 psf.

7. Moderate to heavily loaded structures should be supported on deep foundations. Recommendations for 12-inch square precast concrete piles and 12-inch diameter auger cast-in-place (ACIP) piles are included herein.

8. Light-duty flexible pavements for the planned access roads and maintenance areas should consist of a minimum 4 inches of asphalt concrete over 6 inches of crushed limestone aggregate base placed on prepared subgrade. Light-duty rigid pavements should consist of a minimum of 6 inches of reinforced Portland cement concrete.

9. Due to shallow groundwater at the site, dewatering of foundation and utility excavations may be required depending on site groundwater levels at the time of construction.

This executive summary provides a brief presentation of some of the key recommendations of this report. It is by no means intended to be a comprehensive or stand-alone representation of the findings of our study. The full text of the report and the attached appendices contain important information which the reader should use to come to more informed conclusions about the information presented herein.

1. Introduction

1.1 Project Description

Fugro understands that Jacobs is assisting the Federal Aviation Administration (FAA) in the design and construction of new airport lighting structures located in Terrebonne Parish, Louisiana. A *Vicinity Map* is included on Plate 1. Fugro performed the scope of services described in detail below in general accordance with the provided Request for Proposal: Geotechnical Investigative Services for the Runway 18 Approach Lighting project located at the Houma-Terrebonne Airport (HUM).

Based on the information provided by Jacobs, the project includes construction of infrastructure for the replacement of the Runway 18 Medium Intensity Approach Lighting System with Runway Alignment Indicator Lights (MALSR) on Runway 18 at HUM. This also includes the installation of concrete foundations for an equipment shelter and lights at various locations in the proximity to the runway, and an HDD utility crossing in the vicinity of Terrebonne Bayou. In addition, pavements will be constructed for access roads and maintenance areas.

Based on provided plan and profile information, existing grades along the lighting alignment range from approximately El. +5 to +9 ft, except in the vicinity of Terrebonne Bayou where a bottom elevation of approximately El. -5 ft is indicated. As shown on a conceptual grading and drainage plan provided by Jacobs, grade will be raised between approximately 6 and 8 feet at the MALSR equipment shelter location. The shelter structure is planned to be supported on a slab on grade foundation. Jacobs provided Fugro with the proposed boring locations and requested Fugro perform a geotechnical investigation to support the project.

1.2 Scope of Work

The purposes of our geotechnical study were to 1) explore and evaluate the subsurface soil and groundwater conditions at the proposed exploration locations, and 2) provide geotechnical recommendations to assist Jacobs in the design and construction of foundations for the proposed structures.

We accomplished these purposes by:

- Drilling a total of five (5) soil borings. One (1) soil boring to a depth of 60-ft below existing site grade, three (3) soil borings to a depth of 40-ft, and one (1) soil boring to a depth of 12-ft. Boring locations and depths were provided by Jacobs.
- Performing one (1) concrete coring within the existing blast pad at a location provided by Jacobs.

- Performing field and laboratory tests on selected soil samples to assess pertinent geotechnical engineering soil properties.
- Analyzing field and laboratory test results to develop engineering recommendations for this project.
- Preparing this report, summarizing our findings and recommendations.

Environmental assessment, compliance with state and federal regulatory requirements, assessment of potential migration, and/or environmental analyses, including those associated with biological agents, were beyond the scope of this study. We were neither asked nor authorized to evaluate the environmental conditions at the site. A geologic fault study was also beyond the scope of this project.

1.3 Applicability of Report

The explorations and analyses for this study and the conclusions and recommendations presented in this report were selected or developed based on our understanding of the project, as described above and in later sections of this report. If pertinent details of the project have changed or otherwise differ from our descriptions, we request notification and authorization to review the changes and modify our conclusions and recommendations if needed.

We have prepared this report exclusively for Jacobs to use as a guide for the design's geotechnical aspects and construction of proposed structures. We have conducted this study using the standard level of care and diligence normally practiced by recognized engineering firms performing similar services under similar circumstances at the same time and locality. We intend for this report, including all illustrations and appendices, to be used in its entirety. Fugro makes no claim or representation concerning any activity or condition falling outside the specified purposes to which this report is directed. 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. This report should be made available for information only and not as a warranty of subsurface conditions.

1.4 Variations in Subsurface Conditions

Our interpretations of soil and depth-to-water conditions, as described in this report, are based on data obtained from our 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, especially the extent and depth of any surficial conditions and possible fill materials. 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.

2. Field Exploration

This section includes information relating to our field exploration activities for this project. We have included a general discussion on drilling methods, sampling methods, depth-to-water observations, and borehole completion.

2.1 General

Five (5) soil borings (designated as B-1 through B-5) and one (1) pavement core (designated as P-1) were performed for the project. The pavement core was retained by FAA personnel. The boring target locations and depths were provided by Jacobs. The soil boring and core locations were located and staked in the field by FAA and Jacobs onsite personnel using a handheld GPS. The borings were drilled on March 8 and March 9, 2024, with an ATV-mounted drilling rig using a combination of dry-auger and wet-rotary drilling techniques. The as-drilled boring locations are presented on the *Exploration Plan* on Plate 2. The boring logs for this study are presented on Plates 3 through 7 with a key identifying the terms and symbols used on Plates 8a and 8b. The coordinates for the explorations and approximated (from Google Earth) ground surface elevation at exploration locations are indicated on the logs.

2.2 Soil Sampling Methods

Soil samples from soil borings were generally taken continuously from the existing grade to a depth of about 16 feet and at 5-ft intervals thereafter to the boring completion depths. Borings were logged in general accordance with the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)* (ASTM D2488).

Our field procedure for cohesive soil sampling was conducted in general accordance with the *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes* (ASTM D1587). Undisturbed samples of cohesive soils were generally obtained by hydraulically pushing a 3-inch-diameter, thin-walled tube a distance of about 24 inches. The samples were extruded in the field and visually classified by one of our geotechnical personnel. Field estimates of the undrained shear strength of the recovered samples were obtained by using a hand penetrometer and/or torvane. Portions of each recovered soil sample were placed into appropriate containers for transportation to our laboratory.

Our field procedure for sampling cohesionless and select cohesive soils was conducted in general accordance with the *Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils* (ASTM D1586). A description of the SPT tests is presented on Plate 8b. An automatic hammer was used to obtain the hammer blows for each SPT sample. Our geotechnical personnel recorded the hammer blows for each sampling interval. The SPT N-value, described on

Plate 8b, is recorded on the boring logs and is uncorrected. The soil samples obtained from the split barrel sampler were visually classified and packaged for transportation to our laboratory.

2.3 Short-Term Depth-to-Water Observations

Each boring was initially drilled using dry-auger techniques to determine depth-to-water levels at the project site. Once water was encountered, drilling was temporarily stopped, and water level readings were obtained. Drilling then continued to the boring completion depth using the wet-rotary method. The interpreted depth-to-water conditions are discussed further in Section 4.3 of this report and are noted on the boring logs (Plates 3 through 7).

2.4 Borehole Completion

The soil borings performed for this study were backfilled upon completion with cement-bentonite grout from the bottom up. When grout returned to the surface, the excavations were topped off by pouring grout from the surface.

3. Laboratory Testing

The laboratory testing program for this study was directed primarily toward evaluating the classification properties, undrained shear strength, and compressibility characteristics of the foundation soils. Our laboratory tests were performed in general accordance with the appropriate standards as shown in Table 3.1.

Table 3.1: Summary of Tests Performed and Methods

Test Type	Test Destination
Moisture Content	ASTM D2216
Atterberg Limits	ASTM D4318
Percent Finer than a No. 200 Sieve	ASTM D1140
Sieve Analysis	ASTM D6913
UU-Triaxial Compression	ASTM D2850
One-Dimensional Incremental Consolidation	ASTM D2435
pH	ASTM D4972
Electrical Resistivity	ASTM G-57
Sulfate Content	ASTM D516
Chloride Content	ASTM D512

3.1 Classification Tests

The classification tests included tests for natural moisture content, liquid and plastic limits (collectively termed Atterberg Limits), sieve analysis, and percent finer than a No. 200 sieve. 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 presented on the boring logs on Plates 3 through 7, in Appendix A, Summary of Test Results, and in Appendix B, Grain Size Curves.

3.2 Undrained Shear Strength Tests

We measured the undrained shear strength of select undisturbed samples of cohesive soils by performing unconsolidated undrained triaxial compression tests. Natural moisture contents and dry unit weights were determined as routine parts of the compression tests. The results of the laboratory shear strength tests, along with the field estimates of shear strengths, are presented on the boring logs on Plates 3 through 7, and in Appendix A, Summary of Test Results.

3.3 One-Dimensional Incremental Consolidation Test

We measured the compressibility characteristics of the foundation soils by performing one (1) one-dimensional incremental consolidation test. Natural moisture contents and dry unit weights were routinely determined for the test samples. Consolidation test results and plots of effective vertical stress versus void ratio, are presented in Appendix C, Consolidation Test Curve.

3.4 Corrosion Tests

Three (3) corrosion series tests were performed for this project. Corrosion series tests included pH, resistivity, chlorides, and sulfates. Corrosion testing was performed in Fugro's Houston, Texas laboratory. The results of the laboratory corrosion tests are discussed in Section 10 and are presented in the Appendix D, Corrosion Test Results.

4. General Site Conditions

This section contains interpreted site and subsurface conditions based on our field exploration and laboratory testing programs. This section also includes a discussion and interpretation of the depth-to-water conditions encountered at the site during our field exploration.

4.1 Site Description

The project site/alignment is located at the existing Houma-Terrebonne Airport, beginning near the northern end of Runway 18 and generally extending northward past the intersection of Aviation Road and East Main Street, across Bayou Terrebonne, and terminating at East Park Avenue. Jacobs provided Fugro with the pavement core and boring locations. Pavement core P-1 was located within the existing blast pad near the northern terminus of Runway 18. Boring B-1 was located in the grass field northeast of the existing Runway 18 and directly east of the existing approach lighting structures. Borings B-2, B-3, and B-4 were located in the grassy area within the alignment of the existing approach lighting structures extending from Runway 18 north across Bayou Terrebonne. Boring B-5 was located along the southern side of East Park Avenue and approximately 250 feet southeast of the intersection between East Park Avenue and Lafayette Woods Boulevard. Overhead powerlines were located within the vicinity of Boring B-5. A *Vicinity Map* provided on Plate 1, depicts the site's general location in reference to East Main Street and East Park Avenue. The site was generally flat and covered with minor vegetation at the time of our field exploration. Existing access roads are gravel-surfaced. The *Exploration Plan*, provided on Plate 2 shows the approximate locations of the borings, however Fugro did not survey the boring locations.

4.2 Subsurface Conditions

A generalized subsurface profile across the site is presented on Plate 9. Based on the review of laboratory test results, the following is a general description of the soils encountered in the borings along with their Unified Soil Classification System (USCS) designation.

Boring B-1 generally consisted of alternating layers of lean and fat clay (CL, CH) and silt (ML) from the existing ground surface to a depth of 18.5 ft, followed by a layer of silty sand (SM) to a depth of 43.5 ft, underlain by a layer of fat clay (CH) to the completion depth of the boring. Boring B-3 disclosed a layer of lean clay (CL) from the existing ground surface to a depth of 18.5 ft, followed by relatively thin layers of silty sand (SM), poorly graded sand with silt (SP-SM), and clayey sand (SC) to a depth of 38.5 ft, underlain by a layer of fat clay (CH) to the completion depth of the boring. Borings B-1 and B-3 were drilled to depths of 60 ft and 40 ft, respectively. The strength tests and N-values conducted on cohesive materials generally indicated a soft to stiff consistency. The N-values in the cohesionless layer indicated a loose to dense consistency.

Borings B-2 and B-4 generally consisted of alternating layers of lean and fat clays (CL, CH) from the existing ground surface to the completion depth of the boring. Boring B-5 disclosed a lean clay (CL) fill layer from the existing ground surface to a depth of 2 ft, followed by a layer of native lean clay (CL) to a depth of 3 ft, underlain by a layer of silt (ML) to the completion depth of the boring. Notable details included approximately 4 ft of organic clay (OH) encountered in B-2 at 14 ft below existing grade and 5 ft of wood encountered in B-4 at 23.5 ft below existing grade. Borings B-2 and B-4 were drilled to a depth of 40 ft and Boring B-5 was drilled to a depth of 12 ft. The strength tests and N-values conducted on cohesive materials generally indicated a very soft to very stiff consistency.

Detailed information relating to the subsurface conditions encountered at the site is presented on the boring logs on Plates 3 through 7. A key identifying the terms and symbols used on the boring logs is presented on Plates 8a and 8b.

4.3 Depth-to-Water Conditions

To estimate the depth-to-water at the project site, soil borings for this study were initially drilled using dry auger drilling techniques. A summary of the groundwater measurements is presented in Table 4.1.

Table 4.1: Summary of Groundwater Measurements

Boring ID	Depth to Initial Groundwater (ft)	Depth of Groundwater After 15-minute waiting period (ft)
B-1	10.0	3.9
B-2	10.0	5.0
B-3	8.0	3.9
B-4	10.0	3.0
B-5	N/E	N/E
N/E = Groundwater was not encountered.		

It should be noted that short-term water levels recorded in the open boreholes should not be considered to represent a long-term condition. It is possible that the water levels in the open boreholes may not have had enough time to completely stabilize. More accurate determinations of groundwater levels are usually made from long-term standpipe piezometer readings; however, the water level is likely to be influenced by the adjacent bayou.

4.4 Seismic Site Classification

In accordance with the 2021 International Building Code (IBC) and ASCE 7-16, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site. As part of the procedure to evaluate seismic forces, the code requires the

evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100-ft of the ground surface. The borings for the project extended to a maximum depth of 60 feet, therefore we extrapolated soil conditions from the maximum depth of the borings to the depth required for site classification purposes. The site classification for seismic design was based on correlations with the shear strength values and standard penetration test (SPT) blow counts obtained from the soil borings. Site specific dynamic soil properties were not obtained as part of our study. We recommend a Site Classification 'E' be applied for this project.

5. Site Preparation

This section presents our recommendations associated with site drainage, subgrade preparation, site grading, the results of our area settlement analysis due to the proposed site grade raise at the MALSR equipment shelter location, and settlement monitoring.

5.1 Site Drainage

Establishing proper drainage and removing ponded water, where present at the time of construction, will be a key aspect of site preparation. Prior to construction, an adequate drainage plan should be provided to reduce the potential to saturate the subgrade soils. Most subgrade problems are attributable, at least in part, to poor site drainage. We recommend that an effective "site drainage plan" be devised by others prior to commencement of construction to provide positive drainage away from the planned limits of construction and off the site, both during and after construction. The drainage plan should consist of sloping the subgrade in conjunction with the use of lateral ditches and drainage inlets to transmit water away from the features associated with this project and offsite, as necessary.

5.2 Subgrade Preparation

Subgrade preparation should consist of stripping and discarding existing pavements and crushed stone, vegetation/organic materials, topsoil, debris, weak or soft soils, fill soils and any deleterious materials to expose competent, stiff natural, stiff cohesive soils capable of supporting construction traffic and subsequent backfill activities. Based on our borings, a minimum stripping depth of between 6 and 8 inches is recommended; however, this depth should be revised as necessary at the start of site grading as surficial soils are exposed. If present, trees and their entire "root-ball" systems should also be completely removed from any structure and pavement locations. Any existing utilities and other underground obstructions should be identified and relocated as necessary to facilitate construction activities.

Stripped subgrades in structure and paving areas should then be moisture conditioned as necessary to achieve a moisture content between 1 and 4 percent wet of optimum. The moisture content at the time of construction should be monitored. If the moisture content is dry of optimum, the upper 8 inches of soil should be scarified, followed by moisture conditioning and recompaction in accordance with Section 12.3 of this report.

The Geotechnical Engineer-of-Record, or their qualified representative, should observe the exposed subgrade during site preparation. Where possible, onsite observation should include proof rolling the exposed subgrade with a heavy (20 ton), rubber-tired vehicle (e.g., fully loaded dump truck) to evaluate the condition of the subgrade. Areas of the subgrade that are observed to be soft, wet, weak, or contain deleterious materials should be over-excavated to expose

competent, cohesive soils and replaced using structural sand fill, structural clay fill, lime-stabilized clay fill, or flowable fill. Material and placement requirements for structural sand fill, structural clay fill, lime-stabilized clay fill, and flowable fill are provided in Section 12.3 of this report. In confined areas where proofrolling is not feasible (such as small foundation excavations), visual observation and probing should be conducted to verify the competency of the foundation soils. The Geotechnical Engineer-of-Record, or their qualified representative, should observe proofrolling and/or probing operations to detect weak soils. If the subgrade is deemed to be competent via a properly performed proofrolling or probing operation, then backfill operations may commence.

5.3 Site Grading

Per Jacobs, grade will be raised between approximately 6 and 8 feet at the planned MALSR equipment shelter location. Elsewhere, fill depths on the project are expected to not exceed approximately 2 feet. At the shelter location, we recommend that structural sand fill (Section 12.3.1) be placed in compacted lifts to the proposed finish grade. Fill slopes be inclined no steeper than 4H:1V (horizontal:vertical).

We anticipate that a 6-month settlement waiting period following completion of grade raise will be incorporated into the construction schedule. Temporary surcharge loads and/or ground improvement (e.g., wick drains) are not required to accelerate settlements unless the estimated post-construction settlements presented in Section 5.4 are not acceptable for the proposed structure.

5.4 Settlement Due to Site Grade Raise

Raising grade at the proposed equipment shelter location will incur significant settlements within the underlying foundation soils. Settlements will occur over time, both during and following fill placement, and will vary across the grade raise area. In general, settlements due to grade raise will include settlement within the fill under its own weight and settlement of subgrade soils due to the weight of the fill. Settlement analysis from the self-weight of the structural sand fill was not performed as these immediate settlements are considered negligible compared to primary consolidation settlements. This subsection presents soil design parameters used in the settlement analyses, the settlement analysis method, and computed settlements with estimated time-rate of settlement.

5.4.1 Soil Compressibility Parameters

Soil compressibility parameters used in the settlement analyses are presented in Table 5.1. We relied on laboratory data in conjunction with established correlations from the local geotechnical literature for similar soils to assign compressibility parameters.

Table 5.1: Design Parameters for Area Settlement Analyses – MALSR Equipment Shelter

Top Depth [ft]	Bottom Elevation [ft]	Soil Description	Total Unit Weight γ_t [pcf]	E_s [ksf](1)	C_c(2)	C_r(3)	e_0(4)	P'_c [ksf] (5)
0	4	Fat Clay (CH)	115	-	0.30	0.03	0.95	6.7 to 3.5
4	19	Lean Clay and Silt (CL, ML)	118	-	0.23	0.05	0.87	3.5 to 2.1
19	43	Silty Sand (SM)	120	400	-	-	-	-
43	60	Fat Clay (CH)	100	-	0.68	0.14	1.59	3.1 to 4.9

Notes:
1 = 1D Modulus (Constrained modulus)
2 = Compression Index
3 = Re-compression Index
4 = Initial void Ratio
5 = Preconsolidation Pressure

5.4.2 Computed Consolidation Settlement and Time Rate of Settlement

Long-term consolidation settlements will occur over a period of several years as the cohesive soils consolidate. In general, the consolidation process begins with the application of load on the soil resulting in the generation of excess pore water pressures. The excess pore water pressures gradually dissipate due to the expulsion of water from the soil voids accompanied by a concurrent increase in the effective stress of soils. As the excess pore water dissipates, the load is transferred to soil particles compressing the soils and resulting in settlement. The rate of settlement is a function of soil characteristics such as compressibility, permeability, and the distance of the drainage path for the excess pore water to travel.

We calculated the settlements of the foundation soils due to the weight of the fill using the computer program Settle 3 developed by Rocscience. This program first computes net stress changes at selected locations and depths beneath loaded areas using Boussinesq's theory of stress distribution. Then, the software calculates consolidation settlements of clay by computing net stress changes at selected locations and depths beneath the loaded areas. The program uses soil compressibility parameters to evaluate the change in thickness of individual layers and computes the overall movement at the selected locations. Immediate settlements were calculated within cohesionless soil layers, and primary consolidation settlements were calculated within cohesive layers.

Our settlement model incorporates the assumptions listed below.

- An embankment load with approximate top plan dimensions of 45 feet by 45 feet and 4H:1V side slopes
- Structural sand fill unit weight of 120 pounds per cubic foot (pcf);

- Incremental placement of up to 8 feet of sand over a period of 30 days; and
- Settlement was evaluated to the maximum explored depth of 60 feet below existing site grades. Loading stresses below this depth are considered minor and therefore, settlement below this depth is considered negligible.

Table 5.2 below provides estimated settlement of the subsurface soils over a period of 3 years upon completion of grade raise due to the weight of structural sand fill for the anticipated fill height.

Table 5.2: Estimated Settlement Due to Site Grade Raise

Area	Total Settlement at Ground Surface, inches			
	1 Month after COG[1]	6 Months after COG	1 Year after COG	3 Years after COG
MALSR Shelter	2.4	3.5	3.7	3.9

[1] COG = Completion of Grade Raise

5.4.3 Variations in Settlement Analysis

We performed detailed settlement analyses using our interpreted design parameters based on the soil conditions encountered during this study and the anticipated loading from fill placement. Actual compressibility characteristics of natural soils may vary and differ from what we assumed in our analyses leading to variation in our settlement estimates. The actual post-construction settlements will vary depending on several factors such as construction period, variability of permeability and compressibility of natural soils, and the actual amount of fill placed to achieve design grade. As such, differential settlements should be expected within the project site because of variable consolidation characteristics of the soils.

5.5 Consolidation Settlement Monitoring

Settlement monitoring can be used to compare predicted and actual settlements and may be used to estimate future volumes of fill needed to achieve the final site grade. We recommend that the results of the settlement-monitoring program be reviewed by the Geotechnical Engineer during the construction phase. Based on our time rate of settlement estimates, we recommend monitoring settlements for at least 6 months after completion of grade raise.

Settlement plates can be used to monitor near-surface settlements. Please note that settlement plates must be installed prior to placement of fill to provide meaningful data. As such, the instrumentation must be modified to accommodate the grade increase as it occurs and must also be protected from construction equipment during fill placement. Monitoring of the instrumentation and subsequent evaluation must be performed by the Geotechnical Engineer. At least two (2) settlement plates, at opposing corners of the filled area, should be installed to monitor the progression of settlement as the site grade is raised.

6. Soil Parameters for Engineering Design

The project consists of infrastructure for the replacement of the Runway 18 Medium Intensity Approach Lighting System with Runway Alignment Indicator Lights (MALSR) on Runway 18 at HUM and equipment shelters and lights at various locations in the proximity to the runway. This section summarizes design soil parameters used in the engineering analyses and presents soil parameters for use by others in the design of the HDD utility crossing below Bayou Terrebonne.

6.1 Design Soil Properties - General

Soil parameter profiles were prepared based on the results of the field and laboratory test programs. Due to the variable soil conditions across the site, two separate soil profiles were developed for this study and are shown in Tables 6.1 and 6.2.

Table 6.1: Design Soil Parameters (based on Borings B-1 and B-3)

Depth	Soil Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
0 to 5	Lean to Fat Clay (CL,CH)	115	700	-
5 to 18	Lean Clay (CL)	115	500	-
18 to 33	Silty Sand (SM, SP-SM)	120	-	28
33 to 38	Silty to Clayey Sand (SM, SC)	120	-	30
38 to 43	Fat Clay (CH)	100	500	-
43 to 60	Fat Clay (CH)	100	700	-

Table 6.2: Design Soil Parameters (based on Borings B-2, B-4, and B-5)

Depth	Soil Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
0 to 5	Lean to Fat Clay, Silt (CL, CH, ML)	115	700	-
5 to 10	Lean to Fat Clay, Silt (CL, CH, ML)	115	500	-
10 to 20	Lean to Fat Clay, Organic Clay (CL, CH, OH)	90	500	-
20 to 40	Lean to Fat Clay (CL, CH)	100	500	-

6.2 Design Soil Properties – HDD Utility Crossing

Soil parameters presented in Tables 6.3 and 6.4 may be used by others for engineering evaluation of the HDD utility crossing.

Table 6.3: Design Soil Parameters for HDD Utility Crossing (based on Boring B-3)

Depth	Soil Type	Unit Weight (pcf)	Cohesion at Top of Layer (psf)	Cohesion at Bottom of Layer (psf)	Friction Angle (degrees)	Estimated Shear Modulus (ksf)
0 to 5	Lean Clay (CL)	115	700	700	-	140-720
5 to 18	Lean Clay (CL)	115	500	500	-	50-280
18 to 33	Silty Sand (SM, SP-SM)	120	-	-	28	570-1,000
33 to 38	Clayey Sand (SC)	120	-	-	30	1,000-1,500
38 to 40	Fat Clay (CH)	100	500	500	-	50-280

Note:
Soil parameters interpreted based on results of geotechnical laboratory testing and correlations with strength parameters. Range of estimated shear modulus values based on empirical relationships presented in FHWA GEC No. 5, *Evaluation of Soil and Rock Properties*. Shear wave velocity measurements should be performed for more accurate determinations of shear modulus, as needed.

Table 6.4: Design Soil Parameters for HDD Utility Crossing (based on Borings B-4 and B-5)

Depth	Soil Type	Unit Weight (pcf)	Cohesion at Top of Layer (psf)	Cohesion at Bottom of Layer (psf)	Friction Angle (degrees)	Estimated Shear Modulus (ksf)
0 to 5	Fill & Native: Lean to Fat Clay (CL, CH)	115	700	700	-	140-720
5 to 10	Silt & Lean Clay (ML, CL)	115	500	500	-	50-280
10 to 20	Silt & Lean Clay (ML, CL)	90	500	500	-	50-280
20 to 40	Fat Clay (CH), Wood	100	500	500	-	50-280

Note:
Soil parameters interpreted based on results of geotechnical laboratory testing and correlations with strength parameters. Range of estimated shear modulus values based on empirical relationships presented in FHWA GEC No. 5, *Evaluation of Soil and Rock Properties*. Shear wave velocity measurements should be performed for more accurate determinations of shear modulus, as needed.

7. Shallow Foundations

We understand that the lightly loaded equipment and structures may be supported on shallow foundations and/or shallow slabs-on-grade assuming the equipment and structures can accommodate the anticipated settlement presented herein. From a geotechnical perspective, the performance of a shallow foundation system should provide an adequate factor of safety against shear failure of the foundation soils and reduce the potential for excessive settlements due to stressing of the underlying foundation soils. As such, shallow foundations should be designed such that their applied bearing pressures do not exceed the allowable bearing pressure of the underlying soils. In addition, settlement resulting from the applied pressures (both total and differential) should be considered.

7.1 Foundation Depth

Shallow spread and strip footings should bear on competent in-situ soils at a depth of at least 2-ft below the lowest adjacent finished grade and at least 1-ft into undisturbed, natural in-situ soils (Footings should not be placed on fill). Frost depth is not a concern at this site with regard to foundation embedment. To mitigate differential settlement, foundations for any particular structure should bear on similar materials and not a combination of different materials, such as fill and native soils. If weak, soft, or otherwise unsuitable soils are encountered at the proposed foundation depths, these soils should be over-excavated and replaced with flowable fill. Based on groundwater levels at the time of our field investigation, foundation excavations greater than 2 feet in depth may encounter groundwater.

7.2 Allowable Bearing Pressure

The allowable bearing pressure for shallow foundations is a function of, among other items, the depth of the foundation, the strength of the foundation soils, the shape of the foundation, and the recommended factor of safety. We are providing allowable bearing pressures for shallow spread and strip footings, under the various loading conditions in the table on the following page. These allowable bearing pressures do not consider the potential settlement associated with the applied pressures. Our settlement estimates for various foundation sizes corresponding to different applied pressures are presented later in this section.

Table 7.1: Allowable Bearing Capacity Recommendations

Allowable Bearing Pressure Recommendations				
Foundation Type *	Foundation Embedment†	Loading Conditions#		
		Sustained (F.S‡= 3)	**Total (F.S. = 2)**	**Transient (F.S. = 1.5)**
Square Spread Footings	2 ft	1,000 psf	1,500 psf	2,000 psf
Rectangular Spread Footings and Strip Footings	2 ft	850 psf	1,250 psf	1,650 psf

Notes
* = The allowable bearing pressures presented are applicable for square and rectangular spread footings (widths and lengths between about 5 ft and 15 ft) and strip footings (widths of at least 1.5 ft).
† = Spread and strip footings should bear on natural, firm to stiff cohesive soils a minimum of 2 ft below final grade and should have an embedment of at least 1 ft below into natural, undisturbed soils. See Section 7.1.
‡ = F.S. refers to factor of safety.
# = Refer to report text for additional information relating to loading conditions.

Sustained loading conditions as described in this report refer to the combination of properly factored dead and, in some cases, a portion of the sustained live loads. Total loading conditions as described in this report refers to the combination of properly factored dead and live loads. Transient loading conditions refer to the combination of properly factored dead, live, and infrequent transient (i.e., wind and seismic) loads.

With respect to failure of the foundation soils, the allowable bearing capacities in the previous table include a factor of safety of at least 3 for sustained loading, 2 for total loading, and 1.5 for transient loading. Foundations should be proportioned so that the maximum contact pressure under various load combinations does not exceed the allowable bearing capacity given previously. Net bearing pressure and gross bearing pressure are defined on Plate 10 of this report. To calculate values of W_e, W_s, W_f from Plate 10, we recommend using buoyant unit weights of 60 pcf for soil and 90 pcf for reinforced concrete.

7.3 Resistance to Uplift Loads

The uplift capacity of a shallow foundation formed in an open excavation will be limited to the weight of the foundation and any soil directly above the foundation. For design purposes, the ultimate uplift resistance may be based on buoyant unit weights for soil and concrete of 60 pcf and 90 pcf, respectively. We recommend that the calculated ultimate uplift capacity be reduced by a factor of safety of 1.2 to compute the allowable uplift resistance.

7.4 Resistance to Lateral Loads

Horizontal loads acting on a shallow foundation formed in an open excavation will be resisted by passive earth pressure acting on one side of the foundation and by adhesion at the base.

However, we recommend neglecting passive pressure unless area paving, consisting of at least 4 to 6 inches of concrete or hot-mix asphaltic concrete extending out at least 5 feet from the edge of the foundation is provided on all sides. For design purposes, the resistance due to adhesion and passive soil pressure can be assumed to develop simultaneously.

We should be contacted for additional discussions on passive pressure if pavements, as discussed above will be utilized. For concrete foundations poured in good contact with undisturbed, natural firm to stiff cohesive soils as previously discussed, the allowable base adhesion may be taken as 175 psf. For the soil conditions encountered at this site at the recommended depth, we feel this allowable value will provide a factor of safety of at least 2 with respect to the ultimate lateral resistance.

7.5 Resistance to Overturning Moments

The contact pressure distribution of shallow foundations subject to overturning moments will be trapezoidal. The design of the foundation should be such that 1) full contact and positive pressure is maintained below the foundation base and 2) the maximum contact pressure does not exceed the allowable bearing capacity recommendations previously presented for the foundation soils.

7.6 Estimated Settlement

A detailed settlement analysis was beyond the scope of this study. However, based on our experience, we estimate the settlement due to soil compressibility from structural loads of properly designed and constructed shallow spread footings less than about 10-ft by 10-ft with a sustained net pressure less than 1,000 psf to be less than about 1-inch if founded on competent natural soils. Estimated settlement is reduced to ½-inch or less for a sustained net pressure value of approximately 500 psf.

In addition to the settlement induced by structure loads, if footings are placed in or immediately adjacent to areas where the site grade has been recently raised, additional settlement can be expected due to consolidation of the underlying soils from placement of the fill. The amount of this additional settlement will depend on the amount of grade raise at the footing location.

All things equal, differential settlements between individual foundations should be expected and may be on the order of one-half of the total settlement. However, differential settlements between foundations may be greater if there are variances in subsurface conditions, loading conditions, and construction procedures. If shallow foundations are selected for use on the project, we would be pleased to perform a detailed settlement analysis to evaluate differential movements on a case-by-case basis.

7.7 Slabs-on-Grade

We understand that selected structures may be supported on a slab-on-grade foundation with turned-down edges. The reinforcement design in the slab-on-grade will be dependent on the anticipated slab loading. For lightly to moderately loaded slabs conventional reinforcement will likely be suitable, whereas for heavily loaded slabs additional reinforcement, up to a structural slab, may be required. We recommend limiting the applied pressure beneath slabs-on-grade to 500 psf. Applied pressures greater than 500 psf will likely require the use of supplemental foundations such as spread footings. The following subsections provide recommendations for the design and construction of a slab-on-grade foundation.

7.7.1 Subgrade Preparation

To reduce the potential for foundation distress, the subgrade below the slab-on-grade foundation should be properly prepared including stripping all significant organic and other deleterious materials. The moisture content at the time of construction should be monitored. If the moisture content is dry of optimum, moisture conditioning may be required.

Before grade is raised, the exposed subgrade should be proofrolled and observed for areas that appear to be soft, wet, weak, or containing deleterious materials. In confined areas where proofrolling is not feasible (such as small foundation excavations), probing should be conducted to verify the competency of the foundation soils. The Geotechnical Engineer-of-Record, or their qualified representative, should observe proofrolling and probing operations to detect weak soils.

Weak, wet, and otherwise deleterious soils should be removed and replaced with properly compacted structural clay fill. The removal and replacement activities should extend laterally at least 3 feet beyond the edge of the slab footprint. If the subgrade is deemed to be competent via a properly performed proofrolling or probing operation, then backfill operations may commence. The exposed subgrade should then be brought to design grade using properly placed and compacted structural clay fill. The objective is to create a uniform, low permeability, relatively homogeneous foundation base of low plasticity structural clay fill for the slab-on-grade. All slabs-on-grade should be underlain by a minimum of 12 inches of structural clay fill to provide uniform support across the slab area. We recommend that concrete be placed soon after the construction of the slab on grade pad to protect the pad from the influence of the weather. Note that the slab-on-grade for the MALSR equipment shelter will bear on structural sand fill due to the planned grade raise in this area.

7.7.2 Grade Beams

Exterior grade beams (or turned down edges) and interior grade beams should be used to support loads between footings and to stiffen floor slabs. The depth of exterior and interior

grade beams can be varied according to the structural requirements of the floor slab. However, we recommend that exterior grade beams (or turned down edges) for the slab-on-grade foundations extend at least 24 inches below lowest adjacent grade. Exterior and interior grade beams should be designed using an allowable net bearing pressure of 1,000 psf. We do not recommend the use of void boxes below grade beams for slab-on-grade foundations because of the potential to collect free water within the void space. Grade beams may also be used as moisture barriers for the slab-on-grade foundation.

7.7.3 Additional Slab-on-Grade Considerations

Moisture content fluctuations resulting from the presence of vegetation may significantly affect the performance of the clay soils at the site thereby affecting the performance of the structure. As such, trees and other vegetation that require large amounts of water from the subsurface to survive should be kept away from the proposed structure. The water demand of such vegetation may contribute to significant moisture content fluctuations within near surface cohesive soils thus increasing the potential for shrink/swell movements.

The following design details are also recommended to reduce the risk of slab-on-grade damage from shrinking and swelling soils:

1. Grade the site to provide positive drainage away from the structures and slabs-on-grade. Grade supported slabs should be raised above the adjacent grade, which should be sloped a minimum of 2 percent away from slabs-on-grade for a distance of at least 5 ft from the exterior of the structure. Water should not be allowed to pond adjacent to the slab. It is also important to provide positive drainage during the construction of the foundation system to protect the exposed subgrade from moisture content fluctuations.
2. Maintain uniform moisture conditions in the exposed subgrade soils prior to construction of the floor slab. The exposed subgrade soils should not be allowed to dry out or to become wet during construction.
3. Backfill all utility trenches or potential conduits of water underneath floor slabs using structural clay fill within a distance of 5 ft outside and adjacent to floor slabs. Utility trenches have been identified as conduits that permit the migration of moisture under slabs. Where utility trenches extend out from underneath the floor slabs, we recommend that a bentonite clay plug be placed in the trench to restrict exterior water from migrating beneath the slab.
4. Provide additional grade beams below interior and exterior walls as well as provide extra reinforcement in the slab to increase rigidity, thereby resisting differential movements below walls and across the slab.
5. Downspouts should be extended to discharge water away from the foundations.
6. Irrigation systems should be designed and monitored so as not to over water around the foundations.

If the vertical movements for shallow foundations discussed herein are beyond tolerable limits for a slab-on-grade foundation, then the floor slab for the structures should be constructed as a structural slab-on-grade foundation. A structural slab-on-grade foundation is generally thicker than a standard slab-on-grade and may include interior grade beams. The underlying soil is used only to form the underside of the structural slab-on-grade during construction. A structural slab-on-grade foundation will be more costly than a standard slab-on-grade. Ultimately, the type of slab selected will depend on the amount of risk the owner is willing to accept.

7.8 Coefficient of Subgrade Reaction

Often, the design of slab foundations and mat foundations involves the concentration of various magnitudes of point loads across the foundation. Typical analyses require a coefficient of subgrade reaction, which is defined as the ratio between the pressure at any given point on the surface of contact and the deformation produced by the load application at that point. For relatively small and rigid foundations, settlement analyses are generally performed considering only the size and magnitude of the applied load as well as the compressibility characteristics of the foundation soils. However, for large flexible foundations the response of the foundation to the load should also be considered (soil/structure interaction). For large slab and mat foundations, the applied stress can vary across the bottom of the foundation and is dependent on the location of the applied loads, the rigidity of the foundation, and stiffness of the foundation soil. Under these conditions the foundation will undergo some deflection that is not necessarily uniform across the foundation. The deflections of the foundation need to be similar to the foundation soil movements, which are associated with the applied stresses.

To evaluate the soil/structure interaction for large slab and mat foundations an iterative process between the Geotechnical Engineer and the Structural Engineer is usually required. The iterative process typically involves the Geotechnical Engineer determining the soil movements (and resulting moduli values) using an estimated pressure distribution or a pressure distribution provided by the Structural Engineer. The Structural Engineer then models the soil as a series of springs (using the moduli values provided by the Geotechnical Engineer for the spring stiffness) to determine the resulting mat deflections and new pressure distribution. The new pressure distribution is then used by the Geotechnical Engineer to reevaluate the soil movements (and resulting moduli). This process is continued until the soil movements estimated by the Geotechnical Engineer converge with the mat deflections computed by the Structural Engineer.

For preliminary analyses, input dimensions of the value of the coefficient of subgrade reaction for specific computer programs should be reviewed. Some programs require a subgrade value for a 1 ft by 1 ft plate while others require the subgrade value for the actual foundation size. For preliminary analyses, we are providing a coefficient of subgrade reaction value for a 1 ft by 1 ft plate. Terzaghi (1955) provides equations for computing the coefficient of subgrade reaction for foundations on clay.

Based on site specific soil data and published correlations for similar materials, the coefficient of subgrade reaction for a 1 ft by 1 ft plate can be taken as 98 kcf (57 pci) for the cohesive soils to a depth of about 5 ft below existing grade. These values can be used for both vertical and horizontal coefficients. Terzaghi also provides equations for adjusting the coefficient of subgrade reaction for a 1 ft plate to actual foundation dimensions.

8. Deep Foundation Recommendations

For heavily loaded structures, or in areas where predicted shallow foundation settlement will exceed tolerable amounts, deep foundations should be utilized. For deep foundation alternatives,12-inch square precast concrete piles and 12-inch diameter auger cast-in-place (ACIP) piles were evaluated. The following paragraphs include our recommendations for Axial Capacity, Factors of Safety and Verification, Lateral Load Analysis, Group Effects, and Estimated Settlement.

8.1 Axial Capacity

The ultimate axial capacity of individual, isolated deep foundations was computed using the static method of analysis. In this method, the ultimate compressive capacity for a given penetration is computed as the sum of the skin friction on the pile wall and the end bearing on the pile tip. We neglected the end-bearing in our analyses for auger cast-in-place piles because the movement associated with mobilizing the end bearing is typically beyond tolerable structural limits.

The ultimate compressive and tensile capacity was computed for 35-ft long, 12-inch square prestressed precast concrete (PCPS) piles using the computer program APILE (Reese et al, 2015). Ultimate axial capacity curves for cast-in-place piles were computed for 35-ft long, 12-inch diameter ACIP piles using the computer program SHAFT (Reese et al, 2017a and 2017b). The ultimate axial pile capacity curves are presented in Appendix E, *Ultimate Axial Capacity Curves*. The penetration should be referenced from the natural ground surface at the time of our exploration.

8.2 Factors of Safety and Verification

The recommended factors of safety for the deep foundations are as follows:

- We recommend a factor of safety of 3.0 be applied to the ultimate axial capacity for piles loaded in sustained and transient compression and tension with no verification testing.
- A factor of safety of 2.5 may be applied if dynamic testing using the Pile Driving Analyzer (PDA) is performed on a long-term restrike (greater than 3 days) on at least 5% of the production piles for capacity verification.
- A factor of safety of 2.0 may be applied provided at least one load test is performed (to failure or to a load exceeding the estimated axial capacity by 50%), and the site and subsurface conditions are reasonably uniform and do not pose difficult construction conditions. We recommend sonic integrity testing of drilled foundations as described below to provide quality assurance of drilled foundations.

- The reported axial capacities included herein are geotechnical capacities. The structural capacity of the reinforced concrete should be checked by the Structural Engineer.

8.3 Lateral Load Analysis

We performed a simplified evaluation of the lateral resistance for 12-inch square precast concrete piles and 12-inch diameter auger cast-in-place (ACIP) piles for the soil conditions encountered at the site. We performed our lateral load analyses using the computer program LPILE, developed by Ensoft. The following assumptions were made in our lateral load analyses of single, isolated pile:

- Young's Modulus equals 3,600 ksi for straight-sided concrete piles (f'_c = 4,000 psi);
- Lateral loads only were applied at the top of the pile; and
- No axial loads or external moments were applied at the top of the pile.

We assumed in our analyses that the piles are installed to a penetration of at least 30 feet below the existing ground surface. We analyzed both free-head and fixed-head conditions with lateral loads applied at the ground surface. We performed our analyses using limiting deflections (at the ground surface) of ¼ and ½ inch. We also evaluated the maximum bending moment in the pile section. We analyzed each case using both "uncracked" and "cracked" stiffness (EI). A "cracked" stiffness of 35 percent of the actual stiffness (EI) was used for our calculations as recommended by ACI 318-19. The computed lateral loads applied at the ground surface and the maximum internal bending moments for isolated, individual pile installed at least 30 ft below final grade, are provided in Tables 8.1 and 8.2.

Table 8.1: Lateral Load Resistance and Resulting Maximum Bending Moment (Borings B-1 and B-3)

Pile Section	Pile Top Fixity	Stiffness	Top Deflection ¼ inch		Top Deflection ½ inch	
			Lateral Load (kips)	Max Moment (in-kips)	Lateral Load (kips)	Max Moment (in-kips)
12-Inch-Square Precast Concrete Pile	FREE-HEAD	CRACKED	2.6	109.1	4.0	176.8
	FIXED HEAD	CRACKED	6.3	280.2	9.4	458.7
	FREE-HEAD	UNCRACKED	4.0	193.9	6.0	314.6
	FIXED HEAD	UNCRACKED	9.2	505.8	13.7	825.9
12-inch Diameter Auger Cast-In-Place Pile	FREE-HEAD	CRACKED	2.1	81.3	3.2	132.5
	FIXED HEAD	CRACKED	5.1	207.5	7.7	339.9
	FREE-HEAD	UNCRACKED	3.2	144.9	4.9	235.5
	FIXED HEAD	UNCRACKED	7.6	376.1	11.3	614.3

Table 8.2: Lateral Load Resistance and Resulting Maximum Bending Moment (Borings B-2, B-4, and B-5)

Pile Section	Pile Top Fixity	Stiffness	Top Deflection ¼ inch		Top Deflection ½ inch	
			Lateral Load (kips)	Max Moment (in-kips)	Lateral Load (kips)	Max Moment (in-kips)
12-Inch-Square Precast Concrete Pile	FREE-HEAD	CRACKED	2.6	109.1	4.0	176.8
	FIXED HEAD	CRACKED	6.3	280.3	9.4	458.7
	FREE-HEAD	UNCRACKED	4.0	193.9	6.0	315.8
	FIXED HEAD	UNCRACKED	9.2	507.0	13.7	827.5
12-inch Diameter Auger Cast-In-Place Pile	FREE-HEAD	CRACKED	2.1	81.3	3.2	132.5
	FIXED HEAD	CRACKED	5.1	207.5	7.7	339.9
	FREE-HEAD	UNCRACKED	3.2	144.9	4.9	235.5
	FIXED HEAD	UNCRACKED	7.6	376.2	11.3	615.0

8.4 Group Effects

The overall allowable load carrying capacity of a group of 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 piles width/diameter for axial loads, and 5 or more diameters for lateral loads. The reduction in individual capacity depends on many factors including the configuration of the group, number of piles in the group, piles size, the depth of installation, and the piles spacing. If piles are spaced closer than 3 to 5 diameters or if groups larger than about 5 by 5 are planned, we recommend that we be retained to review the final design and comment on group effect.

8.5 Estimated Settlement

A detailed settlement analysis was beyond the scope of this study. Based on our experience and the results of index tests performed on soil samples collected at the site we estimate post-construction settlements for properly installed individual isolated piles to be less than about 1/2 inch.

9. Pavements

We understand that flexible and/or rigid pavements will be constructed for planned access roads and maintenance areas. Per Jacobs, design traffic loading is considered 'light duty' with two passes per day of pickups and an occasional tractor-trailer (i.e., 18-wheeler) over a 20-year design life. This section presents discussions and recommendations on subgrade preparation, drainage, and pavement sections.

9.1 Subgrade Preparation

Subgrade preparation for pavement areas is discussed in Section 5 of this report.

9.2 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 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 future facility maintenance.

9.3 Pavement Sections

Table 9.1 provides recommended flexible and rigid pavements section based on the anticipated design traffic loading and subgrade soil properties. A design ESAL value of 25,000 was assumed for both flexible and rigid pavement designs based on the provided information. Design pavement sections for other ESAL values can be provided upon request. A California Bearing Ratio (CBR) value of '2' and modulus of subgrade reaction (k value) of 57 pci was assigned to the subgrade soils based on correlations with the subsurface conditions encountered in the borings performed for this project.

Table 9.1 Pavement Section Recommendations

Type and Recommended Use	Material	Minimum Thickness	Reference or Specification
Flexible Pavement – Light Duty (25,000 ESALs) (Access Roads & Maintenance areas)	Hot-Mix Asphaltic Concrete over	4 inches	LADOTD 502
	Crushed Limestone Aggregate Base over	6 inches	LADOTD 1003.03.01 Limestone
	Compacted Subgrade	-	See Text
Rigid Pavement – Light Duty (25,000 ESALs) (Access Roads & Maintenance areas)	Portland Cement Concrete (Flexural strength ≥ 550 psi) over	6 inches	LADOTD 601
	Compacted Subgrade	-	See Text

A nonwoven geotextile such as Mirafi 160N, or equivalent, should be placed at the interface of the properly prepared subgrade and aggregate base. The crushed limestone aggregate base should be in accordance with the 2016 edition of the LADOTD *Standard Specifications for Roads and Bridges*. The stone should be placed in 6- to 8-inch-thick loose lifts and uniformly compacted to at least 98 percent of the maximum dry density as determined by ASTM D698 (Standard Proctor) within 2 percent of optimum moisture.

Hot mix asphaltic concrete (HMAC) should be placed in accordance with LADOTD (2016 Item 502. The HMAC should be either a Type D or Type C surface course mix. The asphaltic concrete should be compacted to 95 percent of the theoretical density as described by LADOTD (2016).

Portland cement concrete pavement should be in accordance with LADOTD (2016) Item 601. The concrete should have a compressive strength of 4,000 psi or greater at 28 days and be placed in accordance with American Concrete Institute (ACI) guidelines. The Structural Engineer of Record for the project should evaluate reinforcement and joint spacing for the concrete section. Fugro recommends that the Portland cement concrete pavement be steel reinforced and that the concrete slabs have sufficient joints to allow for contraction and expansion of the concrete.

9.4 Site Grade Raise Effects on Pavements

Pavements located in areas of site grade raise will be subject to differential settlements and will have to be maintained and possibly repaved with the passage of time. To lessen potential for differential settlements it is recommended that final paving of permanent access roads and parking areas not be performed until settlement monitoring data, following completion of fill placement (for final grading), is evaluated by the geotechnical engineer and construction personnel to confirm that consolidation settlement is complete, or nearly so.

10. Soil Corrosion Potential

10.1 General

Steel and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried steel and concrete elements should be designed to resist corrosion and degradation based on accepted practices. Soil pH, electrical resistivity, and chloride (CL) and sulfate (SO4) ion concentration tests were performed on three (3) soil samples obtained from the borings drilled for this study. These tests were performed to assess the corrosion potential on buried steel and concrete. Fugro's laboratory test results are discussed in the following sections. The potential for corrosion of buried steel and degradation of buried concrete based on a comparison of the laboratory test results with published guidelines is also discussed. The guidelines used to evaluate corrosion and degradation potential are also presented. These results of corrosion tests are presented in Table 10.1.

10.2 Laboratory Test Results

The results of Fugro's laboratory corrosion potential testing are summarized in Table 10.1.

Table 10.1: Summary of Soil Corrosion Potential Results

Boring	Depth (ft)	pH	Sulfate (ppm)	Chloride (ppm)	Electrical Resistivity(ohm-cm)
B-1	2.0	6.9	<100	<100	2048
B-2	2.0	6.7	<100	<100	1249
B-3	2.0	7.9	<100	<100	2519

10.3 Corrosion of Buried Steel

The corrosion potential of steel is influenced by electrical resistivity, chloride ion concentration, and pH. Corrosion of steel is more likely in soil environments with low resistivity, high chloride ion concentrations, or low pH. For example, it is not necessary to have a resistivity between 0 and 1000 ohm-cm and a pH between 0 and 4.5 to indicate a "Very High" for corrosion potential. Table 10.2 presents general guidelines for estimating the corrosion potential of steel as a function of chloride ion concentration, pH, and electrical resistivity.

Table 10.2: Corrosion Potential Based on Resistivity, pH, and Chloride Content

Resistivity, ohm-cm	Chloride Content (ppm)	pH	Corrosion Potential
0-1,000	-	0-4.5	Very High
1,000-2,000	>500	4.5-5.5	High
2,000-5,000	<500	5.5-6.5	Moderate
>5,000	-	>6.5	Mild

Each of the columns in the above table should be used independently of the others for estimating corrosion potential. The results indicate that the potential for corrosion due to chloride ion concentration is *moderate* and the potential due to pH is *mild* in the samples tested. Based on the electrical resistivity results, the corrosion potential for buried steel is generally *moderate to high*. The design should consider corrosion potential.

10.4 Degradation of Concrete

The degradation of concrete is caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate compounds which cause cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete. Sulfate concentrations in soil can be used to evaluate the need for protection of concrete based on Table 10.3.

Table 10.3: Potential of Concrete Degradation Based on Sulfate Ion Concentration

Sulfate Ion Concentration, ppm	Aggressiveness
20,000	Very Severe
2,000 to 20,000	Severe
1,000-2,000	Moderate
<1,000	Mild

The results of sulfate tests on the selected soil samples from this study indicate the potential for deterioration of concrete, based on sulfate ion concentrations, is *mild*. Per ACI 318-19, the laboratory test results indicate a sulfate exposure class of 'S0'.

11. Pile Installation Considerations

11.1 General

Recommendations for driven pile and auger cast-in-place pile installation are provided in this section. Fugro has included discussions of pile driving specifications, pile driving equipment, installation methods, pile drivability analyses, pile driving records, contractor experience and equipment, pile installation and monitoring, and construction sequence.

11.2 ACIP Pile Installations Considerations

The proper installation of augered cast-in-place piles (ACIP) is dependent upon the Contractor's experience, construction procedures, and equipment. Installation factors have a greater effect on the performance of ACIP piles compared to any other type of deep foundation. Detailed pile installation specification specifically addressing the special concerns of ACIP piles should be prepared as part of the construction package. The pile specifications should be in general accordance with Deep Foundation Institute (2010). We should also be given an opportunity to review the proposed specifications prior to construction.

11.2.1 Contractor Experience and Equipment

The selected contractor should have relevant project experience with augering and pumping equipment, installation of similar sized augered piles in similar subsurface soil conditions, placement of reinforcing steel, as well as experience handling the special grout mixes and admixtures. The contractor's crane should generally be equipped with a torque converter to allow the auger to be withdrawn at a slow continuous rate without sudden jerking. Our experience indicates that not all contractors have the specialized equipment and experience required to successfully install ACIP piles and particularly for the installation of piles with diameters greater than 24-inches. We strongly recommend that the contractor's qualifications and experience records be reviewed as part of the selection process.

11.2.2 Grout Mix

The pile contractor should be able to demonstrate that a grout mix can be furnished to meet this project's requirements. The results of a test mix design are generally required to be submitted prior to approval of the grout mix for construction. During construction, the fluidity of the grout mix should be frequently tested using a ¾ inch diameter flow cone. Flow rates of 10 to 25 seconds are typically specified. Compressive strength of the grout mix should be checked by making at least six, 2-inch square cubes for each day of pile installation. Test cubes should be cured and tested in accordance with ASTM C109/C109M and may be restrained from expansion

as described in ASTM C942. The Deep Foundation Institute recommends testing two grout cubes at 7 days, two at 28 days, and keeping two on hold.

11.2.3 Pile Installation and Monitoring

The installation process makes it inherently difficult to verify the integrity of the installed ACIP piles, yet the integrity is essential to the load carrying capacity of ACIP piles. A comprehensive construction monitoring and record keeping program is recommended to help reduce the risks associated with improper construction techniques. Construction monitoring should be performed in accordance with Deep Foundation Institute (2010). We recommend that construction monitoring be performed by qualified personnel independent of the contractor.

There are several aspects of the installation procedure that can be monitored to aid in assessing whether the ACIP is being installed properly. These aspects are: 1) viscosity of the grout mixture to be pumped, 2) initial grout placement prior to raising the augers and resulting grout head observed upon completion of pile installation, 3) incremental grout factor (volume of grout pumped relative to the theoretical volume) computed over intervals of 5 foot or less as the auger is withdrawn, 4) uniformity of grout placement and computed grout factor along the length of the completed pile, 5) continuous grout placement and auger withdrawal without delays or grout pressure fluctuations, and 6) unobstructed and successful reinforcing steel placement in a timely manner following completion of grout placement. Records kept during construction monitoring should include these aspects of the pile installation.

The incremental volume of grout pumped as the auger is withdrawn is one of the more important installation controls. Common practice is to have the contractor calibrate the grout pump at the start of the work. The number of pump strokes are then computed to provide an initial grout head of 5-ft to 10-ft and at least 15% greater than the theoretical volume for each 5-ft increment of auger withdrawal. Typically, the actual volume of grout pumped is in the range of 15% to 60% greater than the theoretical volume. The required grout volume to obtain a uniform pile will vary depending on the soil type and consistency. An independent observer should continually monitor and document the number of pump strokes pumped over each 5-ft interval.

11.2.4 Reinforcing Steel Placement

The placement of reinforcing steel cages and tension steel can be improved significantly by using a second crane or cherry picker. If a second crane is possible, the benefits of using a second crane to place the reinforcing steel are: 1) the pile installation crane can be more productive drilling and grouting piles, 2) the second crane has a better reach to pick up reinforcing steel away from the congested pile location, 3) there is better control of cage alignment within the grouted pile, and 4) placement time is reduced by allowing the reinforcing steel to be lowered into the pile as soon as the auger is removed and the grouted pile top is cleaned.

11.2.5 Construction Sequence

The Contractor's installation plan should be reviewed prior to the start of construction. The installation plan should address the allowable spacing between piles to be grouted the same day. In general, previously cast piles should achieve their initial set (at least 6 hours and preferably 1 day) prior to augering adjacent piles. We strongly recommend installing ACIP piles spaced closer than about 6-ft (clear spacing) on alternate days to reduce the risk of interference between adjacent piles. A drop in the grout level of a completed pile when installing an adjacent pile is an indication of a possible "blow-out" condition and should be addressed. As such, providing sufficient time at this site for individual piles to achieve their initial set before installing adjacent piles is essential.

11.2.6 Sonic Integrity Logging

A non-destructive technique currently available to observe completed ACIP piles is the Sonic Integrity Logger. We believe it is essential that a qualified Geotechnical Engineer observe the installation of ACIP piles using sonic integrity logging to provide a higher degree of confidence that the piles are properly installed. Defects such as caving, necking or soil inclusions in the pile can be detected using sonic integrity logging. The use of the Sonic Integrity Logger at the start of construction to evaluate alternate installation procedures and to help establish an acceptable production pile installation procedure is particularly beneficial. We can provide literature and/or a demonstration of the sonic integrity logging equipment if requested.

11.3 Driven Pile Installation Considerations

Recommendations for driven pile installation are included in this subsection. We have included discussions of pile driving specifications, pile driving equipment, installation methods, and pile driving records.

11.3.1 Pile Driving Specifications

Detailed pile driving specifications should be prepared by the design engineer in conjunction with the geotechnical consultant. The specifications should cover the project requirements for furnishing and installing the piles including the scope of work, 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. 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 engineer 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. We would be pleased to provide sample specifications or to review the specifications for the project.

It is recommended that static pile load tests be performed to verify the recommended pile capacities in the field. The pile load tests should be performed in accordance with ASTM D1143. The pile load tests should be performed under the guidance of the geotechnical engineer of record so that the data may be interpreted, and the recommended pile capacities adjusted, if necessary, according to the load test results.

11.3.2 Pile Driving Equipment

Foundation 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.

The required energy rating for the hammer will vary depending on the ultimate capacity required, the depth of installation, and the pile type. Once these specifics are known, a wave equation analysis should be performed to aid in the hammer selection. 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.

11.3.3 Driven Pile Installation

Driven pile installation should be in general accordance with LaDOTD (2016), Section 804. Production piles for this site should be installed to a blow count criterion utilizing the wave equation and curves presented in Appendix E, as applicable. We recommend retapping selected production piles periodically to determine if the driving resistance and pile capacity increase or decrease with time.

In addition, we recommend surveying the production piles to detect possible vertical and/or horizontal movement (commonly referred to as heave), which can result from soil displacement

when driving adjacent piles. Piles which heave after driving adjacent piles should be redriven to at least their original penetration and final driving resistance. Pilot holes (Pre-drilling) will reduce heave, if unacceptable pile and ground movements are experienced. However, the diameter of the pilot hole should be limited to no greater than 2/3 of the pile tip diameter and should extend no deeper than 50 percent of the planned pile depth. Following this guideline for pre-drilling, the presented capacities may be used for design. We have based our computed capacities on the assumption that piles will be driven to the desired penetration without any additional supplemental drilling techniques. If supplemental techniques are selected in installation of the piles, in addition to the recommended pre-drilling procedure, we should be retained to evaluate their influence on our computed capacities. Use of jetting to aid pile installation should not be used. Relatively hard driving conditions in medium dense to dense sands should be anticipated below a depth of approximately 30 feet at the B-1 and B-3 boring locations.

11.3.4 Pile Driving Records

An accurate and detailed driving log should be kept by an independent inspector 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 retapping, and notes on installation delays, problems, or unusual occurrences.

12. Construction Considerations

The following sections provide our construction considerations for surface and groundwater control, shallow foundation excavations, fill material and placement, and quality control and field testing.

12.1 Surface Water and Groundwater Control

Surface water and groundwater control should be established early at the site and maintained throughout construction and operation. Groundwater was encountered at depths between approximately 3 and 5 feet in our soil borings at the time of our field investigation. The groundwater table at the time of construction could be lower or higher than that encountered at the time of our field investigation. We recommend that the groundwater level be determined at the time of construction to aid in construction planning.

We recommend that foundation excavations be maintained at least 1 foot above the groundwater level at the time of construction. The Contractor should be prepared to provide groundwater control particularly if excavations extend below these depths. Shallow excavations less than about 8 feet in depth are expected to encounter soft to stiff cohesive soils, in which sumps and pumps should be sufficient to dewater foundation or utility excavations. Deeper excavations may encounter more permeable silts and sands, in which other dewatering means and methods (such as well points) may be required to address larger quantities of water. Silty soils were encountered at a depth of 3 feet in Boring B-5.

12.2 Shallow Foundation Excavations

Excavation safety systems should be in accordance with federal Occupational Safety and Health Administration (OSHA) standards (OSHA, 2006). Details for open-cut slopes and excavation shoring based on soil type and groundwater conditions are provided in the latest amended OSHA federal regulations.

Based on our interpretation of the regulations and the subsurface conditions indicated in our borings, the natural surficial cohesive soils at the site should be classified as “Type B” soil. The OSHA regulations do not generally require shallow excavations to depths of 4 ft or less to be sloped back or braced. If sloughing and/or raveling are experienced, the excavation slopes should be braced or sloped back to 1 horizontal to 1 vertical for “Type B” soils. Flatter slopes or bracing should be used in either case if sloughing or raveling continues.

The excavations should be "dry" and free of loose soil, ponded water, and debris prior to placing the reinforcing steel and the concrete.

12.3 Fill Material and Placement Recommendations

12.3.1 Structural Sand Fill

Structural Sand Fill may be used as grade raise fill and backfill for utilities and should consist of a clean sand fill with between 5 and 15 percent material passing the No. 200 sieve. Soils meeting structural sand requirements include poorly and well graded sand with silt/clay (SP-SM, SP-SC, SW-SM, SW-SC), and may also include silty sand (SM) and clayey sand (SC). Locally available pumped river sand typically meets material requirements. Structural sand used for grade raise fill should be placed in 8-inch thick loose lifts, at a moisture content within two points of the optimum moisture content, as determined by ASTM D698. Structural sand used for utility backfill should be placed in 6- to 8-inch thick loose lifts, at a moisture content within two points of the optimum moisture content. Structural sand fill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698. Grade raise fill for the proposed equipment shelter should consist of structural sand fill compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698, with an average value of 98 percent. Fugro recommends using hand-operated compaction equipment and 4-inch-thick loose lifts in confined areas and immediately adjacent to structures. Care should be taken not to over compact next to below grade walls, as applicable. Excessive compaction could induce lateral pressures that are greater than the design pressures.

Structural sand fill has medium to high erosion potential and will require protective cover, such as erosion control blanket or structural clay cap, on permanent slopes to mitigate the development of rills and gullies.

12.3.2 Structural Clay Fill

Structural clay fill may be used as previously recommended in this report. Structural clay fill should have a liquid limit of less than 40 and a plasticity index between 10 and 20. Soils meeting structural clay fill requirements include sandy lean clay or lean clay (CL) and may also include clayey sand (SC). The effective diameter of clay clods should not be larger than 3 inches. Structural clay fill should be free of deleterious matter. The overburden clay soils at the site are not suitable for use as structural clay fill unless stabilized as described in Section 12.3.3.

Structural clay fill should be placed in loose lifts not more than 6- to 8-inches thick and uniformly compacted to at least 95 percent of the maximum dry density as determined by ASTM D698 (Standard Proctor). In confined areas, structural clay fill should be placed in 4-inch-thick loose lifts and compacted using hand-operated compaction equipment. Compaction should be performed 1 to 4 percent above the optimum moisture content. Prior to the start of fill placement, representative samples of each proposed fill material should be collected and classified. The compaction properties of the proposed fill soil should be determined during this testing. Structural clay fill should be compacted to provide a “kneading” compaction equivalent

to the sheepsfoot or pad foot roller. If wet weather or extended dry periods deteriorate the surface whereby a good bond cannot be formed between successive lifts, the earthwork 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.

12.3.3 Lime-Stabilized Clay Fill and Subgrade

Lime-stabilization may be utilized to strengthen and stiffen potential structural clay fill. Lime stabilization may also be utilized for "in place" stabilization of at least the final 8-inch lift of structural clay fill below temporary aggregate surfacing and pavement sections. Laboratory tests should be conducted on pertinent samples from an onsite or offsite borrow source at the time of construction to determine the optimum lime content. The optimum lime content is the amount of lime necessary to achieve a pH of 12.4 (which represents lime fixation), while trying to achieve a plasticity index (PI) of less than 20. For estimation purposes, about 4 to 6 percent lime, by dry weight, may be required to stabilize low to moderate plasticity cohesive fill soils and about 7 to 9 percent lime, by dry weight, may be required to stabilized moderate to high plasticity cohesive soils. Organics, chemical fertilizers, and some clay minerals can affect the amount of lime necessary for lime fixation. We recommend that a lime series be performed using the specific soil samples and proposed lime additive.

Lime stabilization should be performed in accordance with the Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime. Key items for lime (cement)-stabilizing clay soils include placing the proper percentage of lime (cement), thoroughly mixing the lime (cement) into the clay soils, bringing the stabilized soil to the proper moisture content, allowing the stabilized soil to cure or "mellow" for at least 48 hours, adjusting the moisture content from 1 percent dry to 3 percent wet of optimum moisture content, pulverizing the soils again until the lime (cement) is thoroughly blended, then placing the stabilized soil in accordance with the recommendations for structural fill discussed as follows.

Lime-stabilized clay should be placed in 8-inch-thick loose lifts and compacted to at least 95 percent of the maximum dry density as determined by ASTM D698 (Standard Proctor), as should the stabilized pavement subgrades. In confined areas, lime (cement)-stabilized clay fill should be placed in 4-inch thick loose lifts and compacted using hand-operated compaction equipment. The moisture content at the time of placement should be -1 (wet) to +3 (wet) percentage points of the optimum moisture content. The moisture-density relationship should be established based on a material sample obtained from onsite after stabilization with lime. Care should be taken to ensure that proper compaction has been achieved for the full depth and width of each lift.

12.3.4 General Fill

General clay fill may be used to raise site grade outside areas planned for current or future development. Clay fill should be free of deleterious material and should have an effective clod diameter less than 5 inches. Clay fill should have a liquid limit of less than 65 and plasticity index between 8 and 40. Typical soils meeting general fill requirements include lean clay and sandy lean clay (CL), fat clay and sandy fat clay (CH), and clayey sand (SC). Please note that crushed stone, lime (cement)-stabilized clay fill, or structural clay fill can also be used in place of clay fill. Clay fill should be placed in 6- to 8-inch thick loose lifts. We recommend using a sheepsfoot roller or equivalent rolling equipment for compaction. Clay fill should be uniformly compacted to at least 92 percent maximum dry density at a moisture content of 1 percent dry to 4 percent wet of optimum as determined by ASTM D698 (Standard Proctor).

12.3.5 Flowable Fill

Flowable fill or lean concrete may be used to replace over-excavated deleterious soils and to form seal slabs beneath the proposed foundations. In general, sandy soils containing less than about 30 percent fines are suitable for use as flowable fill. Flowable fill to be used as backfill material should meet the following criteria:

- sandy soils used in flowable fill should contain no more than 30% fines,
- soil and cement for flowable fill should be thoroughly mixed,
- proportioning of the sandy soil and cement should be sufficient to produce an unconfined compressive strength of at least 100 psi at 7 days while remaining workable for placement,
- flowable fill should not be placed when ambient air temperature is below 40 degrees Fahrenheit,
- placement of the flowable fill should completely fill the space to be backfilled,
- flowable fill should be properly compacted using standard ASTM procedures for soil-cement materials; if a fluid consistency is used for the flowable fill, an “elephant trunk” vibrator may be used to consolidate and remove air voids from the mixture, and
- backfill or foundation concrete over the flowable fill should not be placed until after initial set (4 to 24 hours) has been achieved for the flowable fill.

12.4 Quality Control and Field Testing

Fill material index properties must be obtained upon each change in soil properties, soil color or texture, compactor, or lift thickness, and at minimum frequencies recommended by the Geotechnical Engineer of Record based on the range and variation of fill and source soil properties. For preliminary planning purposes, minimum frequencies of at least every 1,000 loose cubic yards can be estimated. Fill soil sampling should be planned and coordinated to fit the construction production goals, and generally at least two days in advance of the compaction of

corresponding fill lifts, depending on confirmation of other process controls and source soil ranges and variations.

The Geotechnical Engineer of Record should perform field moisture-density tests at appropriate frequencies on compacted lifts to ensure that the compaction control specifications are achieved. Typically for mass fills, such tests would be performed every 2,500 square feet of compacted lift or fraction thereof. For trench backfill and small areas of filling, field moisture content/density testing should be performed for every 200 cubic yards of backfill placed for each lift or for every 200 linear feet of trench, whichever represents the greater frequency of testing. The Geotechnical Engineer of Record or his representative shall determine the acceptability of each compacted lift based on this regime of field moisture content/density testing. Field density gauge probes must be deployed to measure the average properties of the compacted lift.

The Geotechnical Engineer of Record will develop a QC/QA work plan designed to fit the contractors fill construction production goals with effective process controls.

13. Geotechnical Design Review and Construction Monitoring

Following the final design, we should be given the opportunity to review the drawings and specifications to verify that our design recommendations were adhered to and interpreted as we have intended. We strongly recommend that the Geotechnical Engineer-of-Record, or their qualified representative, be present on-site during site preparation, earthwork, and construction to observe and monitor construction activities. Construction monitoring performed by qualified personnel independent of the Contractor is recommended because the performance of foundations and other structures 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, the geotechnical aspects of construction contain inherent uncertainties that can result in unanticipated soil and/or groundwater conditions being 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:

- Site Preparation and Site Excavation: The site preparation activities, including site excavations, should be monitored and the suitability of the exposed subgrade should be observed.

- Placement of Fill: Activities regarding the placement of fill should be monitored to verify that the material being used as fill complies with the recommendations in this report and with the project specifications. This includes performing compaction tests, classification tests, and field density tests.

- Foundation Installation: The installation of foundations should be monitored to verify the suitability of the exposed foundation soils, the stability of the foundation excavation, and the placement of foundation concrete.

- Pavement Construction: The construction of pavements should be monitored to verify the suitability of the exposed subgrade soils and base materials, and the placement of asphalt concrete and Portland cement concrete.

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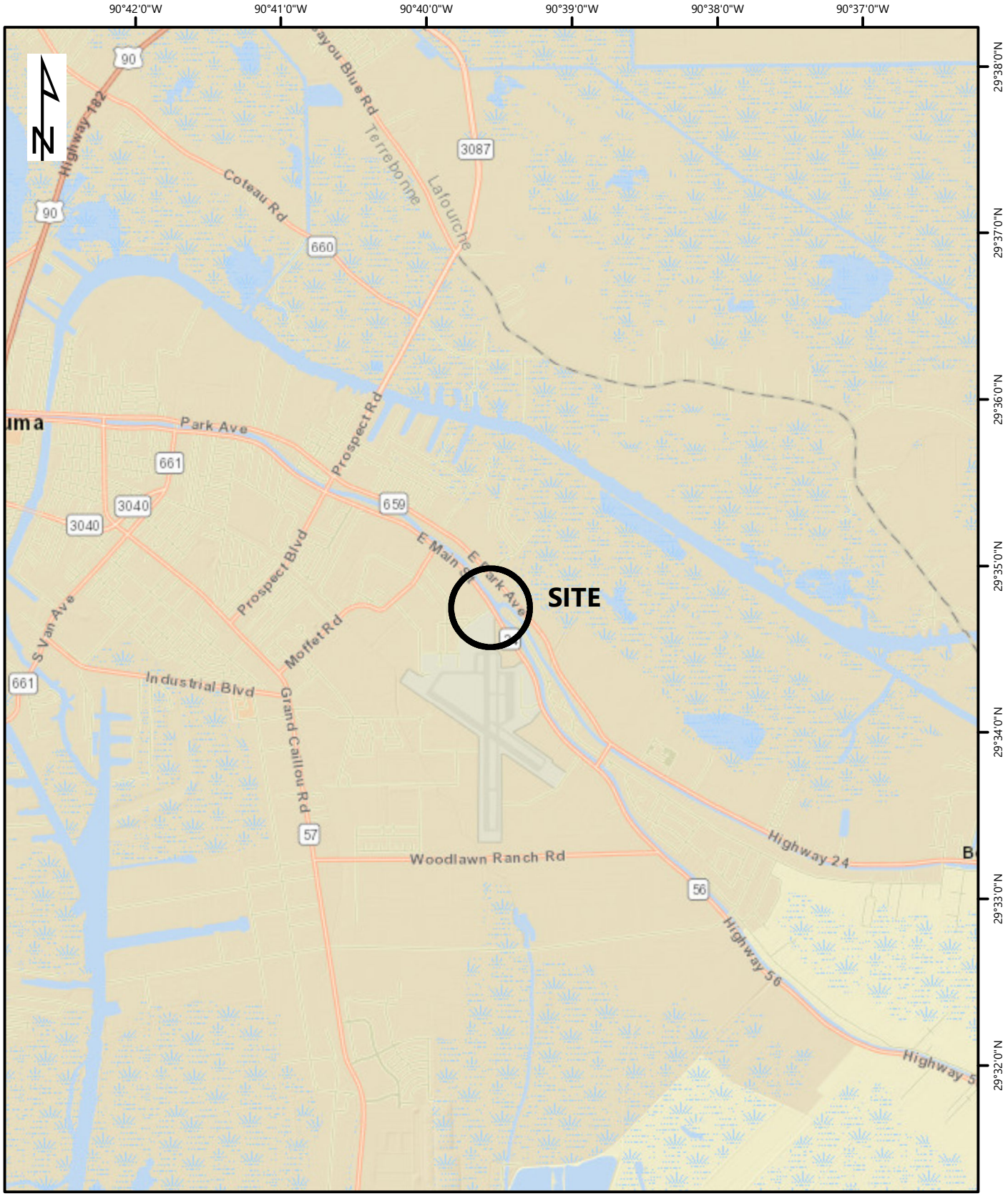
Plates

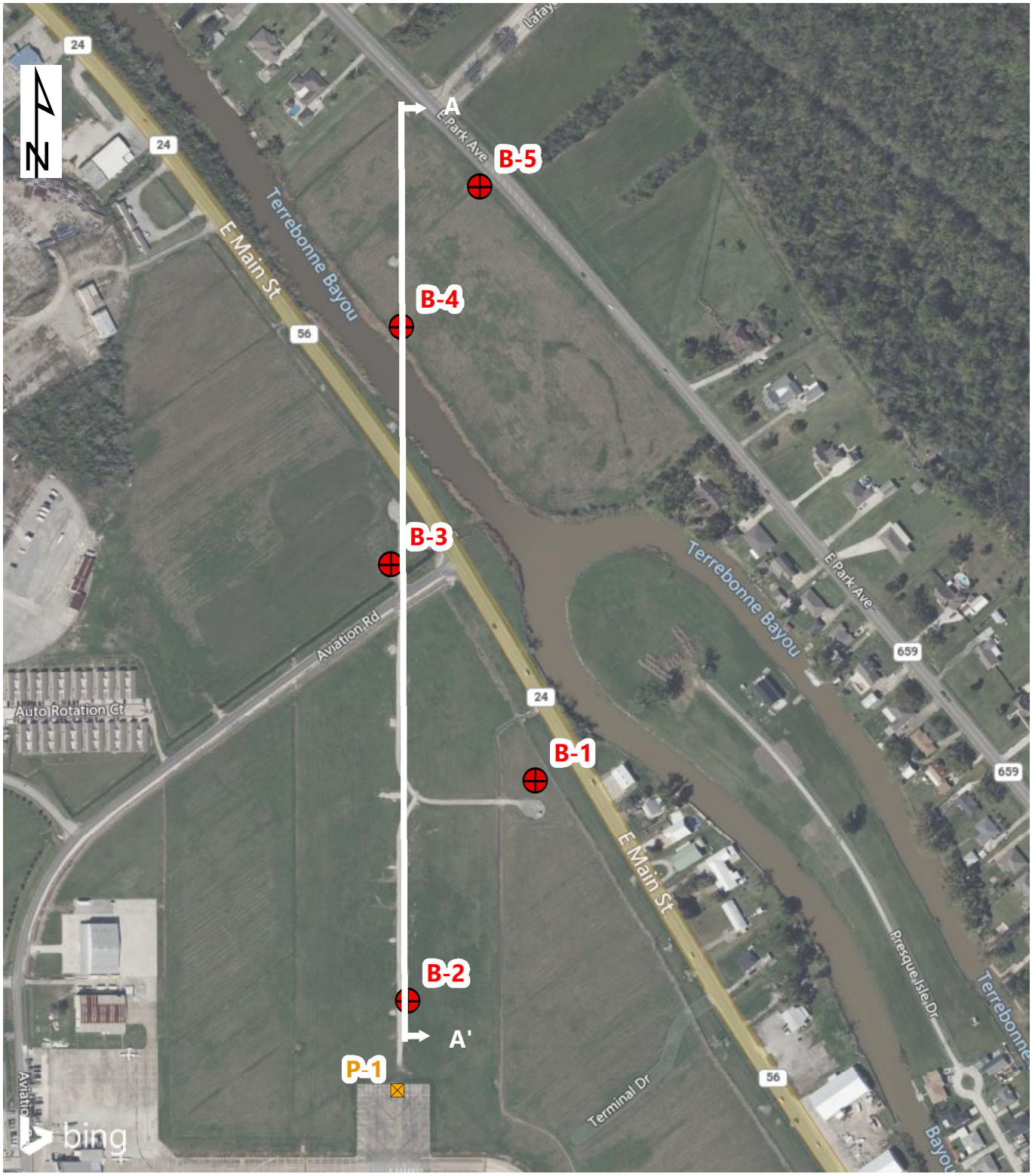


0 0.5 1 2 mi

0 0.75 1.5 3 km

Service Layer Credits: Sources: Esri.

PLATE 1: VICINITY MAP



NOTE:
Borings were located in the field and the locations are approximate.

0 150 300 600 ft
0 30 60 120 m

Service Layer Credits: © 2024 Microsoft Corporation.

Legend

SOIL BORING LOCATION

BLAST PAD CORING LOCATION

SUBSURFACE SOIL PROFILE LOCATION

PLATE 2: EXPLORATION PLAN

LOCATION: See Plate 2

COORDINATES: N 29° 34' 40.17"
W 90° 39' 32.42"

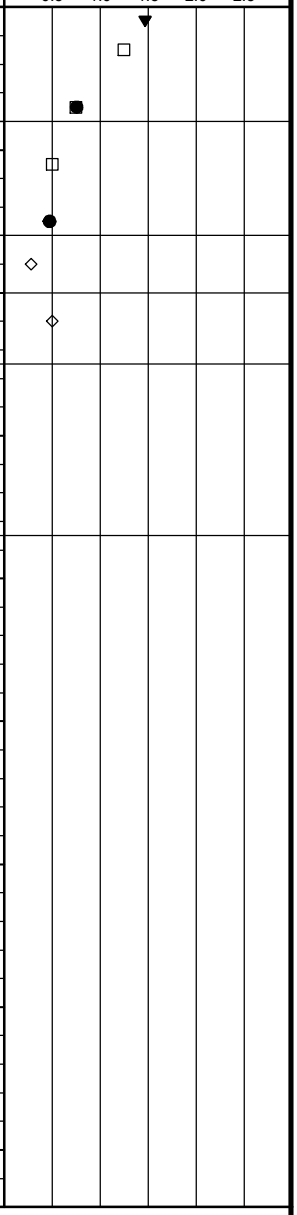
SURFACE EL.: 15.0' Approximate

DEPTH, FT	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		FAT CLAY (CH), firm to stiff, brown, with organics		91		28			
		- with sand pockets at 1'				44	65	21	44
		- with silt lenses and ferrous stains at 3'		85		36			
		LEAN CLAY (CL), soft to firm, reddish brown, with organics and sand pockets	4.0			33	34	20	14
		- gray and brown, with ferrous stains at 7'		88		34	35	21	14
		SILT (ML), gray, with clay and sand pockets	8.0		90	34	30	23	7
		LEAN CLAY (CL), firm, gray, with organics and sand pockets	10.0			33	34	22	12
	N=WOH	**SILT WITH SAND (ML)**, dark brown, with clay traces	12.5						
	N=6				76	32			
	N=10	**SILTY SAND (SM)**, loose to dense, dark brown, with clay traces	18.5						
	N=14				37	27			
	N=4								
	N=31	- dark gray at 33.5'			22	23			
	N=21				13	18			

SHEAR STRENGTH

□ Penetrometer ▼ Unconfined
◇ Torvane ● Triaxial
△ Hand Vane ▲ Miniature Vane

KIPS PER SQ FT



NOTES:

1. ▽: Water First Noticed. ▼: Depth To Water after 15 min
2. Terms and symbols defined on Plates 8a and 8b.
3. Approximate elevations obtained from Google Earth.

COMPLETION DATE: March 10, 2024
TOTAL DEPTH: 60.0'
CAVED DEPTH: Not Applicable
DRY AUGER: 0' to 10'
WET ROTARY: 10' to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Suter
DRILL RIG: Ardco
HAMMER TYPE: Automatic

FUGRO

Runway 18 Approach Lighting Project

LOG OF BORING NO. B-1

Houma-Terrebonne Airport

Houma, Louisiana

Project No. 04.00251305

PLATE 3a

FCBR_LOG (FINAL) 2016 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

LOCATION: See Plate 2

COORDINATES: N 29° 34' 40.17"
W 90° 39' 32.42"

SURFACE EL.: 15.0' Approximate

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
						43.5						
45				N=7	**FAT CLAY (CH)**, firm to stiff, gray, with wood fragments and shell fragments - with sand traces at 43.5'				65			
50							56		74			
55					- with sand seams at 53.3'		64		59			
60				N=6	- with silt pockets at 58.5'	60.0		98	41			
65												
70												
75												
80												

CLASSIFICATION

SHEAR STRENGTH

□ Penetrometer ◇ Torvane △ Hand 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 15 min
2. Terms and symbols defined on Plates 8a and 8b.
3. Approximate elevations obtained from Google Earth.

COMPLETION DATE: March 10, 2024
TOTAL DEPTH: 60.0'
CAVED DEPTH: Not Applicable
DRY AUGER: 0' to 10'
WET ROTARY: 10' to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Suter
DRILL RIG: Ardco
HAMMER TYPE: Automatic

FUGRO

Runway 18 Approach Lighting Project

LOG OF BORING NO. B-1

Houma-Terrebonne Airport

Houma, Louisiana

Project No. 04.00251305

PLATE 3b

FCBR_LOG (FINAL) 2016 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

LOCATION: See Plate 2

COORDINATES: N 29° 34' 34.17"
W 90° 39' 36.45"

SURFACE EL.: 14.0' Approximate

DEPTH, FT	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		FAT CLAY (CH), very soft to stiff, brown				29	60	24	36
		- with organics from 0' to 4'		94		28			
		- with roots from 0' to 2'							
		- with silt lenses from 1' to 8'							
		- gray, with ferrous stains from 1' to 4'		94		29			
5				85		35	56	20	36
		- gray and reddish brown from 7' to 12'		79		45			
		- with sand pockets at 8'			87	32			
10	N=1	- with sand traces at 10.5'			98	40			
	N=WOH	- gray at 12'							
		ORGANIC CLAY (OH), firm, dark gray, with wood fragments	14.0	42		111			
15									
20	N=2	**LEAN CLAY (CL)**, soft, gray and reddish brown, with sand pockets	18.5			32	33	21	12
						43	29	18	11
25									
30	N=7	**SANDY FAT CLAY (CH)**, firm, gray	28.5		67	44			
35	N=3	**FAT CLAY (CH)**, soft to firm, gray, with silt pockets	33.5						
40			40.0	67		56			

SHEAR STRENGTH

□ Penetrometer ▼ Unconfined
◇ Torvane ● Triaxial
△ Hand 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 min
2. Terms and symbols defined on Plates 8a and 8b.
3. Approximate elevations obtained from Google Earth.

COMPLETION DATE: March 10, 2024
TOTAL DEPTH: 40.0'
CAVED DEPTH: Not Applicable
DRY AUGER: 0' to 10'
WET ROTARY: 10' to 40'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Suter
DRILL RIG: Ardco
HAMMER TYPE: Automatic

FUGRO

Runway 18 Approach Lighting Project

LOG OF BORING NO. B-2

Houma-Terrebonne Airport

Houma, Louisiana

Project No. 04.00251305

PLATE 4

FCBR_LOG (FINAL) 2016 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

LOCATION: See Plate 2

COORDINATES: N 29° 34' 46.1"
W 90° 39' 36.9"

SURFACE EL.: 12.0' Approximate

DEPTH, FT	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		LEAN CLAY (CL), soft to stiff, reddish brown - with organics, roots, shell fragments, and sand pockets from 0' to 2'				32	40	17	23
		- gray, with silt pockets, ferrous stains, and ferrous nodules at 3'		89		34			
5						31	38	19	19
	N=2	- dark gray, with sand pockets below 6.5'			97	39			
	N=8								
10	N=4								
	N=5	- with shell fragments at 12.5'			87	35	31	20	11
15	N=12								
	N=14	**SILTY SAND (SM)**, loose to medium-dense, dark gray, with clay traces	18.5		35	30			
20									
25	N=7								
	N=26	**POORLY GRADED SAND WITH SILT (SP-SM)**, medium-dense, dark brown, with shell fragments and clay traces	28.5		10	19			
30									
	N=24	**CLAYEY SAND (SC)**, medium-dense, dark brown, with shell fragments	33.5		47	26			
35									
	N=5	**FAT CLAY (CH)**, firm, gray, with 6-inch clayey sand layer	38.5						
40			40.0						

CLASSIFICATION

SHEAR STRENGTH

□ Penetrometer ◇ Torvane △ Hand 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 15 min
2. Terms and symbols defined on Plates 8a and 8b.
3. Approximate elevations obtained from Google Earth.

COMPLETION DATE: March 10, 2024
TOTAL DEPTH: 40.0'
CAVED DEPTH: Not Applicable
DRY AUGER: 0' to 8'
WET ROTARY: 8' to 40'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Suter
DRILL RIG: Ardco
HAMMER TYPE: Automatic

FUGRO

Runway 18 Approach Lighting Project

LOG OF BORING NO. B-3

Houma-Terrebonne Airport

Houma, Louisiana

Project No. 04.00251305

PLATE 5

FCBR_LOG (FINAL) 2016 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

LOCATION: See Plate 2

COORDINATES: N 29° 34' 52.6"
W 90° 39' 36.52"

SURFACE EL.: 11.0' Approximate

DEPTH, FT	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		FAT CLAY (CH), firm to stiff, brown, with organics, ferrous stains, roots, and sand pockets				35	68	24	44
				87		33			
		LEAN CLAY (CL), soft to very stiff, reddish brown and dark gray, with organics and sand pockets	2.0			34	36	22	14
		- dark gray below 4'			96	35			
5									
	N=6				96	33	32	21	11
10	N=20								
	N=7								
15	N=4	- with wood fragments at 14.5'			94	30			
20	N=11				92	36			
25	N=33	**WOOD**, brown, with gray silt	23.5			23			
30	N=5	**SANDY FAT CLAY (CH)**, firm, dark gray, with wood fragments	28.5		62	39			
35	N=10	**FAT CLAY (CH)**, firm to stiff, dark gray - with sand traces at 33.5'	33.5		99	55			
		- with organics and ferrous stains below 38'							
40			40.0	65		59			

SHEAR STRENGTH

□ Penetrometer ▼ Unconfined
◇ Torvane ● Triaxial
△ Hand 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 min
2. Terms and symbols defined on Plates 8a and 8b.
3. Approximate elevations obtained from Google Earth.

COMPLETION DATE: March 8, 2024
TOTAL DEPTH: 40.0'
CAVED DEPTH: Not Applicable
DRY AUGER: 0' to 10'
WET ROTARY: 10' to 40'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Suter
DRILL RIG: Ardco
HAMMER TYPE: Automatic

FUGRO

Runway 18 Approach Lighting Project

LOG OF BORING NO. B-4

Houma-Terrebonne Airport

Houma, Louisiana

Project No. 04.00251305

PLATE 6

FCBR_LOG (FINAL) 2016 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

LOCATION: See Plate 2

COORDINATES: N 29° 34' 56.42"
W 90° 39' 34.05"

SURFACE EL.: 15.0' Approximate

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, 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: **LEAN CLAY (CL)**, brown, with gravel, sand pockets, and organics				26	49	18	31	Penetrometer
					LEAN CLAY (CH), soft, gray, with silt pockets, ferrous stains, and trace sand pockets	2.0	87		33				Unconfined
					SILT (ML), gray, with clay lenses and ferrous stains	3.0		95	36				Torvane
5				N=3				95	36				
10				N=8				96	33				
						12.0							
15													
20													
25													
30													
35													
40													

Shear strength legend: □ Penetrometer; ◇ Torvane; △ Hand Vane; ▼ Unconfined; ● Triaxial; ▲ Miniature Vane

NOTES:
1. Terms and symbols defined on Plates 8a and 8b.
2. Approximate elevations obtained from Google Earth.

COMPLETION DATE: March 8, 2024
TOTAL DEPTH: 12.0'
CAVED DEPTH: Not Applicable
DRY AUGER: 0' to 12'
WET ROTARY: Not Applicable
BACKFILL: Soil Cuttings
LOGGER: J. Suter
DRILL RIG: Ardco
HAMMER TYPE: Automatic

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Runway 18 Approach Lighting Project

LOG OF BORING NO. B-5

Houma-Terrebonne Airport

Houma, Louisiana

Project No. 04.00251305

PLATE 7

FCBR_LOG (FINAL) 2016 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

SOIL TYPES

Fat CLAY
Sandy Fat CLAY
Lean CLAY
Fill
SILT
Organic CLAY
Clayey SAND
Silty SAND
Poorly Graded SAND with SILT
Wood

SAMPLER TYPES

Thin-walled Tube
Partial Recovery w/ Tube
Auger
Split-barrel
No Recovery
Pitcher
Piston
Geoprobe
Liner

SOIL GRAIN SIZE

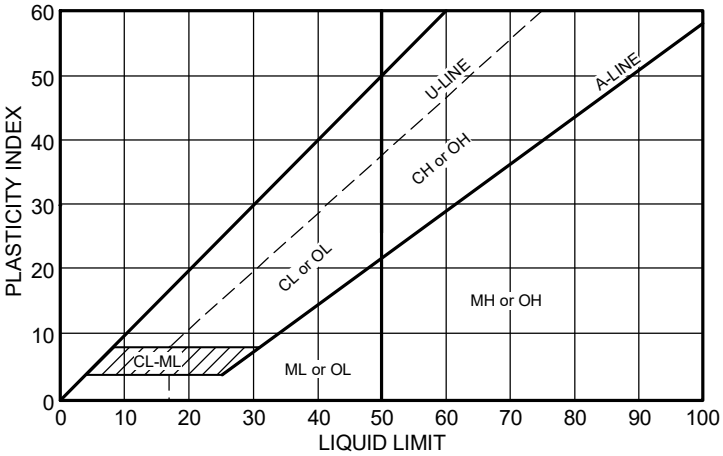
U.S. Standard Sieve

Boulders	Cobbles	Gravel		Sand			Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine		

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

152 75.0 19.0 4.75 2.00 0.425 0.075 0.005 (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.

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TERMS AND SYMBOLS USED ON BORING LOGS

SOIL CLASSIFICATION (1 of 2)

Project No. 04.00251305

PLATE 8a

TERMS 1 2021 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 04/05/24

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 = Hand Vane

HAND PENETROMETER CORRECTION

Pocket penetrometer (PP) strength measurements reported on the boring logs are estimated undrained shear strengths from unadjusted PP readings so that end-users can apply adjustments based on their experience and risk. Fugro generally adjusts the PP strength measurements to account for the small PP test size compared to the larger laboratory test specimen. For Gulf Coast cohesive soils (with PP readings greater than about 1 tsf), our experience indicates that multiplying the raw PP readings (in tsf) by 0.6 provides estimated undrained shear strengths (in ksf) which correlate better with laboratory tests.

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.

Jacobs

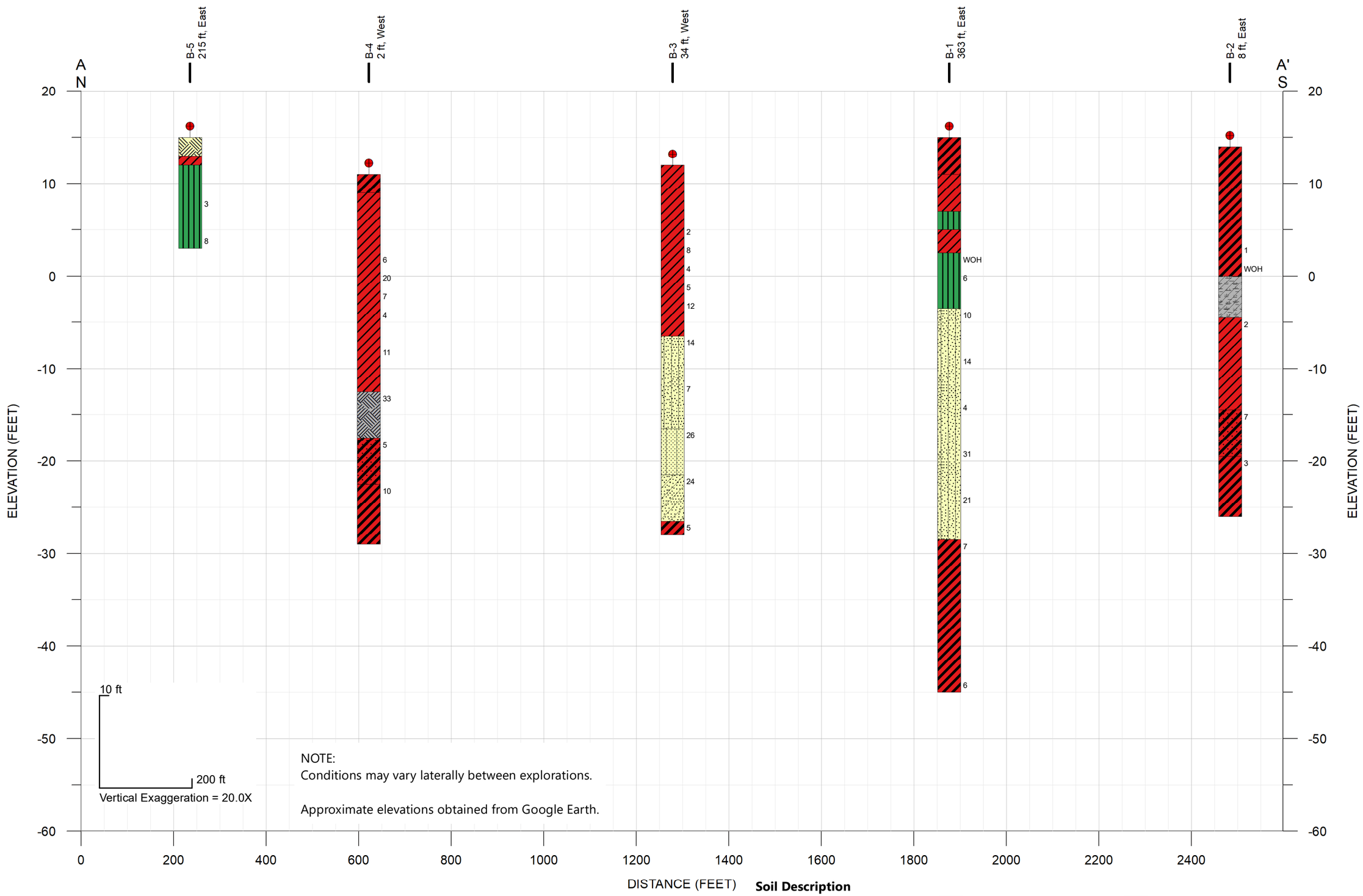

PLATE 9: GENERALIZED SUBSURFACE PROFILE A-A'

Legend

SOIL BORING

10 FIELD SPT N-VALUE

Soil Description

- Lean CLAY (CL)
- Fat CLAY (CH)
- Sandy Fat CLAY (CH)
- Silt (ML)
- Poorly-Graded SAND with Silt (SP-SM)
- Clayey SAND (SC)
- Silty SAND (SM)
- High-Plasticity Organic (OH)
- WOOD
- Fill

Fugro Document No. 04.00251305 | Runway 18 Approach Lighting Project - Houma-Terrebonne Airport

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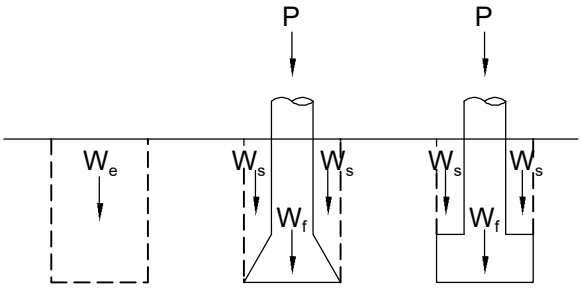
FOUNDATION DESIGN CRITERIA

A properly-sized foundation must satisfy the two following criteria with respect to the supporting soil.

1. For soil strength. The bearing pressure created on the base of the foundation by the maximum design load must be less than that which would cause shear failure in the soil. A factor of safety of 2 or more with respect to the soil shear strength is generally used.
2. For soil compressibility. The bearing pressure created on the base of the foundation by the sustained load must not produce sufficient consolidation in the underlying soil to result in foundation settlement that is detrimental to the safety or utility of the structure.



SYMBOLS

TERMS AND SYMBOLS

P = Column load (subscript can be used to denote character of load: P_s = sustained load, P_n = normal operating load, P_m = maximum design load).

W_e = Weight of soil located above base of foundation excavation and lowest adjacent grade.*

W_s = Weight of soil located above foundation.*

W_f = Weight of foundation.*

A = Area of base of foundation.

p = Average bearing pressure acting on soil (subsript can be used to correspond to column load: P_s, P_n, P_m).

* Position of groundwater level must be considered in determining weights. Effective, or buoyant, unit weights should be used below the highest expected groundwater level.

BEARING PRESSURES

Gross Bearing Pressure, p, for any column load is the total pressure acting on the base of the foundation.

$$p = 1/A\ (P + W_s + W_f)$$

Net Bearing Pressure, p', for any column load is the difference between the gross bearing pressure acting on the base of the foundation and the soil pressure existing at that elevation from the lowest overlying or adjacent soils.

$$p' = 1/A\ (P + W_s + W_f - W_e)$$

For analysis with regard to the first design criterion, soil strength, the column load in the above equations should usually be the maximum design load, P_m. Occasionally, the normal operating load, P_n, may also be used. If footing is loaded eccentrically, the increase in edge bearing pressure due to the eccentricity should be computed in the usual manner.

For analysis with regard to the second design criterion, soil compressibility, the column load in the above equations should be the sustained load, P_s. This load is the dead load plus the sustain live load.

For further references, see pp. 506 - 512, "Soil Mechanics in Engineering Practice" by Karl Terzaghi and Ralph B. Peck (2nd edition); and pp. 564 - 565, "Fundamentals of Soil Mechanics" by Donald W. Taylor.

Appendix A

Summary of Test Results

Sample No.	Depth (ft)	Identification Tests						Field Shear Strength Estimate		Miniature Vane Tests		Compression Tests								
		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (PI)	Moisture Content (%)	Dry Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer (ksf)	Torvane (ksf)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	Type Test	Moisture Content (%)	Confining Pressure (psi)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	ε_{50} Strain (%)	Dry Unit Weight (pcf)	Failure Strain (%)	Type of Failure
LT1A	0.0-1.0				28	91						U	28		1.47		1.7	91	10.3	C
B1B	1.0-2.0	65	21	44	44			1.25												
B2A	2.0-3.0																			
LT2B	3.0-4.0				36	85		0.75				UU	36	2	0.76		1.8	85	15.0	C
B3A	4.0-5.0	34	20	14	33															
LT3B								0.5												
B4A	6.0-7.0	35	21	14	34															
LT4B	7.0-8.0				34	88			0.46			UU	34	6	0.48		3.2	88	15.0	C
B5A	8.0-10.0	30	23	7	34		89.8													
									0.28											
B6A	10.0-12.0	34	22	12	33															
									0.50											
B8A	14.5-16.0				32		76.1													
B10A	23.5-25.0				27		37.2													
B12A	33.5-35.0				23		21.6													
B13A	38.5-40.0				18		13.2													
B14A	43.5-45.0				65															
LT15A	48.0-49.0				74	56						UU	74	38	0.69		0.8	56	8.8	C
B15B								0.5												
LT16B	54.0-55.0				59	64		0.75				UU	59	43	1.08		0.9	64	3.1	C
B17A	58.5-60.0				41		97.7													

Notes:

NP = Non-Plastic Material

TYPE OF TEST
U - Unconfined Compression
UU - Unconsolidated - Undrained Triaxial
CU - Consolidated - Undrained Triaxial

TYPE OF FAILURE
A - Bulge
B - Single Shear Plane
C - Multiple Shear Plane
D - Vertical Fracture
E - Slickensides

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Runway 18 Approach Lighting Project

SUMMARY OF TEST RESULTS - BORING B-1

Houma-Terrebonne Airport

LELAP Lab ID #10001

Houma, Louisiana

Project No. **04.00251305**

PLATE A-1

Sample No.	Depth (ft)	Identification Tests						Field Shear Strength Estimate		Miniature Vane Tests		Compression Tests								
		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (PI)	Moisture Content (%)	Dry Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer (ksf)	Torvane (ksf)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	Type Test	Moisture Content (%)	Confining Pressure (psi)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	ε_{50} Strain (%)	Dry Unit Weight (pcf)	Failure Strain (%)	Type of Failure
B1A	0.0-1.0	60	24	36	29															
LT1B	1.0-2.0				28	94		1.25				U	28		0.85		1.6	94	9.3	C
B2A	2.0-3.0																			
LT2B	3.0-4.0				29	94		1.5				UU	29	2	1.40		1.6	94	14.1	C
LT3A	4.0-5.0	56	20	36	35	85														
B3B								0.5												
LT4B	7.0-8.0				45	79		0.5				UU	45	6	0.51		1.7	79	14.9	A,C
B5A	8.0-10.0				32		86.6													
									0.24											
B6A	10.5-12.0				40		97.5													
LT8B	15.0-16.0				111	42			0.56			UU	111	12	0.51		0.5	42	5.3	A,C
B9A	18.5-20.0	33	21	12	32															
B10A	23.0-24.0	29	18	11	43															
LT10B									0.28											
B11A	28.5-30.0				44		66.8													
LT13B	39.0-40.0				56	67			0.48			UU	56	31	0.55		0.7	67	4.4	C

Notes:

NP = Non-Plastic Material

TYPE OF TEST
U - Unconfined Compression
UU - Unconsolidated - Undrained Triaxial
CU - Consolidated - Undrained Triaxial

TYPE OF FAILURE
A - Bulge
B - Single Shear Plane
C - Multiple Shear Plane
D - Vertical Fracture
E - Slickensides

FUGRO

Runway 18 Approach Lighting Project

SUMMARY OF TEST RESULTS - BORING B-2

Houma-Terrebonne Airport

LELAP Lab ID #10001

Houma, Louisiana

Project No. **04.00251305**

PLATE A-2

FCBR SUMMARY OF TESTS 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

Sample No.	Depth (ft)	Identification Tests						Field Shear Strength Estimate		Miniature Vane Tests		Compression Tests								
		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (PI)	Moisture Content (%)	Dry Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer (ksf)	Torvane (ksf)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	Type Test	Moisture Content (%)	Confining Pressure (psi)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	ε_{50} Strain (%)	Dry Unit Weight (pcf)	Failure Strain (%)	Type of Failure
B1A	0.0-2.0	40	17	23	32															
								1.75												
B2A	2.0-3.0																			
LT2B	3.0-4.0				34	89			0.40			U	34		0.43		6.4	89	15.0	C
B3A	4.0-5.0	38	19	19	31															
LT3B									0.42											
B4A	6.5-8.0				39		96.8													
B7A	12.5-14.0	31	20	11	35		87.4													
B9A	18.5-20.0				30		34.5													
B11A	28.5-30.0				19		9.6													
B12A	33.5-35.0				26		46.6													

Notes:

NP = Non-Plastic Material

TYPE OF TEST
U - Unconfined Compression
UU - Unconsolidated - Undrained Triaxial
CU - Consolidated - Undrained Triaxial

TYPE OF FAILURE
A - Bulge
B - Single Shear Plane
C - Multiple Shear Plane
D - Vertical Fracture
E - Slickensides

FUGRO

Runway 18 Approach Lighting Project

SUMMARY OF TEST RESULTS - BORING B-3

Houma-Terrebonne Airport

LELAP Lab ID #10001

Houma, Louisiana

Project No. **04.00251305**

PLATE A-3

FCBR SUMMARY OF TESTS 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

Sample No.	Depth (ft)	Identification Tests						Field Shear Strength Estimate		Miniature Vane Tests		Compression Tests								
		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (PI)	Moisture Content (%)	Dry Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer (ksf)	Torvane (ksf)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	Type Test	Moisture Content (%)	Confining Pressure (psi)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	ε_{50} Strain (%)	Dry Unit Weight (pcf)	Failure Strain (%)	Type of Failure
B1A	0.0-1.0	68	24	44	35															
LT1B	1.0-2.0				33	87		1.25				U	33		0.84		1.7	87	8.0	C
B2B	3.0-4.0	36	22	14	34				0.42											
B3A	4.0-6.0				35		95.6													
									0.08											
									0.26											
B5A	8.5-10.0	32	21	11	33		95.5													
B8A	14.5-16.0				30		93.9													
B9A	18.5-20.0				36		92.4													
B10A	23.5-25.0				23															
B11A	28.5-30.0				39		61.8													
B12A	33.5-35.0				55		98.5													
LT13B	39.0-40.0				59	65			0.38			UU	59	31	0.54		0.6	65	6.8	C

Notes:

NP = Non-Plastic Material

TYPE OF TEST
U - Unconfined Compression
UU - Unconsolidated - Undrained Triaxial
CU - Consolidated - Undrained Triaxial

TYPE OF FAILURE
A - Bulge
B - Single Shear Plane
C - Multiple Shear Plane
D - Vertical Fracture
E - Slickensides

FUGRO

Runway 18 Approach Lighting Project

SUMMARY OF TEST RESULTS - BORING B-4

Houma-Terrebonne Airport

LELAP Lab ID #10001

Houma, Louisiana

Project No. **04.00251305**

PLATE A-4

FCBR SUMMARY OF TESTS 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

Sample No.	Depth (ft)	Identification Tests						Field Shear Strength Estimate		Miniature Vane Tests		Compression Tests								
		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (PI)	Moisture Content (%)	Dry Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer (ksf)	Torvane (ksf)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	Type Test	Moisture Content (%)	Confining Pressure (psi)	Shear Strength (ksf)	Remolded Shear Strength (ksf)	ε_{50} Strain (%)	Dry Unit Weight (pcf)	Failure Strain (%)	Type of Failure
B1A	0.0-1.0	49	18	31	26															
LT1B								0.75												
LT2A	2.0-3.0				33	87						U	33		0.40		2.5	87	13.5	C
B2B	3.0-4.0				36		95.0		0.36											
									0.52											
B4A	6.5-8.0				36		94.8													
B6A	10.5-12.0				33		95.9													

Notes:

NP = Non-Plastic Material

TYPE OF TEST
U - Unconfined Compression
UU - Unconsolidated - Undrained Triaxial
CU - Consolidated - Undrained Triaxial

TYPE OF FAILURE
A - Bulge
B - Single Shear Plane
C - Multiple Shear Plane
D - Vertical Fracture
E - Slickensides

FUGRO

Runway 18 Approach Lighting Project

SUMMARY OF TEST RESULTS - BORING B-5

Houma-Terrebonne Airport

LELAP Lab ID #10001

Houma, Louisiana

Project No. **04.00251305**

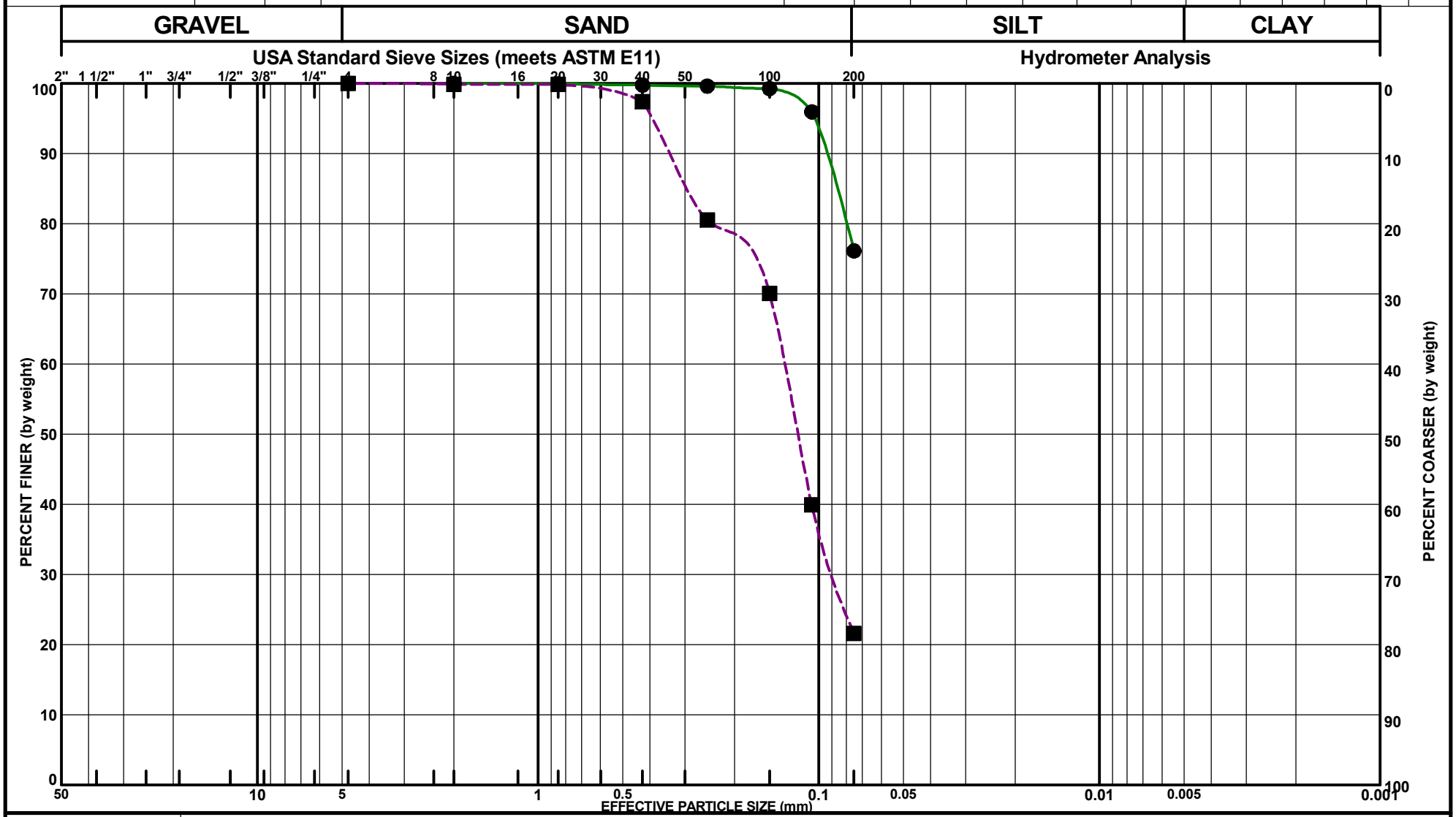
PLATE A-5

FCBR SUMMARY OF TESTS 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

Appendix B

Grain Size Curves

Boring Number	Sample Number	Depth (ft)	Material Description	%Gravel	%Sand	%Silt	%Clay	D100	D60	D30	D10	Cc	Cu	LL	PL	PI
● B-1	B8A	14.5-16	SILT WITH SAND (ML), dark brown, with clay traces	0.0	23.9			2								
■ B-1	B12A	33.5-35	SILTY SAND (SM), dark gray, with clay traces	0.0	78.4			4.75	0.134	0.088						
▲																



FCBR GSA LANDSCAPE 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

FUGRO

Runway 18 Approach Lighting Project

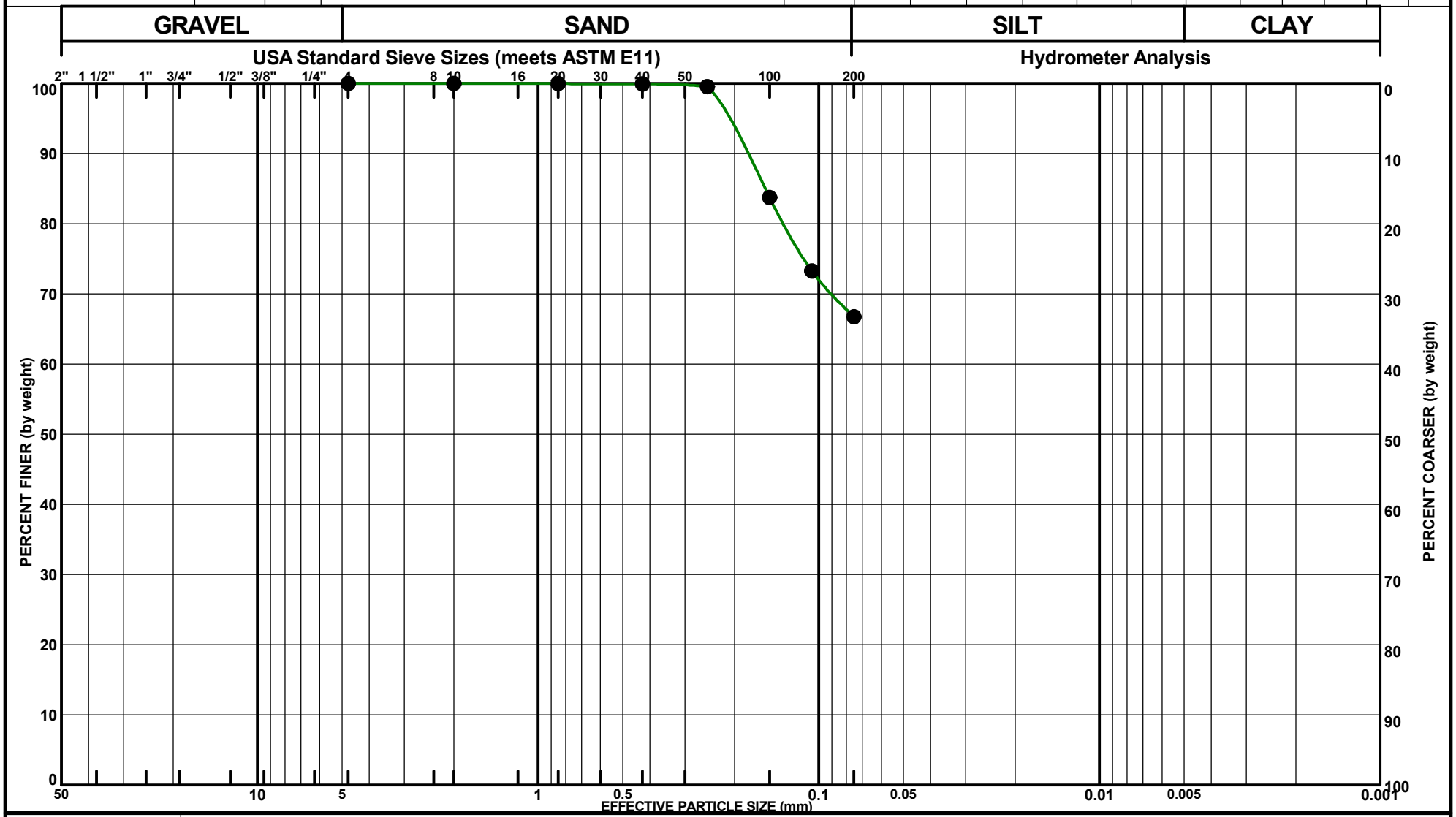
Houma, Louisiana

PARTICLE-SIZE ANALYSIS

ASTM D422/D6913/C136

Tested By:	Date Tested:	Reviewed By:	Date Reviewed:	Project No.	
Jim Guohua	3/21/2024	Deborah Meyer-Sayer	3/26/2024	04.00251305	PLATE B-1

Boring Number	Sample Number	Depth (ft)	Material Description	%Gravel	%Sand	%Silt	%Clay	D100	D60	D30	D10	Cc	Cu	LL	PL	PI
● B-2	B11A	28.5-30	SANDY FAT CLAY (CH), gray	0.0	33.2			4.75								
■																
▲																



FCBR GSA LANDSCAPE 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

FUGRO

Runway 18 Approach Lighting Project

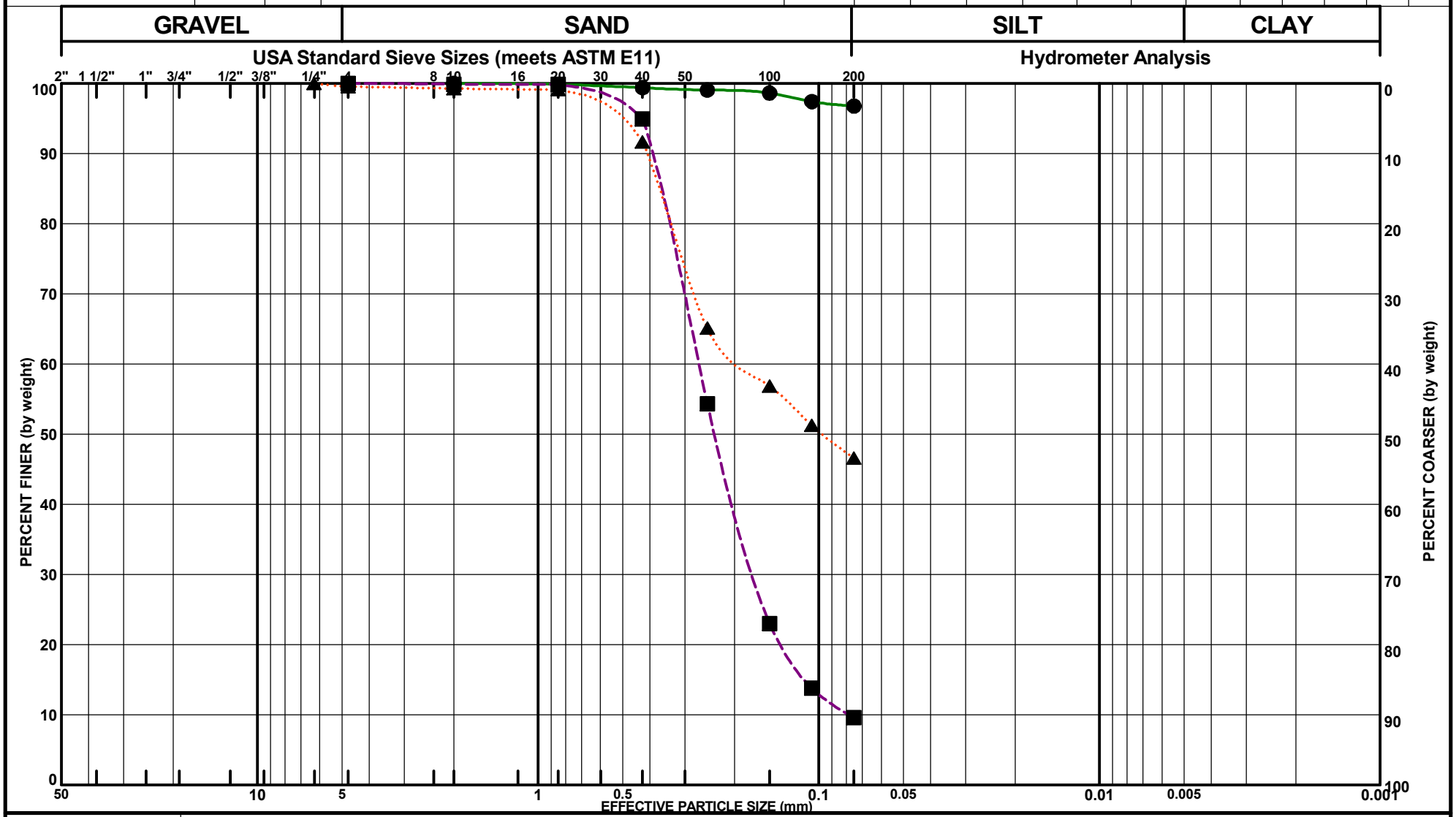
Houma, Louisiana

PARTICLE-SIZE ANALYSIS

ASTM D422/D6913/C136

Tested By:	Date Tested:	Reviewed By:	Date Reviewed:	Project No.	
Jim Guohua	3/21/2024	Deborah Meyer-Sayer	3/26/2024	04.00251305	PLATE B-2

Boring Number	Sample Number	Depth (ft)	Material Description	%Gravel	%Sand	%Silt	%Clay	D100	D60	D30	D10	Cc	Cu	LL	PL	PI
● B-3	B4A	6.5-8	LEAN CLAY (CL), dark gray, with sand pockets	0.0	3.2			2								
■ B-3	B11A	28.5-30	POORLY GRADED SAND WITH SILT (SP-SM), dark brown, with shell fragments and clay traces	0.0	90.4			4.75	0.269	0.168	0.078	1.35	3.47			
▲ B-3	B12A	33.5-35	CLAYEY SAND (SC), dark brown, with shell fragments	0.5	52.9			6.3	0.182							



FCBR GSA LANDSCAPE 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

FUGRO

Runway 18 Approach Lighting Project

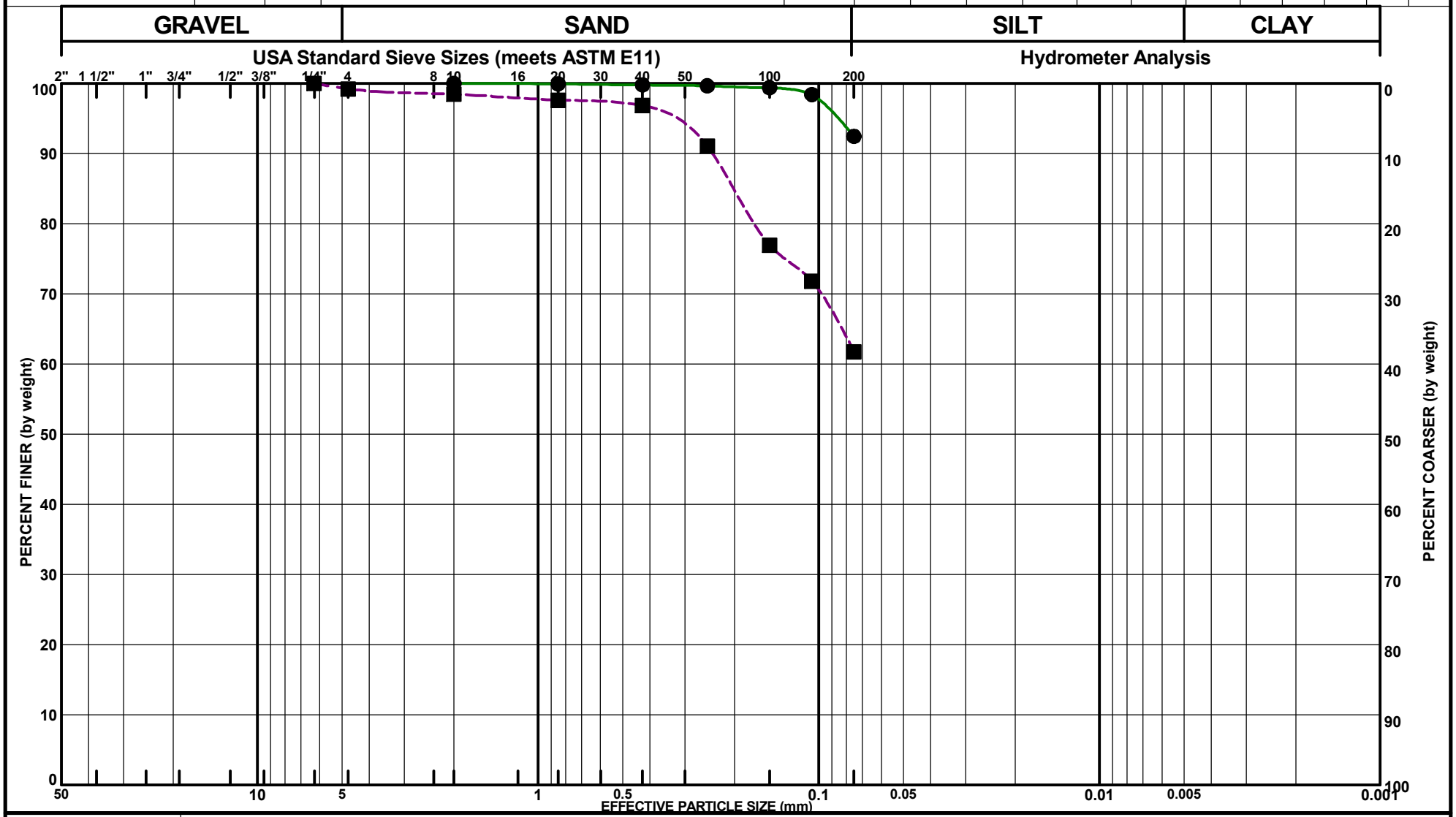
Houma, Louisiana

PARTICLE-SIZE ANALYSIS

ASTM D422/D6913/C136

Tested By:	Date Tested:	Reviewed By:	Date Reviewed:	Project No.	
Jim Guohua	3/27/2024			04.00251305	PLATE B-3

Boring Number	Sample Number	Depth (ft)	Material Description	%Gravel	%Sand	%Silt	%Clay	D100	D60	D30	D10	Cc	Cu	LL	PL	PI
● B-4	B9A	18.5-20	LEAN CLAY (CL), dark gray, with organics and sand pockets	0.0	7.6			2								
■ B-4	B11A	28.5-30	SANDY FAT CLAY (C), dark gray, with wood fragments	0.8	37.5			6.3								
▲																



FUGRO

Runway 18 Approach Lighting Project

Houma, Louisiana

PARTICLE-SIZE ANALYSIS

ASTM D422/D6913/C136

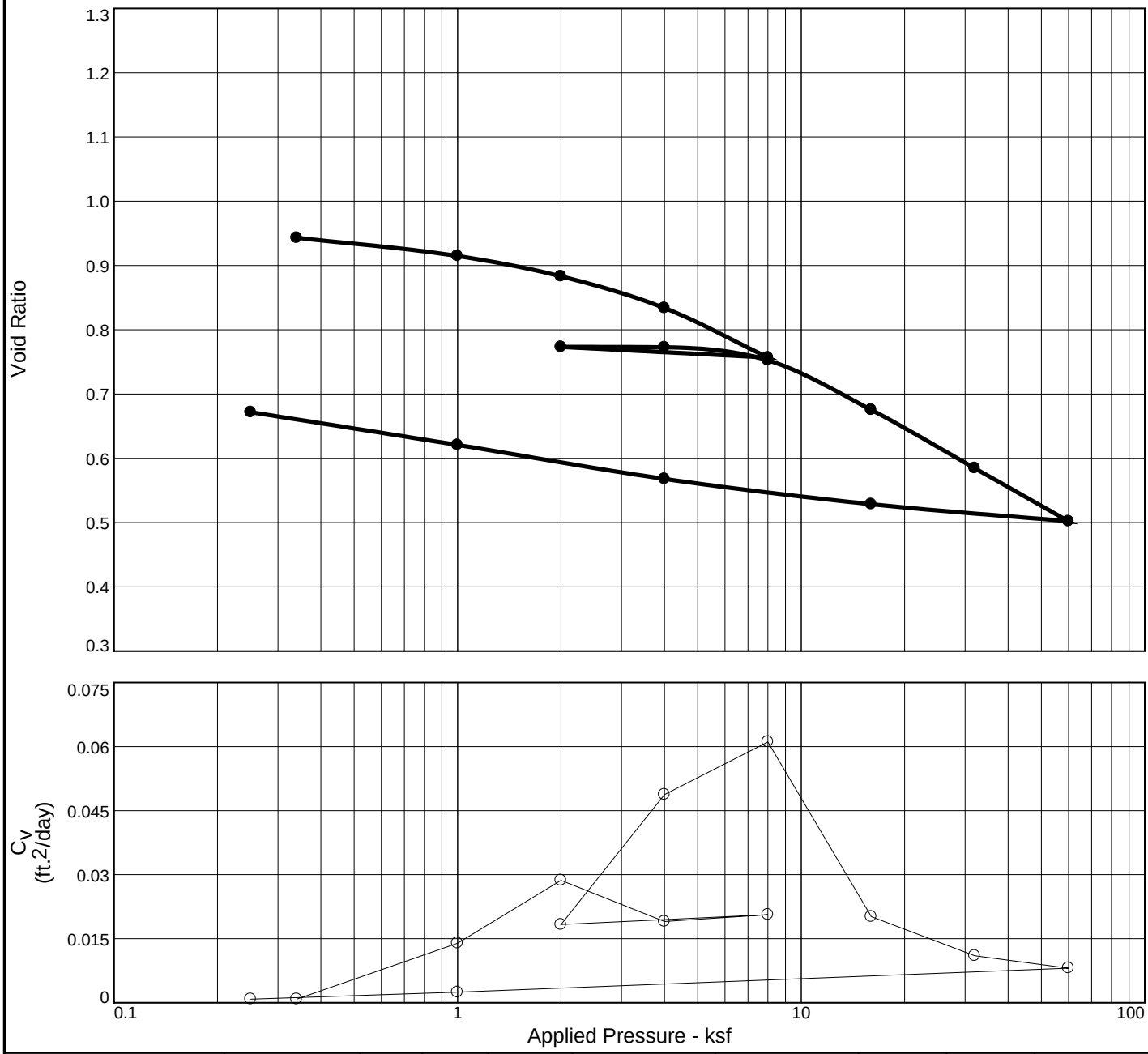
Tested By:	Date Tested:	Reviewed By:	Date Reviewed:	Project No.	
Jim Guohua	3/27/2024			04.00251305	PLATE B-4

FCBR GSA LANDSCAPE 251305.GPJ FUGRO DATA TEMPLATE 042610.GDT 05/14/24

Appendix C

Consolidation Test Curve

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
97.0 %	34.7 %	84.9	56	36	2.653		3.5	0.30	0.03	0.950

MATERIAL DESCRIPTION	USCS	AASHTO
FAT CLAY, gray, with silt lenses and ferrous stains	CH	

Project No. 04.00251305 **Client:** JACOBS

Project: RUNWAY 18 APPROACH LIGHTING PROJECT
HOUMA-TERREBONNE AIRPORT

Source of Sample: B-2 **Depth:** 4.0 **Sample Number:** LT3A

Remarks:

Date Tested: 04/02/2024
Tested by: S. Cobb
Reviewed by: V. Le

Fugro USA Land, Inc.

Baton Rouge, LA

PLATE C-1

Appendix D

Corrosion Test Results

FUGRO USA MARINE
ANALYTICAL SERVICES
6100 HILLCROFT
HOUSTON, TEXAS 77081
713 369-5400

ANALYTICAL RESULTS

JOB No:	**04.0025-1305**	**DATE SAMPLED:**	**NA**
DATE RECEIVED:	**3/27/2024**	**TIME SAMPLED:**	**NA**
TIME RECEIVED:	**700**	**REPORTED TO:**	**HOLLIER**
BY:	**SD**	**OFFICE:**	**BATON ROUGE**
REPORT DATE:	**4/2/2024**		

		pH	Sulfate	Chloride	Resistivity	REDOX	
		D 4972	**D 516****	**D-512 ****	**G-57 @ 15.5°C**	**SM 2580B**	
		(SU)	(mg/kg)	(mg/kg)	(ohm-cm)	mV	
		DATE	**DATE**	**DATE**	**DATE**	**DATE**	
		04/02/24	04/01/24	04/01/24	04/01/24	4/2/2024	**TECH**
SAMPLE ID	**LAB No.**						
B-1, B2A, 2 - 3'	24-0327093	6.9	<100 *	<100 *	2048	509	SD/EK
B-2, B2A, 2 - 3'	24-0327094	6.7	<100 *	<100 *	1249	512	SD/EK
B-3, B2A, 2 - 3'	24-0327095	7.9	<100 *	<100 *	2519	516	SD/EK

Submitted by,

QC: 025-24

* Dry Weight Basis

Steve De Gregorio
Chemist

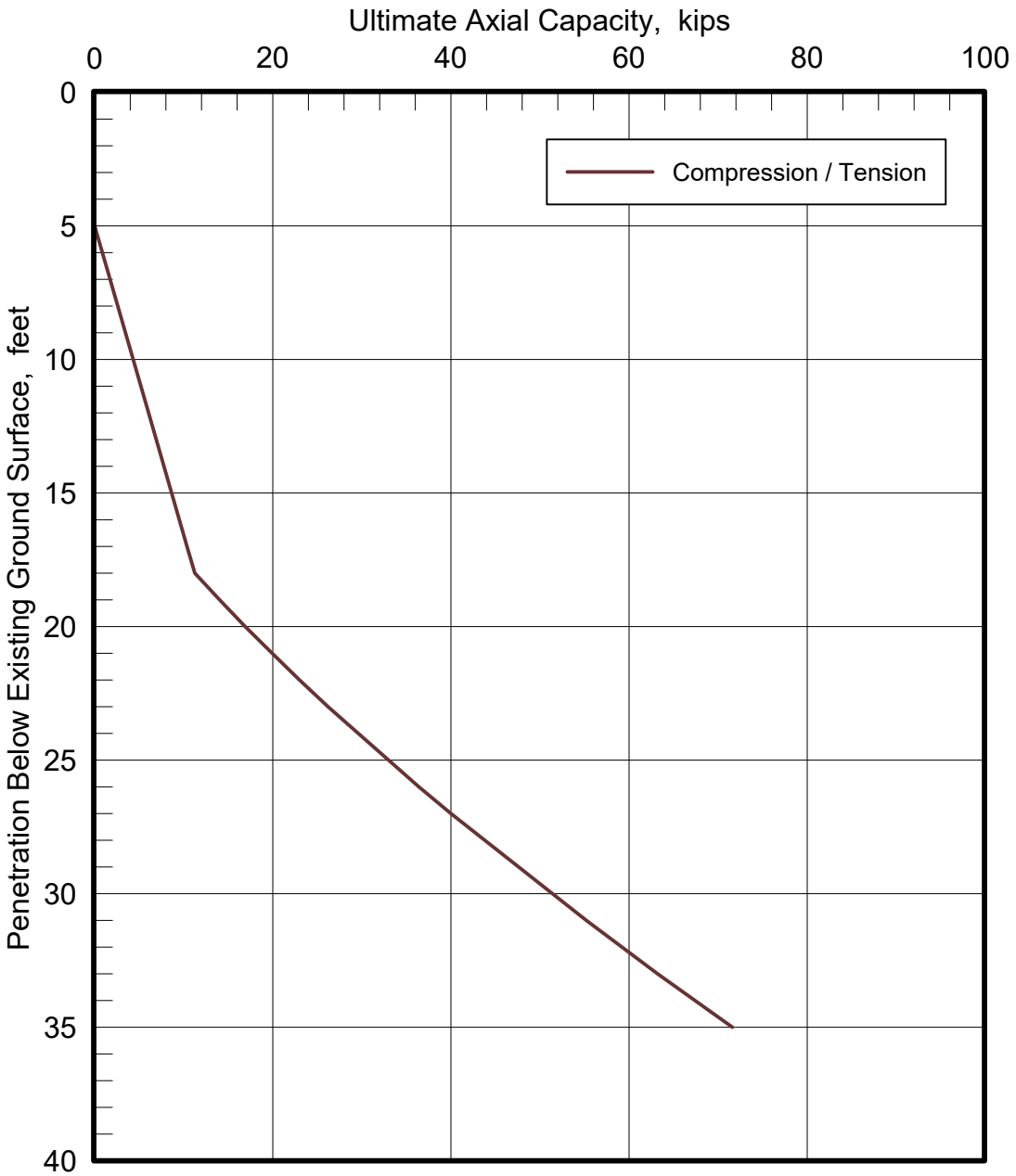
**** WATER EXTRACTION PERFORMED BY USING A 1:10 RATIO OF SAMPLE AND REAGENT WATER FOLLOWED BY CENTRIFUGE AND VACUUME FILTRATION. THE WATER EXTRACT IS THEN ANALYZED USING THE ASTM D-512 AND D-516 METHODS**

THE RESULTS RELATE AS TO THE LOCATION TESTED AND NO OTHER REFERENCE SHALL BE MADE. THIS REPORT SHALL NOT BE REPRODUCED EXCEPT IN FULL WITHOUT THE WRITTEN APPROVAL OF THE LABORATORY

END OF REPORT

Appendix E

Ultimate Axial Capacity Curves

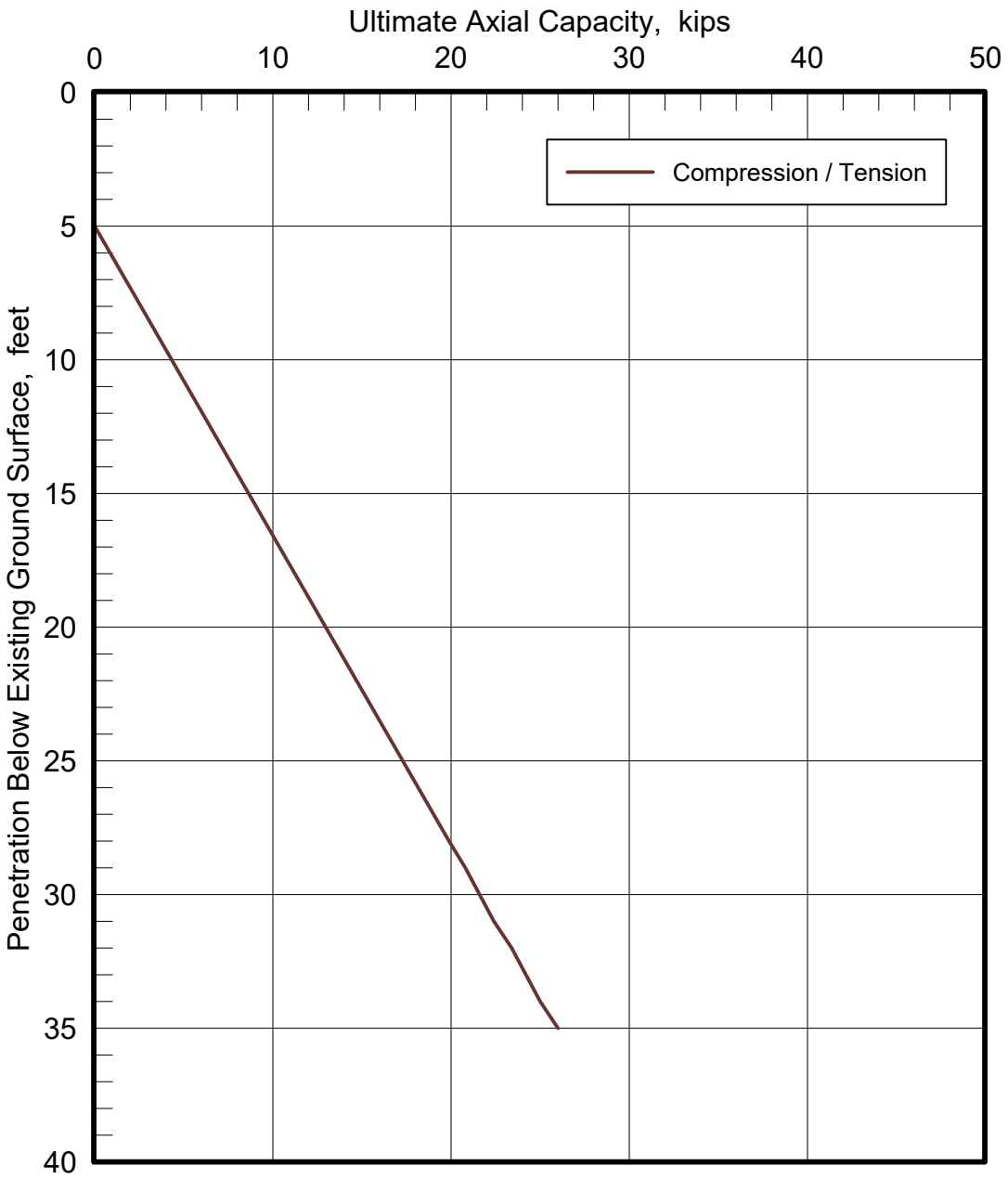


NOTES:
1. This curve represents ***ultimate*** values of axial capacity for both compression and tension.
2. The curve should be used for design of individual, isolated pile as discussed in the report text.
3. See report text for recommended safety factors and other discussions on the use of this curve.
4. Penetration refers to the depth below bottom of foundation.

ULTIMATE AXIAL CAPACITY CURVE
12-INCH DIA. AUGERED CAST-IN-PLACE (ACIP) PILE (B-1 & B-3)
RUNWAY 18 APPROACH LIGHTING PROJECT - HOUMA TERREBONNE AIRPORT
HOUMA, LOUISIANA

Project No.: 04.00251305

Reviewed by: Viet Le	Date: 05/13/24	Approved by: Marty Skyrman	Date: 05/13/24	PLATE E-1

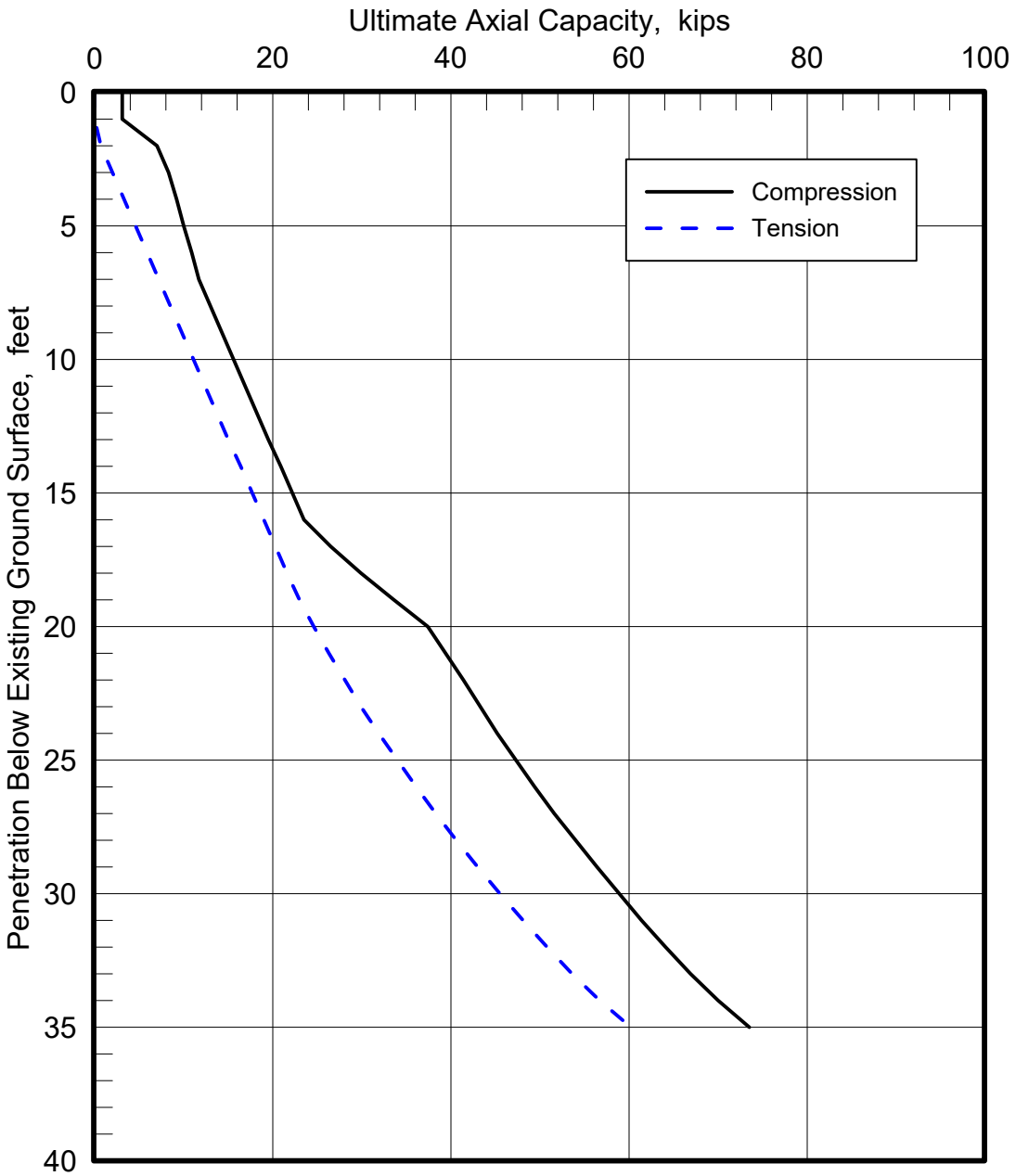


NOTES:
1. This curve represents ***ultimate*** values of axial capacity for both compression and tension.
2. The curve should be used for design of individual, isolated pile as discussed in the report text.
3. See report text for recommended safety factors and other discussions on the use of this curve.
4. Penetration refers to the depth below bottom of foundation.

ULTIMATE AXIAL CAPACITY CURVE
12-INCH DIA. AUGERED CAST-IN-PLACE (ACIP) PILE (B-2, B-4, & B-5)
RUNWAY 18 APPROACH LIGHTING PROJECT - HOUMA TERREBONNE AIRPORT
HOUMA, LOUISIANA

Project No.: 04.00251305

Reviewed by: Viet Le	Date: 05/13/24	Approved by: Marty Skyrman	Date: 05/13/24	PLATE E-2

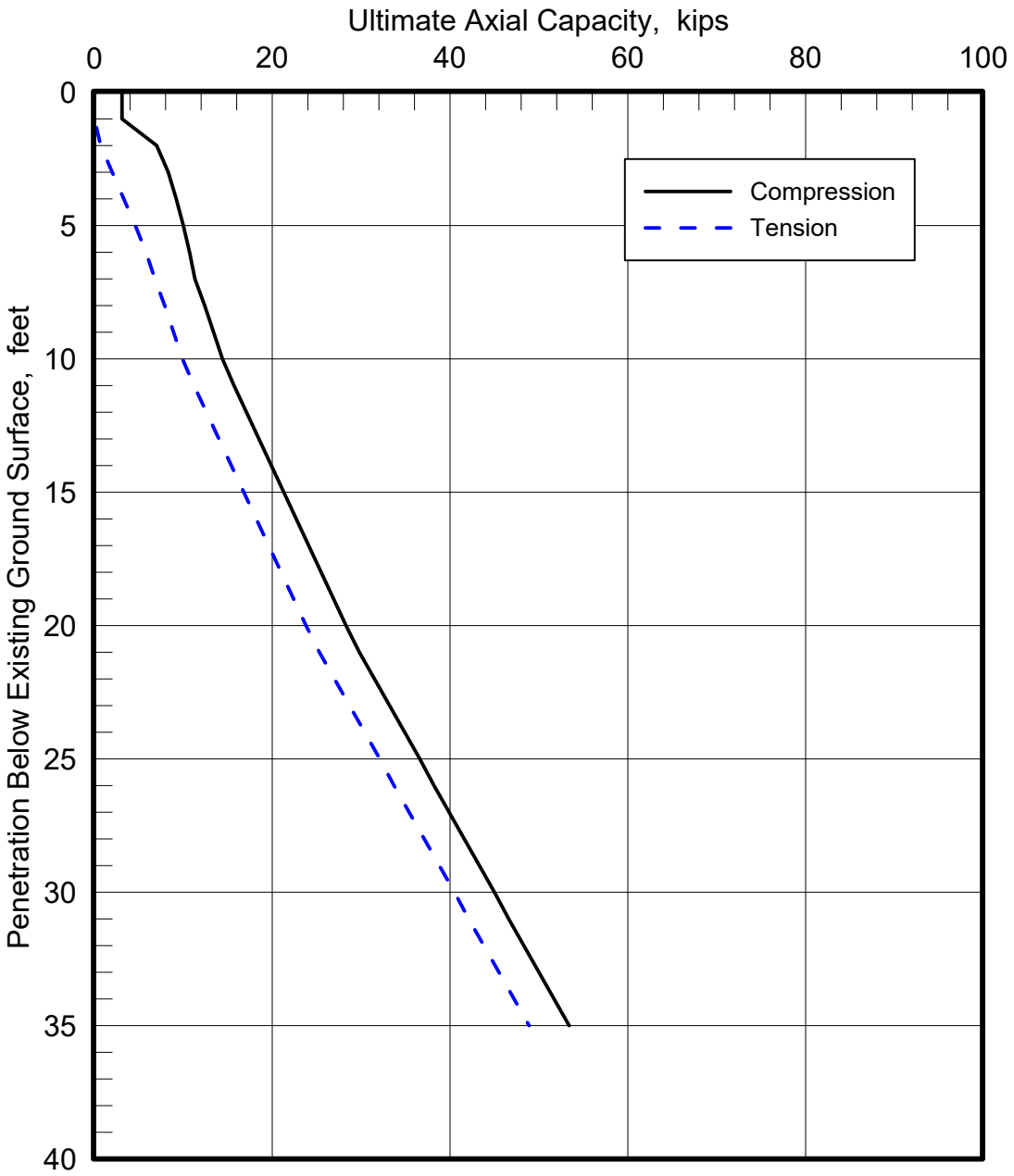


NOTES:
1. This curve represents **ultimate** values of axial capacity for compression (with end bearing) and tension (without end bearing). Factors of safety and installation guidelines are discussed in the text.
2. This curve is for a single isolated pile. Group effects are discussed in the text.
3. Tension capacity should be limited to the depth of steel reinforcement that can develop proposed tensile loads.
4. This curve includes geotechnical axial capacity. Structural capacity should be checked by designer.
5. Penetration refers to depth below bottom of foundation.

ULTIMATE AXIAL CAPACITY CURVE
12-INCH SQUARE PRECAST CONCRETE PILE (B-1 & B-3)
RUNWAY 18 APPROACH LIGHTING PROJECT - HOUMA TERREBONNE AIRPORT
HOUMA, LOUISIANA

Project No.: 04.00251305

Reviewed by: Viet Le	Date: 05/13/24	Approved by: Marty Skyrman	Date: 05/13/24	PLATE E-3



NOTES:
1. This curve represents **ultimate** values of axial capacity for compression (with end bearing) and tension (without end bearing). Factors of safety and installation guidelines are discussed in the text.
2. This curve is for a single isolated pile. Group effects are discussed in the text.
3. Tension capacity should be limited to the depth of steel reinforcement that can develop proposed tensile loads.
4. This curve includes geotechnical axial capacity. Structural capacity should be checked by designer.
5. Penetration refers to depth below bottom of foundation.

ULTIMATE AXIAL CAPACITY CURVE
12-INCH SQUARE PRECAST CONCRETE PILE (B-2, B-4, & B-5)
RUNWAY 18 APPROACH LIGHTING PROJECT - HOUMA TERREBONNE AIRPORT
HOUMA, LOUISIANA

Project No.: 04.00251305

Reviewed by: Viet Le	Date: 05/13/24	Approved by: Marty Skyrman	Date: 05/13/24	PLATE E-4