EUSTIS ENGINEERING L.L.C.
SINCE 1946

18 February 2019

Compton Engineering, Inc.
Post Office Box 686
Pascagoula, Mississippi 39567

Attention Mr. James Buras, P.E.

Ladies and Gentlemen:

Geotechnical Exploration
Hancock County Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi
Eustis Engineering Project No. G0389

Transmitted is an electronic copy of our engineering report covering a geotechnical exploration for the subject project. Hard copies are available upon request.

Thank you for asking us to perform these services.

Your very truly,

EUSTIS ENGINEERING L.L.C.

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GEOTECHNICAL EXPLORATION

HANCOCK COUNTY PORT AND HARBOR COMMISSION

RESTORE PROJECT

PROPOSED FACILITY

PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING PROJECT NO. G0389

FOR
COMPTON ENGINEERING, INC.
BAY ST. LOUIS, MISSISSIPPI

By
Eustis Engineering L.L.C.
Gulfport, Mississippi

18 FEBRUARY 2019

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INTRODUCTION

1. This report contains the results of a geotechnical exploration performed for Hancock County Port and Harbor Commission's (HCPHC) proposed facility to be located at Port Bienville near Pearlington, Mississippi. Our professional services were performed in general accordance with the scope of service outlined in Eustis Engineering L.L.C.'s revised proposal dated 17 October 2018. Authorization to proceed with the exploration was provided by signed acceptance of our proposal on 18 October 2018 by Mr. James Buras, P.E., representing Compton Engineering, Inc. (Compton), the designer of record. HCPHC is the project owner.

2. This report has been prepared in accordance with generally accepted geotechnical engineering practice for the exclusive use of Compton and HCPHC for specific application to the subject site. In the event of any changes in the nature, design, or location of the proposed construction, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed, and the conclusions of this report are modified and verified in writing. Should these data be used by anyone other than Compton or HCPHC, the user should contact Eustis Engineering for interpretation of data and to secure other information which may be pertinent to the project.

3. The analyses and recommendations contained in this report are based, in part, on data obtained from the soil borings, cone penetration tests (CPTs), and dynamic cone penetration tests (DCPTs) performed for this exploration. The logs of the borings, CPTs, and DCPTs are considered representative of subsurface conditions at their respective locations on the dates completed. No warranty is given that the logs of the borings, CPTs, or DCPTs are representative of subsurface or ground water conditions at other locations or times. The nature and extent of variations in subsurface or ground water conditions, between and away from the boring, CPT, and DCPT locations, may not become evident until construction. If such variations then appear, it will be necessary to reevaluate the recommendations contained in this report.

4. Recommendations and conclusions contained in this report are to some degree subjective having partial basis in engineering judgement and experience particular to the design engineer. For this reason, this report should not be included in the contract plans and specifications. However, the results of the soil borings, laboratory tests, CPTs, and DCPTs, contained in Appendices I through III of this report, may be included in the plans and specifications. The construction contractor will then be obliged to make an independent interpretation of subsoil conditions at the site and their potential impact on means and methods used to achieve the plans and specifications.

SCOPE

5. The scope of the exploration included drilling 14 undisturbed sample type soil test borings and performing 15 CPTs to evaluate subsoil conditions and stratification, and to obtain samples of the various strata encountered. This scope was expanded to include DCPTs and auger borings to further evaluate existing site access roadways. Soil mechanics laboratory tests, performed on samples obtained from the borings as well as in situ tests

performed during the CPTs and DCPTs, were used to evaluate the physical properties of the various substrata. The borings, laboratory tests, and CPTs were used to provide estimates of allowable soil bearing values, allowable axial pile load capacities for various types of piles, and settlement for foundation design of project components across the site. These data were also used to develop design recommendations for a new keyway slip partially constructed with a bulkhead system and for mooring dolphins to be constructed in the new and existing keyway slip off of Mulatto Bayou. In addition, recommendations were developed for site preparation, general construction procedures, railroad embankments, and components and thicknesses for aggregate surface pavements.

6. Eustis Engineering's scope of work does not include the exploration or detection of the presence of any biological pollutants in or around the subject site. The term "biological pollutants" includes but is not limited to molds, fungi, spores, bacteria, viruses, and the byproducts of any such biological organisms.

FIELD EXPLORATION

7. Subsurface Exploration. Details of the depths and designations of the undisturbed borings, CPTs, auger borings, and DCPTs are presented in Table 1. The undisturbed borings were drilled between 23 October 2018 and 6 November 2018 and the CPTs were performed between 26 and 30 October 2018 at the approximate locations shown on Figure 1. The DCPTs and auger borings were performed on 31 January 2019. The borings and CPTs were grouped into profiles based on their stratigraphy and proposed project features in the vicinity of the exploratory locations. The exploration locations for each profile are presented in Table 1 and shown on Figure 1.

TABLE 1: SUBSURFACE EXPLORATION DETAIL

PROFILE	FEATURE(S)	EXPLORATION LOCATION	EXPLORATION TYPE	DEPTH IN FEET
A	Barge Slip (BS), Crushed Stone (CS) Material Storage, and Pile Supported Dock Apron	CSB-1	Boring	80
		CSB-2	Boring	80
		CSB-3	Boring	25
		BSB-1	Boring	80
B	200' X 500' Warehouse and Manufacturing (WM) Building	WMC-1	CPT	44.6
		WMB-1	Boring	25
		WMC-2	CPT	45
		WMC-3	CPT	41.3
		WMB-2	Boring	80
		WMC-4	CPT	40.4
C	Power Plant (PP) Building, Security Gate (SG) Building, Guardhouse (GH) Building, and Water Tower (WT)	PPC-1	CPT	45.6
		SGC-1	CPT	42.5
		GHB-1	Boring	80
		WTC-1	CPT	42.2
D	230' X 250' Warehouse or Staging (WS)	WSC-1	CPT	44.6
		WSB-1	Boring	25
		WSC-2	CPT	43.6
E	Tank Farm (TF) - West (Seven Tanks)	TFC-1	CPT	80.4
		TFB-1	Boring	80
		TFC-2	CPT	42.2
		TFB-2	Boring	80
		TFC-3	CPT	70.4
F	Tank Farm (TF) - East (Four Tanks)	TFC-4	CPT	80.5
		TFB-3	Boring	80
		TFC-5	CPT	80.5
		TFB-4	Boring	80
		TFC-6	CPT	80.7
G	Railroad (RD) and Roadways	RDB-1	Boring	25
		RDB-2	Boring	25
		DCPT-1	DCPT	5
		A-1	Auger Boring	5
		DCPT-2	DCPT	5
		A-2	Auger Boring	5
		DCPT-3	DCPT	5
		A-3	Auger Boring	5
		DCPT-4	DCPT	5
		A-4	Auger Boring	5
		DCPT-5	DCPT	5
		A-5	Auger Boring	5

8. Upon completion of the field operations, the borings and CPTs were grouted and/or sealed with cement-bentonite grout mix in accordance with current regulatory requirements. Detailed descriptive logs of the undisturbed borings and laboratory tests performed for the exploration are shown in both tabular and graphical form in Appendix I. Records of the CPT soundings performed for the exploration are shown in Appendix II. Results of the DCPTs and auger borings are shown in Appendix III. GPS coordinates were obtained at each of Eustis Engineering's exploratory locations using a handheld unit and should be considered approximate. These coordinates are shown on the logs in Appendices I through III in the form of latitude and longitude.

9. <u>Undisturbed Borings.</u> Undisturbed samples of cohesive or semi-cohesive subsoils were obtained at close intervals or changes in strata using a 3-in. diameter thinwall Shelby tube sampling barrel. The samples were immediately extruded from the sampling barrel, inspected, and visually classified by Eustis Engineering's soil technician. Pocket penetrometer tests were performed on the soil samples to give a general indication of their shear strength or consistency. The results of these tests are shown on the boring logs under the column heading "PP." Representative samples were placed in moisture proof containers and sealed for preservation of their natural moisture content.

10. Samples of cohesionless and semi-cohesive materials were obtained during the performance of in situ Standard Penetration Tests. This test consists of driving a 2-in. diameter sampler 1 foot into the soil after first seating it 6 inches. A 140-lb weight dropped 30 inches is used to advance the sampler. The number of blows required to drive the sampler is indicative of the relative density of cohesionless soils and the consistency of cohesive soils. The samples were retained in moisture proof containers for preservation of their natural moisture content. The results of the Standard Penetration Tests are shown on the boring logs under the column heading "SPT." The Standard

Penetration Test values presented on the boring logs have not been corrected for hammer efficiency or overburden in the in situ materials.

11. Auger Borings. The auger borings were performed using handheld equipment. Grab samples were collected from the auger blades and were visually inspected and classified by Eustis Engineering's soil technician. Representative samples were placed and sealed in moisture proof containers for transportation to our laboratory.

12. Cone Penetration Tests. The CPT soundings were performed using an electronic piezocone penetrometer having either a 5 or 10-ton capacity. The 5 and 10-ton piezocones have a 10-cm^2 cross-sectional area with a 60° apex angled tip and a 150-cm^2 friction sleeve area. The sleeve friction for a 5-ton piezocone is measured directly using a tension load cell. The sleeve friction for a 10-ton piezocone is measured by subtraction from the tip resistance. The testing was carried out in accordance with methods and procedures outlined in ASTM D5778-12. The penetrometer was hydraulically advanced into the ground at a rate of 2 cm/sec. During testing, CPT parameters (tip resistance, friction resistance, and pore pressure) were recorded at 5-cm depth intervals.

13. The CPT plots presented in Appendix II provide measurements of tip bearing (q_t), sleeve friction (f_s), and pore pressure (u_2). Also shown on these plots are interpreted undrained shear strengths (S_u), estimated standard penetration resistances (N_{60}), and interpreted soil behavior type (SBT). Undrained shear strength estimates and estimated standard penetration resistances are interpreted from correlations developed by Lunne, Robertson and Powell (1997 and 1986) and our engineering experience in the region. The interpreted SBT is determined according to the non-normalized friction-ratio based soil behavior chart established by Robertson and Campanella (1986). Please refer to the CPT correlation document in Appendix II following the CPT logs for additional details.

14. Dynamic Cone Penetration Tests. The DCPTs were performed in accordance with ASTM D6951-09 using a K-100 dynamic cone penetrometer. Five DCPTs (DCPT-1 through DCPT-5) were performed at the locations indicated as Borings A-1 through A-5, respectively, as shown on Figure 1. The DCPTs were performed using either a 10.1 or 17.6-lb hammer. With the DCPT device held in a vertical or plumb position, the hammer was repeatedly lifted and released from the standard height of 575 millimeter (22.6 inches) until the tip penetrated between 25 and 50 millimeter (1 to 2 inches) into the existing soils. The number of blows from the hammer required to penetrate the tip to this depth was recorded until an approximate depth of 5 feet was achieved. Estimations of the California Bearing Ratio (CBR) were made according to Formula No. 2 of the ASTM standard based on correlations developed by the U.S. Army Corps of Engineers (USACE). Plots of estimated CBR and interpreted bearing capacity values versus depth for each DCPT are presented in Appendix III. Note, these values are based on an equivalent 1-ft diameter plate and intended for the design of pavements. Use of these data for the design of structures is not appropriate. Eustis Engineering should be consulted if other uses are proposed.

LABORATORY TESTS

15. Soil mechanics laboratory tests, consisting of visual classification, natural water content, organic content (ORG), unit weight, percent passing the No. 200 mesh sieve (-#200), and either unconfined compression shear (UC) or one-point unconsolidated undrained triaxial compression shear (OB), were performed on samples obtained from the borings. In addition, Atterberg liquid and plastic limits tests were performed on selected representative samples to aid in classification and to give an indication of their relative compressibility. The results of these laboratory tests are shown on the boring logs in Appendices I and III.

16. Grain size sieve analysis (SV) were also performed on selected representative samples to determine particle size distribution. These tests were performed to help classify samples encountered in the deep borings. Plots of the particle distribution curves are provided on separate sheets following the boring logs in Appendix I.

17. In addition, consolidation (CONS) tests were performed on selected samples from the undisturbed borings. The CONS tests were performed to help define the stress history and compressibility parameters. The results of the CONS tests are shown on separate sheets following the boring logs in Appendix I.

CHEMICAL TESTS

18. Subsequent to our proposal, chemical testing was requested to assess the corrosivity potential of the site soils in the barge slip area as it pertains to the proposed anchorage tieback system. Two composite samples were submitted to Pace Analytical Services, LLC, St. Rose, Louisiana, for chemical testing on 21 January 2019. The chemical tests included pH, soil specific conductance (resistivity), sulfate content, and chloride content. Results of this testing are included in Appendix IV and summarized in Table 2.

TABLE 2: RESULTS OF CHEMICAL TESTING

COMPOSITE SAMPLE NO.	SAMPLE NOS.	pH	CHLORIDE CONTENT (PPM)	SULFATE CONTENT (PPM)	SPECIFIC CONDUCTANCE (UMHOS/CM)
1	CSB-1: 1 and 2 CSB-2: 2 and 3 CSB-3: 1 and 2	5.6	1,100	9,100	307
2	CSB-1: 3 and 4 CSB-2: 4 and 5 CSB-3: 3	6.0	2,230	10,400	276

19. Using guidelines established by the National Association of Corrosion Engineers and the Ductile Iron Pipe Research Association, soil resistivity can be used to predict the corrosivity of soils on metal. Resistivity is calculated as the reciprocal of specific conductance as outlined below. Ranges of resistivity to predict corrosion are shown in Table 3.

TABLE 3: RANGES OF RESISTIVITY TO PREDICT CORROSION

RESISTIVITY (OHM-CM)	DESCRIPTION
Below 500	Very Corrosive
500 to 1,000	Corrosive
1,000 to 2,000	Moderately Corrosive
2,000 to 10,000	Mildly Corrosive
Above 10,000	Progressively Less Corrosive

20. Composite Sample No. 1 had a specific conductance of 307 umhos/cm which converts to a resistivity of 3,257 ohm-cm. Composite Sample No. 2 had a specific conductance of 276 umhos/cm which converts to a resistivity of 3,623 ohm-cm. Based on these soil resistivities, the soils would be considered mildly corrosive to metal. The sulphate concentration was detected as 9,100 mg/kg (equal to approximately 9,100 ppm) in Composite Sample No. 1, resulting in severe sulphate exposure (i.e., within the range between 1,500 and 10,000 ppm) in accordance with ACI 318-11, Table 4.2.1. The sulphate concentration was detected as 10,000 mg/kg (equal to approximately 10,000 ppm) in Composite Sample No. 2, resulting in very severe sulphate exposure (i.e., above 10,000 ppm) in accordance with ACI 318-11, Table 4.2.1. These results should be reviewed to determine minimum material requirements for the project. Also note, the samples submitted for chemical testing were preserved in accordance with standard geotechnical practice and not environmental practice. Additional chemical testing may be required to determine the extent and severity of corrosion potential of the soils at the site.

DESCRIPTION OF SURFACE AND SUBSOIL CONDITIONS

21. Site Observations. The site is located near the intersection of Port and Harbor Drive and Road G in the Port Bienville Industrial Park approximately two and one-half miles southeast of Pearlington, Mississippi. Mulatto Bayou runs along the northwestern side of the site. At the time of our field activities, the site was observed to be a generally flat, grassy field with an existing keyway slip on the bayou and a railroad track on the site. Single-lane, unpaved roads, comprised of compacted miscellaneous fill, were observed throughout the site. Existing development near the site included other industrial developments.

22. General. Subsoils encountered at the project site are variable. For our recommendations, we subdivided the site into seven soil design reaches corresponding to Subsurface Profiles A through G as shown in Figures 1 and 2. The soil design reaches to be referenced for specific site features are summarized subsequently in Table 4 and are based on the boring and CPT designations shown in Table 1**.** Subsurface profiles, showing the interpreted stratigraphy for each soil design reach, are shown in Figure 2, Sheets 1 through 7, and referenced in Table 4.

TABLE 4: SOIL DESIGN REACHES

DESIGN REACH	BORING/CPT LOCATIONS	SITE FEATURES	SUBSURFACE PROFILE
BS/CS	CSB-1, CSB-2, CSB-3, and BSB-1	Barge Slip/Mooring and Crushed Stone Area	A Figure 2, Sheet 1 of 7
WM	WMC-1, WMB-1, WMC-3, WMB-2, and WMC-4	Warehouse and Manufacturing Areas	B Figure 2, Sheet 2 of 7
PP,SG,GH,WT	PPC-1, SGC-1, GHB-1, and WTC-1	Power Plant Security Gate Guardhouse Water Tower	C Figure 2, Sheet 3 of 7
WS	WSC-1, WSB-1, and WSC-2	Warehouse and Staging Areas	D Figure 2, Sheet 4 of 7
TF - West	TFC-1, TFB-1, TFC-2, TFB-2, and TFC-3	Tank Farm - West	E Figure 2, Sheet 5 of 7
TF - East	TFC-4, TFB-3, TFC-5, TFB-4, and TFC-6	Tank Farm - East	F Figure 2, Sheet 6 of 7
RD	RDB-1 and RDB-2	Railroad Embankment	G Figure 2, Sheet 7 of 7

23. As noted in Figure 2, these subsurface profiles do not attempt to identify subsurface conditions between or away from specific exploratory locations. Rather, these profiles are schematics of the generalized stratigraphies considered for our analyses. These groupings are based on similar stratigraphies, project features, and engineering design parameters. These groupings are not based on geologic variations, and geophysical testing was not conducted to assess subsurface variations.

24. <u>General Topography.</u> Reference to the existing site grades on the wetland contour map Sheet C0.0, provided by Compton, indicates significant variation in the ground surface elevations in some portions of the site. Our estimates of the ranges in existing grade and average grade for the seven project soil reaches are presented in Table 5.

TABLE 5: APPROXIMATE SITE GRADES

DESIGN REACH/FEATURE(S) (SUBSURFACE PROFILE)	ESTIMATED RANGE OF EXISTING GROUND SURFACE IN FEET (NAVD 88)	ESTIMATED AVERAGE GROUND SURFACE ASSUMED FOR ANALYSES IN FEET (NAVD 88)
BS/CS (A)	3 to 11	6
WM (B)	5 to 12	8
PP,SG,GH,WT (C)	6 to 9	8
WS (D)	10	10
TF – West (E)	11	11
TF – East (F)	11 to 12	11
RD (G)	10 to 12	11

25. <u>Stratigraphy - Soil Design Reach BS/CS (Barge Slip/Mooring and Crushed Stone Area).</u> Review of the logs of Borings CSB-1 through CSB-3 and Boring BSB-1 indicates near surface soils at these locations are generally fill to depths of 1 to 7 feet comprising loose brown and tan silt, loose to medium dense brown sand and clayey sand with roots and gravel, and medium stiff tan sandy clay with silty sand pockets and concretions. Below this fill, interbedded strata of soft to medium stiff reddish-brown, tan, gray, and light gray clay, silty clay, and sandy clay and medium dense greenish-gray clayey sand are encountered to depths of 12 to 15 feet that generally overlie loose to very dense tan, gray, and light gray fine sand and silty sand that continues to the terminal depths of 25 to 80 feet in the borings in this area. An exception to this general stratigraphy is encountered at Boring CSB-2 where a 5-ft thick stratum of soft gray silty clay is encountered between depths of 28 and 33 feet. Subsurface Profile A (Figure 2, Sheet 1) depicts the interpreted stratigraphy for these borings. CPTs were not performed in this soil reach.

26. <u>Stratigraphy - Soil Design Reach WM (Warehouse and Manufacturing Areas).</u> Review of the logs of Borings WMB-1 and WMB-2 indicates near surface soils at these locations are generally miscellaneous topsoil and fill to depths of 1 to 3 feet comprising soft to medium stiff tan sandy clay and silty clay with roots, organic matter, concretions, shell fragments, and gravel. Below this topsoil and fill, interbedded strata of very loose to medium dense tan and gray silty sand and medium stiff to stiff gray, light gray, and tan clay, sandy clay, and silty clay continue to depths of 11.5 to 15 feet that overlie medium dense to very dense tan, gray, and greenish-gray fine sand, silty sand, and clayey sand that continue to the terminal depth of 25 feet in Boring WMB-1 and to the 32-ft depth in Boring WMB-2. Next in the stratigraphy in Boring WMB-2, medium stiff gray clay is encountered to a depth of 41 feet that overlies very dense tan fine sand that continues to the terminal depth of 80 feet in this boring. Review of the logs of the CPTs performed in this area (WMC-1 through WMC-4) indicates a good correlation with this boring stratigraphy. Subsurface Profile B (Figure 2, Sheet 2) shows the interpreted stratigraphy at these boring and CPT locations.

27. <u>Stratigraphy - Soil Design Reaches PP, SG, GH, WT.</u> This soil design reach is defined as the project vicinity encompassing the water tower, guardhouse, scale, administration building, and power plant. Review of the log of Boring GHB-1 indicates near surface soils at this location are fill to the 1-ft depth comprising soft brown silty clay with gravel, fine sand, roots, and vegetation. Below this fill, medium stiff to stiff tan and gray clay and silty clay are encountered to the 12-ft depth that overlie loose to dense tan and light gray fine sand and clayey sand that extend to a depth of 28.5 feet. Next in the stratigraphy, very soft to medium stiff light gray, dark gray, and brown clay, organic clay, and silty clay are encountered to the 41.5-ft depth, followed by very dense light gray fine to medium sand that continues to a depth of 73.5 feet. Below this depth, stiff gray clay is encountered to the terminal depth of 80 feet in the boring. Review of the logs of the CPTs performed in

this area (PPC-1, SGC-1, and WTC-1) indicates a good correlation with this boring stratigraphy with practical refusal of the CPTs in the lower sand deposits. Subsurface Profile C (Figure 2, Sheet 3) includes the interpreted stratigraphy at the referenced boring and CPTs in this selected design reach.

28. Stratigraphy - Soil Design Reach WS (Warehouse and Staging Areas). Review of the log of Boring WSB-1 indicates near surface soils at this location are generally topsoil to the ½-ft depth comprising of medium stiff brown silty clay with fine sand pockets, roots, organic matter, and traces of shell fragments. Below the near surface soils, medium stiff brown sandy clay is encountered to the 3-ft depth that overlies medium dense to dense gray, tan, and reddish-brown silty sand and fine sand that continue to the terminal depth of 25 feet in the boring. Review of the logs of the CPTs performed in this area (WSC-1 and WSC-2) indicates a good correlation with this boring stratigraphy to this depth. In these CPTs, the sand generally extends to depths of 29 to 30 feet and is underlain by medium stiff to stiff cohesive deposits to depths of 41 to 43 feet. Below these depths, the CPTs encountered dense to very dense sands. The CPTs refused within these sand deposits at approximate depths of 44 to 45 feet. Subsurface Profile D (Figure 2, Sheet 4) shows the interpreted stratigraphy at this boring and CPTs in the warehouse and staging area.

29. Stratigraphy - Soil Design Reach TF - West (West Tank Farm). Review of the logs of Borings TFB-1 and TFB-2 indicates near surface soils at these locations are generally miscellaneous topsoil to depths of ½ to 2 feet that comprising soft brown silty clay with fine sand pockets and roots. Below these near surface deposits, interbedded strata of medium stiff to stiff light gray, tan, and reddish-brown sandy clay and medium dense tan and reddish-brown silty sand are generally encountered to depths of 8 to 11 feet that overlie sand deposits comprising loose to dense tan, brown, and gray fine sand and silty sand that extend to depths of 27 to 38.5 feet. Next in the stratigraphy, soft to stiff gray and dark gray clay,

silty clay, and sandy clay are encountered to depths of 52 to 58 feet. Strata of loose to very dense gray, light gray, and tan fine sand and silty sand are then encountered to terminal depths of 80 feet in these borings. Review of the logs of the CPTs performed in this area (TFC-1 through TFC-3) generally indicates a good correlation with these borings, although stiff clay appears to be present in the lower 14 feet of TFC-1. Subsurface Profile E (Figure 2, Sheet 5) provides a graphical depiction of these interpreted stratigraphies at these boring and CPTs.

30. <u>Stratigraphy - Soil Design Reach TF - East (East Tank Farm).</u> Review of the logs of Borings TFB-3 and TFB-4 indicates near surface soils at these locations are generally miscellaneous topsoil and fill to depths of 1 to 2 feet comprising medium stiff brown silty clay with silty sand pockets and vegetation and medium dense brown and gray fine sand with organic matter and roots. Below these near surface soils, interbedded strata of soft to stiff light gray and tan sandy clay and loose to medium dense tan, gray, and light gray clayey sand and silty sand extend to depths of 12 to 12.5 feet. Below these depths, loose to medium dense tan and gray fine sand and silty sand are encountered to depths of 32 feet that overlie interbedded strata of soft gray sandy clay and loose to medium dense gray clayey sand and silty sand that continue to depths of 48 to 53 feet. Next in the stratigraphy, soft to stiff gray clay generally continues to the terminal depths of 80 feet in the borings. Note, a medium dense gray sand was encountered in the final foot of Boring TFB-3. Review of the logs of the CPTs performed in this area (TFC-4 through TFC-6) indicates a good correlation with this boring stratigraphy. Subsurface Profile F (Figure 2, Sheet 6) provides the interpreted stratigraphy for these borings and CPTs.

31. Stratigraphy - Soil Design Reach RD (Railroad Embankment). Review of the logs of Borings RDB-1 and RDB-2 indicates near surface soils at these locations are generally miscellaneous fill and topsoil to depths of ½ to 2 feet comprising soft brown silty clay with trace roots and vegetation and loose tan and light brown clayey sand. Stiff light gray and tan silty clay is encountered below the topsoil to the depth of 6 feet in Boring RDB-1. Below this silty clay deposit and the sand fill in Boring RDB-2, medium dense to dense tan and reddish-brown fine sand, clayey sand, and silty sand are encountered to terminal depths of 25 feet in these borings. Refer to Subsurface Profile G for a graphical representation of these borings. CPTs were not performed in this soil reach.

32. Stratigraphy - Existing Unpaved Roadways. Five auger borings were performed through the existing unpaved roadways across the site, as shown on Figure 1. Review of the logs of the auger borings performed in the unpaved roadways indicates these locations are generally mantled by approximately 12 inches of tan, gray, and brown medium to fine sand and silty sand with roots, gravel, shell fragments, and crushed limestone with traces of clay pockets. Boring A-2 was performed on the shoulder of the existing roadway within a swale/ditch, and the near surface soils at this location generally comprise soft brown silty clay with few crushed limestone to the 12-in. depth below the ground surface. Below 12-in. depths in the auger borings, soils encountered were generally tan, gray, light gray, brown, and weak red silty clay and sandy clay. A 1-ft thick stratum of tan clayey silt with fine sand and few clay pockets is encountered below the near surface sands in Boring A-4.

33. Ground Water. In order to determine ground water conditions at the time of the field exploration, observations were made in auger borings drilled near the undisturbed borings. These auger borings were generally advanced without the addition of water to a depth where ground water was initially encountered. Additional measurements were

obtained after an observation period. Refer to Table 6 below for a summary of these observations.

TABLE 6: SUMMARY OF GROUND WATER OBSERVATIONS

AUGER BORING	GROUND WATER OBSERVATION	
	INITIAL OBSERVATION	AFTER OBSERVATION (OBSERVATION PERIOD IN HOURS)
TFB-1 A	3.5	2 (23.5)
WMB-2A	3	0.25 (25.5)[1]
CSB-2A	9	5.5 (32.5)
CSB-3A	9	2 (22.75)
WSB-1A	7	1.5 (19.5)
TFB-1A	3.5	2 (23.5)
GHB-1A	3	0.5 (7.25)
A-1	5	2 (5.75)

[1]Rain event overnight.

34. Note, standing water was observed at isolated portions of the site at the time of our field activities. The depth to ground water will vary with climatic conditions, drainage improvements, water levels in Mulatto Bayou, and other factors. The depth to ground water should be determined by those persons responsible for construction immediately prior to beginning work.

FOUNDATION ANALYSIS

Furnished Information/Background

35. We understand the HCPHC is planning to develop a facility in Port Bienville, Mississippi. Compton furnished a site plan with proposed features along with a summary of features

planned to be supported on shallow or deep foundations. The furnished site plan is superimposed on the site image and exploration plan on Figure 1 and is also included in Appendix V with other furnished data.

36. The design features anticipated to be founded on deep foundations include a pile supported apron adjacent to new bulkhead walls, mooring dolphins at the existing and new keyway slip, a water tower, and tank foundations in the tank farm areas. Features ***proposed*** to be supported on shallow foundations include two warehouses, a truck scale, a guardhouse, and an administration building. In addition, a new railroad extension, access roads, a parking lot, and a crushed stone laydown are all proposed at the site. The access roads will generally follow the alignment of the existing unpaved roads but will be widened. The railroad extension will tie into existing tracks at the site.

37. Per correspondence with Compton, the proposed railroad will be constructed at the approximate existing grade with 2 to 3 feet of soil excavated and replaced with ballast and subballast. A profile of a proposed bulkhead to be constructed on the bayou was also furnished that indicates a design mudline at el -16, a proposed anchor/tieback location at el 4, and a retained grade at el 11. New bulkhead walls will be constructed along Mulatto Bayou and the northern side of the new keyway slip only. Dredging of the bayou bank slope after construction of the bulkhead to el -16 is anticipated. The southern and eastern sides of the new keyway slip will not have bulkheads but will be constructed as open slope with riprap to armor against erosion. A cross-section detailing the proposed slope configuration was not furnished. The existing slip will not be dredged. We understand post-construction surcharge loads within 40 feet behind the proposed bulkhead wall will bear on a pile supported apron. Based on the furnished site plan, the apron is proposed for construction along Mulatto Bayou and one side of the new barge slip only. Beyond

the apron, loads in the crushed stone area are proposed to be supported at grade as discussed below.

38. Compton provided a 50-ton lateral load to be applied 20 feet above the mudline for analyses of proposed mooring structures in the new and existing keyway slips. Preliminary designs indicate the mooring structure will comprise 24-in. diameter open end steel pipe piles (⅜-in. wall thickness). Additionally, Compton furnished a wetlands contour map providing existing elevations of the site including the existing keyway slip and the adjacent bayou. A proposed grading plan was not furnished; however, Compton indicated the majority of the project features (tanks, warehouses, the water tower, truck scale, power plant, and guardhouse) will be raised to el 14 for construction, with site grading fill, and the bulkhead slip/crushed stone area raised to el 11.

39. In additional correspondence with Compton, it was indicated the crushed stone laydown area would be partially used to store rolls of steel cables offloaded from barges and other break bulk cargo. These cable spools would be transported across the pile supported aprons and stored in the crushed stone laydown area. The bearing intensity of these spools was indicated to be approximately 1,500 psf. Compton requested we evaluate a surcharge loading of 1,500 psf across the crushed stone laydown area.

Foundation Recommendations

40. Based on conversations with Compton, we understand all proposed tanks (TF), warehouses (WM), the water tower (WT), truck scale, power plant (PP), and guardhouse (GH) will require site grading fill placement to at least el 14 to reach construction grades. Based on this and review of furnished topographic survey data, we do not recommend the truck scale/guardhouse or the warehouse/manufacturing building be supported on

shallow foundations due to an estimated high settlement potential from site grading fill (unless these structures can tolerate our estimates of settlement due to fill and structural loads).

41. The warehouse/staging building (WS), the guardhouse/administration building (GH), and the power plant (PP) may be supported on shallow foundations provided the recommendations contained in this report regarding drainage and site preparation are adhered to, the foundations are designed to support the structural loads with an acceptable factor of safety, and the structures' foundation designs will accommodate potential settlements associated with fill placed to el 14 and other requirements to meet the project specifications. The structure foundations should be made as rigid as possible to minimize the effects of settlement and differential settlement. The near surface soils at the site are moisture sensitive. Without proper preparation and drainage, the underlying soils could produce excessive settlement and differential settlement. Therefore, site preparation and drainage should not be neglected.

42. All structures sensitive to settlement, including tanks, should be supported on deep foundations consisting of driven piles or augercast piles. Each pile supported structure and any structurally integrated appurtenant features should be supported on piles having the same approximate tip embedment below the existing ground surface to minimize differential settlement. Slabs should be cast monolithically with grade beams and pile caps to provide rigidity and minimize the potential for differential settlement.

43. The new bulkhead walls may comprise an anchored sheetpile system provided the bulkhead sheetpiles are installed to the recommended tip embedments, the anticipated deflections from structural loads can be tolerated, anchorage systems are designed to resist the estimated anchor forces, and the sheetpiles elements are designed to resist the

estimated maximum bending moments provided in this report. Anchor tieback systems should also meet minimum setback requirements described subsequently. In addition, grade bearing loads within 40 feet behind the wall should be supported on a pile supported apron system as proposed. Installation criteria should be considered when selecting a bulkhead wall system.

44. We have not evaluated the stability of the non-bulkhead wall sides of the new keyway slip. Such an evaluation was beyond our project scope and would require a proposed side slope, topographic survey data, and grading plans for the areas beyond the slip on these sides. We have also not evaluated the effects of new project features on the existing slip. If new grade supported features and filling are planned within 40 feet of the existing slip, analyses should be performed to evaluate stability. Eustis Engineering should be retained to perform these analyses.

45. Preliminary analyses indicated a single 24-in. diameter pipe pile will deflect excessively under the furnished lateral load for a dredged mudline at el -16. To use a single monopole for the mooring structures with the furnished lateral load and mudline, we estimate the pipe pile would have to be larger than 72 inches in diameter to resist the furnished lateral loads and provide tolerable deflections. We understand alternate designs include utilizing a group of 24-in. diameter pipe piles for the mooring structure comprised of vertical and battered piles. Eustis Engineering has no objection to this methodology; however, our current scope of work does not include evaluation of a complex pile group subject to lateral loads. If the recommendations provided in this report for lateral resistance of battered pile systems and pile spacing are insufficient to meet the project requirements, Eustis Engineering should be retained to provide additional analyses.

46. Fill proposed to raise site grades should be placed as early as possible during the construction schedule to initiate consolidation and minimize post-construction settlement. Once site grading plans and foundation designs are finalized, Eustis Engineering should be contacted to verify our assumptions and recommendations.

47. The design of the site structures should meet the minimum standards outlined in ASCE/SEI 7-16, ASCE/SEI 24-14, and other applicable codes. For a flood hazard area, the site should be assessed for the risk of scour and loss of foundation support. If the risk of inundation is considered low, the proposed structures meeting allowable bearing capacity and settlement requirements may be supported on shallow foundations. Additional movements may be associated with foundations subjected to inundation and loss of support. Such movements are not quantifiable. In addition, the appropriate level of erosion protection should be provided to minimize any potential erosion.

48. Your structural engineer shall determine the appropriate foundation type for each structure/feature based on allowable capacity values, displacement tolerances, settlement estimates, and installation considerations presented in this report.

Site Preparation

49. Drainage During Construction. The initial step to prepare the land based features of the site for construction should be to establish adequate temporary and permanent drainage to prevent ponding of water and ensure immediate runoff of rainfall. We recommend the contractor maintain adequate surface drainage away from all foundations, pavements, and railroad lines/embankments during and after construction. This may be accomplished by setting grades to ensure positive drainage of water away from the

construction areas and providing adequate surface and subsurface drainage structures as required.

50. Permanent Drainage. The near surface soils are subject to a reduction in shear strength and excessive settlement if allowed to become saturated. Therefore, we recommend adequate permanent drainage be provided to collect rainfall away from the proposed foundations and pavements after completion of construction. Particular attention should be given to any surface founded structures, pavements, railroad embankments, and permanent drainage should be installed to discharge rainfall away from these features. All downspouts draining rainfall away from structures/tank roofs should be connected to pipes which discharge into the site's storm water collection and disposal system. Water should not be allowed to collect near the pavement, railroad, or structure foundation areas.

51. Demolition. Any structures or pavements located within the proposed construction areas not being incorporated into the final design should be completely demolished and removed from the site. Provisions should be made to locate any abandoned underground utilities and foundations which could impact new construction. Existing footings or abandoned pipes should be excavated and removed from the site. These structures could impact the performance of new foundations or pavements if not properly removed, proofrolled, and backfilled with structural fill material. Existing utilities may also be affected by the site preparation program and may experience excessive deflection due to the settlement of the existing ground surface. Consideration should be given to relocating all utilities from the areas susceptible to excessive settlement. Existing piles should not be incorporated into the final design, if encountered. Existing piles should not be removed but should be cut off 3 feet below any proposed grade beams, pile caps, footings, slabs, or railroad ballast, or 5 feet below any pavements or the existing ground

surface, whichever is greater. Removal and relocation of structures and obstructions should conform to the requirements outlined in Section 202 of the Mississippi Standard Specifications for Road and Bridge Construction (MSSRBC), 2017 edition.

52. Subgrade Preparation. After the stripping, clearing, and demolition operations, the exposed surface of the construction areas should be proofrolled with a bulldozer, roller compactor, or tracked vehicle exerting a contact pressure of 10 to 15 psi. The vibratory system on the compactor, if present, should not be used during proofrolling. Alternative proofrolling techniques may be proposed, but these methods should be approved by Eustis Engineering prior to their use at the site. Any depressions or weak areas identified should be thoroughly cleaned out to the surface of firm undisturbed soil and backfilled with a select structural fill material placed and compacted under controlled conditions. All clearing, demolition, proofrolling, and compaction operations should only be performed during periods of dry weather.

53. Consideration of Construction Sequence. Sequence of construction should be considered, yet this is beyond our current scope of service. The contractor should develop a construction sequence that will allow for the full installation of the bulkhead wall and anchorage system prior to excavations for the new keyway slip, particularly excavations that breach into Mullato Bayou. Proximity to the existing bayou bank and keyway slip with construction equipment should be carefully monitored if construction activities will be performed within close proximity of these features. Fill placement and temporary storage may also impact the stability of the existing keyway slip and bayou bank. These issues are a function of the construction contractor's means and methods and should be addressed in their work plan.

54. <u>Structural Fill.</u> A select granular material, such as locally available sand (SP), silty sand (SM), or clayey sand (SC), should be used as backfill and structural fill in areas with grade supported pavements and slabs. All fill material selected should be free of wood, roots, clay lumps, and other deleterious materials, and should have an organic content no greater than 5% by weight. Prior to transporting structural fill to the site, a sample of the borrow material should be tested to verify its conformance to the specifications. Existing onsite material may be suitable to be used as structural fill or form fill. Once the site has been cleared and stripped, this material should be tested to verify its conformance to the specifications.

55. If sand fill is selected as backfill, the material should conform to the requirements for an A-3 material according to the AASHTO Soil Classification System or for Class B3 borrow excavation given in Section 703.21 of the MSSRBC. This material will expedite construction and drain freely after inclement weather.

56. Silty sand (SM) or clayey sand (SC) fill (classified as an A-2-4 material by the AASHTO Soil Classification System, or as a Class B1 or B2 borrow excavation given in Section 703.21 of the MSSRBC) may be considered as an alternate to sand fill. However, this material is moisture sensitive and may require greater efforts (than the select sand fill) to process and compact.

57. <u>Placement and Compaction.</u> Structural fill used as form fill beneath pile supported features should be placed in loose lifts of 8 to 10 inches and compacted to at least 95% of its maximum dry density within ±2% of optimum moisture in accordance with ASTM D698 or 92% of its maximum dry density within ±2% of optimum moisture in accordance with ASTM D1557. Structural fill used beneath grade supported features or pavements should be placed in loose lifts of 6 to 8 inches and compacted to at least 98% of its maximum dry

density within ±2% of optimum moisture in accordance with ASTM D698 or 95% of its maximum dry density within ±2% of optimum moisture in accordance with ASTM D1557. Select fill or general fill used for non-structural grading should be spread in loose lifts of 10 to 12 inches and compacted by several passes of a bulldozer.

58. Quality Control. Density tests should be performed on each lift of the compacted structural fill to determine if the contractor has achieved the recommended density. We recommend one density test per 5,000 square feet of each lift of fill. All demolition, clearing, and compaction operations should be performed only during periods of dry weather. The contractor should exercise caution during and after inclement weather to ensure subsoil support is not degraded by construction operations.

Fill Settlement

59. A site grading plan was not available for preparation of this report. Therefore, we have evaluated several average site grades in various areas of the site as outlined below. Based on furnished information, we have assumed the ground surface below proposed structures and tanks may require variable heights of site grading fill to raise each feature footprint to an estimated construction grade at el 14. We have assumed non-structural areas will be filled to el 11. For these analyses, we have assumed a fill bulk unit weight of 120 pcf for site grading sand fill. Our assumptions and analyses are summarized in Table 7.

TABLE 7: ESTIMATES OF GRADE SETTLEMENT DUE TO FILL PLACEMENT

PROFILE	FEATURE (DESIGN REACH)	ESTIMATED FILL PLAN AREA IN FEET[1]	ESTIMATED FILL HEIGHT IN FEET	ESTIMATED SETTLEMENT AT CENTER OF FILL IN INCHES
E and F	Tank Farm (TF - West and TF - East)	500 x 260	3[2]	1¼ to 1¾
D	Warehouse/ Staging (WS)	230 x 250	6[2]	½ to ¾
B	Warehouse/ Manufacturing (WM)	200 x 500	2 to 8[2]	2¾ to 3¼
A	Barge Slip/Mooring and Crushed Stone Area (BS/CS)	650 x 290	0 to 8[3]	¾ to 1
C	Power Plant (PP)	45 x 45	7[2]	¾ to 1¼
C	Guardhouse/ Administration (GH)	85 x 65	6[2]	1¼ to 1¾
C	Water Tower (WT)	120 x 70	7[2]	1½ to 2
C	Scale/Guardhouse (SG)	120 x 160	7[2]	2 to 2½

[1]From furnished Drawing No. S-1, "Port Bienville Dock Plan," prepared by Compton.
[2]From furnished wetland contour map; assumed finished grade at el 14.
[3]From furnished wetland contour map; assumed finished grade at el 11.

60. The corners and edges of individual filled areas are anticipated to settle approximately 25% and 50%, respectively, of the maximum settlement near the center of the filled area. However, edges and corners will experience more settlement when adjacent fill areas are present. Eustis Engineering should be provided an opportunity to review grading plans once they are developed to verify the assumptions used in our settlement models. Should the amount of fill (height or area) vary significantly from our assumption, Eustis Engineering should be contacted to reevaluate settlement due to fill placement.

61. Fill placement will result in differential settlement between grade supported structures, such as pavements, and pile supported structures. Your design should recognize this potential. This includes the use of joints and/or hinged connections at transitions between pile supported structures and pavements. Fill placement will also affect pile foundations as discussed subsequently in the "Deep Foundations" section of this report. These estimates of settlement are due only to new fill placement and do not consider settlement due to existing fill, improper site preparation and drainage, poor fill compaction or quality, and non-uniform foundation conditions. Additional settlement will be expected for larger fill areas or greater fill heights.

62. New fill placement has the potential to impact stability of the existing keyway slip and the bank stability of Mullato Bayou without bulkhead walls. Evaluation of these features were not part of our current scope. However, prior to finalizing construction plans, the final grading plan should be reviewed by Eustis Engineering to determine if fill placement may impact these features. Note, dredging of the existing slip or bayou also has the potential to impact stability in nearby areas that receive new fill. Such activities can affect stability of the slip and bayou bank even without new fill placement or surface loads.

63. <u>Areal Subsidence.</u> Areal subsidence is a result of past filling of a site and lowering of the ground water level over large areas. Areal subsidence is considered a background condition over which people have no control. Sufficient information is not available in this geotechnical exploration to make accurate estimates of areal subsidence in the project area. Ongoing areal subsidence can result in differential settlement of grade supported structures and pavements. Additional settlement of pile founded structures may also be observed. Areal subsidence can also result in differential settlement magnitudes between grade supported and pile supported structures that are larger than what would be estimated by consolidation settlement theory.

Bulkhead Recommendations

64. The new bulkhead walls along the side of the new keyway slip and the bank of the bayou facing the barge slip/mooring and crushed stone area may be constructed as an anchored sheetpile wall system provided the bulkhead sheetpiles are installed to the recommended tip embedments, anchorage systems are designed to resist the estimated tieback forces, the anticipated deflections from structural loads can be tolerated, and the sheetpiles are designed to resist the estimated maximum bending moments presented. Anchorage may be provided by pile supported deadman anchors incorporating vertical and battered piles to resist wall anchor force loads. Pile supported deadman anchors should be positioned a minimum setback distance of 50 feet behind the bulkhead wall so anchorage piles will not intersect with the estimated failure zone behind the bulkhead walls, as discussed subsequently. A schematic of the proposed bulkhead system is provided on Figure 3.

65. General. Review of the boring logs for Profile A (Design Reach BS/CS) indicates the proposed bulkhead sheetpiles for the new barge slip will penetrate medium dense to dense sands. Penetration of sheetpile sections through these deposits will necessitate use of steel sheetpile sections.

66. Steel Sheetpiles. We recommend steel sheetpiles meet the material requirements outlined in Section 719.05 of the MSSRBC. Installation of the sheetpiles should conform to Section 803 of the MSSRBC.

67. Drainage Wedge. We recommend a drainage wedge be provided behind the new bulkhead. Recommended details of positioning and the shape of the drainage wedge are presented on Figure 3. A crushed stone drainage media should be used behind the new bulkhead

68. <u>Crushed Stone Aggregate Fill.</u> Crushed stone should be used to create the drainage wedge behind the proposed bulkhead and should conform to the requirements of Section 704.02 of the MSSRBC for a Type A filter material. The crushed stone should be placed in lifts of 10 to 12 inches loose measure and be compacted to 75% of the relative density determined in accordance with ASTM D4253 and D4254.

69. <u>Weep Holes.</u> Weep holes are necessary to provide adequate drainage and pressure relief of the drainage wedge. Weep holes should be provided in the bulkhead wall to provide for pressure relief during rapid drawdown events. We recommend at least one row of 4-in. diameter weep holes be provided and positioned no higher than el 3.5. These weep holes should be spaced 8 feet center-to-center horizontally. If grading is such that the stone surface course and/or pile supported relieving platform will also direct drainage to the wall, at least one additional row of weep holes should be provided. The diameter and spacing of additional weep holes should be sized by the civil engineer in consideration of drainage requirements. A screen should be placed over the weep hole openings to retain the drainage wedge. This screen may be a galvanized steel or copper mesh or other suitable weather and UV resistant material. The screen should have an opening size no greater than D_{85} of the material retained.

70. <u>Geotextile Filter.</u> A geotextile filter should be placed between the drainage wedge and existing subgrade to prevent clogging of the aggregate with fine materials. We also recommend the top of the drainage wedge be covered with a separator fabric if the surface course will be a different gradation. The geotextile filter should be a non-woven fabric meeting or exceeding the material requirements for Type V geotextiles as presented in Section 714.13 of the MSSRBC. The fabric should be placed directly on the subgrade in accordance with the manufacturer's construction recommendations.

71. <u>Bulkhead Design Methodology and Assumptions.</u> Engineering analyses were performed based on information furnished by Compton and our understanding of the project requirements utilizing data collected from the vicinity of the proposed barge slip. Local and global stability failure mechanisms were evaluated. Eustis Engineering utilized the USACE's program CWALSHT for evaluating the local stability of the bulkhead wall. Global stability analyses were performed using the USACE's Lower Mississippi Valley Division's Method of Planes (LMVD MOP) stability analysis methods.

72. For local stability and deep-seated global slope stability analyses, we considered a top of wall at el 11, a mudline at el -16, and a water level at el 0 (NAVD 88) for the proposed bulkhead. We positioned the bulkhead anchor at el 4 on the bulkhead based on a drawing furnished by Compton. A 20-ft wide uniform surcharge load of 200 psf on the retained side of the bulkhead was modeled in our analyses to account for temporary loads during construction. We understand post-construction surcharge loads will be supported by pile supported relieving platforms extending at least 40 feet behind the walls. If our assumptions are invalid, Eustis Engineering should be contacted to reevaluate our recommendations.

73. <u>Local Stability Evaluation.</u> Based on the furnished design considerations and documented assumptions, our analyses indicate the anchored sheetpile walls should be installed to a minimum tip embedment of el -39 (approximately 50-ft long sheetpiles) ***to maintain local stability with an adequate factor of safety.*** This tip embedment was determined using a factor of safety of 1.3 applied to the soil shear strengths. Anchorage systems installed at el 4 should be designed for an anchor/tieback load of 12.8 kips per linear foot of wall. We estimate the maximum moment in the bulkhead sheetpiles will be 82.2 ft-kips/ft, and the maximum scaled deflection for these sheetpiles will be 1.3×10^{10} $lbs\text{-}in.^3$ for these design assumptions. Maximum moment, anchor forces, and scaled deflections were

determined using a factor of safety of 1.0. Adequate factors of safety should be applied to structural components by the structural engineer. To obtain deflection in inches, divide the maximum scaled deflection by the product of the modulus of elasticity of the sheetpiles in psi and the sheetpile moment of inertia in $in.^4$. Additional analyses may be required after designs are finalized for conditions not foreseen in this report , or to analyze wall loading conditions impacted by construction sequence or heavier surcharge loading requirements.

74. Tieback Anchorage Setback Distance. We recommend pile supported deadman anchor systems be designed with a minimum setback distance of 52 feet behind the bulkhead walls. Recommendations for allowable pile load capacities for vertical and battered tieback anchor systems are provided subsequently in this report. If the minimum setback distances described for pile supported deadman anchors cannot be implemented, pile capacities must be reduced to account for frictional resistances along the piles that intersect with the failure zone behind the bulkhead wall.

75. Global Stability. We have evaluated global stability for a new bulkhead installed with a top of wall at el 11, a low water level at el 0, a mudline at el -16, and a bulkhead sheetpile tip at el -39. We have also included a temporary uniform surcharge of 200 psf applied over a distance of 20 feet at the surface behind the wall to model construction loads. Global stability analyses were performed using the USACE's LMVD MOP stability analysis methods. The results of these analyses indicate the proposed bulkhead with a slip base dredged to or maintained at el -16 can be made with a minimum factor of safety in slope stability of 1.5. This factor of safety assumes support from the sheetpile will result in global stability failure surfaces extending around a sheetpile wall system, designed to provide local stability support with an adequate factor of safety, installed to a tip embedment of at least el -39. A minimum acceptable factor of safety equal to 1.3 was

assumed using the USACE's LMVD MOP stability analysis methods. Thus, the bulkhead wall tip embedment is governed by local stability requirements. If our assumptions regarding dredging of Mulatto Bayou and the new slip to el -16, or our estimates of construction loading are invalid, Eustis Engineering should be contacted to reevaluate our recommendations. As indicated previously, we have not evaluated global or local stability of slopes extending away from the bulkhead at either the existing or new barge slips. These analyses should be performed once additional information is available including grading plans, cross-sections, and operational loading.

76. <u>Global Stability - Barge Slip/Mooring and Crushed Stone Area.</u> The western portion of the site, generally designated as the barge slip/mooring and crushed stone area, is adjacent to Mulatto Bayou and an existing keyway slip. Based on furnished drawings, a new bulkhead will be constructed on Mulatto Bayou along the length of the property between the existing and new keyway slips. We understand the existing slip does not have a bulkhead but is sloped and armored. Placement of new fill adjacent to the bayou prior to construction of the bulkhead system may impact stability of the bayou bank. Allowing a 1,500-psf bulk storage surcharge in the vicinity of these slopes may also pose stability problems. Evaluation of stability of the existing slip or slopes for the new slip beyond the bulkhead is beyond the scope of this exploration. Eustis Engineering is available to perform these services upon request.

77. <u>Trenching and Excavations.</u> In accordance with OSHA Standard No. 1926, Subpart P, Appendix A, the fill material and subsoils encountered may be classified as Type C. This classification should be used to ensure all excavations and trenching operations to construct anchorage systems or to install a drainage wedge behind the bulkhead comply with OSHA requirements. Vertical trenches and excavations for tie rod and anchorage installation to depths of 4 feet or less may be open cut. OSHA requires temporary open

cut slopes no steeper than 1 vertical on 1.5 horizontal (1V:1.5H) for Type C soils. Open cut excavation should be accomplished in a controlled manner. Temporary sheeting and bracing can also act to prevent migration of fine grain soils and water into the excavations. Strut loads, sheeting material properties, and sheeting penetration for temporary retaining structures (TRS) will depend on the excavation depth, width, duration, construction techniques, and bracing system.

78. The construction contractor should have the responsibility for adequacy of temporary sheeting, bracing, and shoring systems and the design of these systems should be made by a registered professional engineer. The construction contractor's engineer should make an independent interpretation of the subsoil conditions encountered at the boring and CPT locations from the information included in Appendices I and II of this report. The design should be submitted to the owner for review of adequacy and to evaluate the design's impact on adjacent structures. The CPTs , boring logs, and laboratory test results contained in Appendices I and II may be used in determining the structural requirements for the sheeting.

79. <u>Dewatering.</u> Based on the logs of the borings in the vicinity of the barge slip (Profile A, BS/CS), excavations of approximately 4 to 5 feet for construction will penetrate primarily low permeability cohesive soils, although some cohesionless or semi cohesive materials may be present. The use of sumps and pumps should be adequate to dewater excavations in these deposits. Deeper excavations may penetrate cohesionless soils that may require well points to dewater. Maintaining a dry excavation for an extended period of time may result in subsidence of adjacent areas. Therefore, the contractor's plan should minimize the period of time when an excavation must be maintained in a dry condition. The magnitude of settlement due to prolonged pumping cannot be estimated without

information regarding the pumping duration, excavation configurations, and foundation types of surrounding structures.

Railroad Track Roadbeds

80. Recommendations were requested for components and thicknesses for new railroad track roadbeds utilizing the soil borings from Profile G. Our analyses were based on the loading intensity exerted by a Cooper E-80 locomotive (approximately 320 kips) and not the lighter railcars. Using this loading intensity, analyses were made to determine the recommended components and thicknesses for railroad track roadbeds installed at a depth so that the rails will be generally congruent with the existing ground surface and rail system. The results of these analyses indicate the railroad track roadbed should consist of at least 8 inches of ballast beneath the cross-ties, underlain by a minimum of 30 inches of subballast, and followed by at least 12 inches of sand subbase. A geotextile fabric should be placed between the natural subgrade and sand subbase and between the subbase and subballast.

81. Should finished grade not allow for the section height recommended in the preceding paragraph, the subballast thickness may be reduced by the use of geogrids placed within this component. Our analyses indicate the railroad track roadbed should consist of at least 8 inches of ballast beneath the cross-ties, underlain by a minimum of 24 inches of subballast. A layer of geogrid should be placed at the midpoint of the subballast. This should be followed by at least 12 inches of sand subbase. A geotextile fabric should also be placed between the natural subgrade and sand subbase and between the subbase and subballast.

82. Any ballast, subballast, or fill extending above existing grade should have side slopes no steeper than 1V:3H. The recommended sections will provide a factor of safety of at least 3 with regard to a shear failure of the soil.

83. <u>Subbase.</u> The sand subbase should be placed directly on the geotextile fabric and compacted to 95% of its maximum dry density within ±2% of optimum water content in accordance with ASTM D1557. The sand subbase should be placed in loose lift thicknesses of 6 to 8 inches. The full depth of sand subbase for the track roadbed should extend at least 4 feet beyond the end of the cross-ties. The subbase should meet the specifications given in this report for structural sand fill. Clayey sand fill should not be used as subbase.

84. <u>Geotextile Fabric.</u> After the subgrade excavation base has been prepared as previously recommended in this report, a geotextile ground stabilization fabric should be provided for material separation between the prepared subgrade/excavation base and the structural sand subbase. Another layer of geotextile fabric should be provided between the subbase and subballast. The geotextile filter should be a non-woven fabric meeting or exceeding the material requirements for Type V geotextiles as presented in Section 714.13.11 of the MSSRBC. Subsequent to clearing and stripping, the fabric should be placed directly on the prepared subgrade or compacted fill in accordance with the manufacturer's recommendations.

85. <u>Geogrid Reinforcement.</u> Geogrid reinforcement may be considered to provide rigidity to the stone mass and to aid in distribution of the rail loads. Geogrid reinforcement should meet the material and strength characteristics of a Tensar BX1100 or equivalent. The biaxial geogrid should be placed on the surface of the first 6-in. lift of the rail ballast stone. Geogrids should also meet the requirements described in Section 714.15 of the MSSRBC.

86. Subballast. We recommend the subballast consist of crushed rock having the gradation shown in Table 8.

TABLE 8: SUBBALLAST GRADATION

SIEVE SIZE	PERCENT PASSING
1½ Inches	100
¾ Inch	70 to 100
⅜ Inch	50 to 80
No. 4	35 to 65
No. 10	25 to 50
No. 40	10 to 25
No. 200	0 to 8

87. The liquid limit of the fraction of material passing the No. 40 sieve should not exceed 25 with a plasticity index no greater than 4. The crushed rock should be compacted to at least 100% of its maximum dry density at optimum water content in accordance with ASTM D1557, or 75% of the relative density of the material determined in accordance with ASTM D4253 and D4254.

88. Ballast. The ballast beneath the cross-ties may consist of graded crushed rocks smaller than 2 inches and larger than ⅜ inch. The ballast beneath the cross-ties should be compacted to the same criteria as the subballast. Additional ballast should be placed between the cross-ties and at least 6 inches beyond the end of the cross-ties.

89. Quality Control. Density tests should be performed on each lift of the compacted structural fill, subballast, and ballast to determine if the contractor has achieved the recommended density. All clearing, proofrolling, filling, and compaction operations should only be accomplished during periods of dry weather. The contractor should

exercise caution during and after inclement weather to ensure subsoil support is not degraded by construction operations.

90. Estimated Settlement. Settlement of the subgrade soils may be attributed to two sources, elastic settlement and consolidation settlement. Elastic settlement is a short term settlement that will occur immediately upon loading, and consolidation settlement is a long term settlement associated with the expulsion of pore water from the foundation soils. Due to the anticipated temporary nature of the rail loading, we estimate negligible amounts of settlement will occur from consolidation settlement. We estimate elastic settlements beneath the rail line may be ¼ to ½ inch due to the short term heavy rail loads. These short term estimates of settlement are time sensitive and assume bearing intensities due to the rail load will last less than three days for any portion of the new track.

Shallow Foundations

91. General. Review of the boring and CPT logs for the warehouse and staging area (Profile D, Design Reach WS) and for the guardhouse/administration building and power plant (Profile C, Design Reach PP,SG,GH,WT) indicates shallow foundations for these structures will generally bear on medium stiff to stiff clay, silty clay, and sandy clay; although, shallow deposits of sands and silts were encountered in some of the borings and CPTs in Design Reach PP, SG, GH, WT.

92. Floor Slab. A minimum 12-in. thick pad of structural fill should be placed beneath all grade bearing floor slabs to provide uniform support for the slab. A Visqueen® vapor barrier should be provided beneath grade bearing floor slabs and pile supported floor slabs for the structures to prevent capillary migration of moisture. Slabs should be cast

monolithically with grade beams and footings or pile caps to provide rigidity and minimize the potential for differential settlement. The slabs, including the reinforcing and anchor connection details, should be designed by a structural engineer.

93. Estimated Settlement of Floor Slabs. Analyses were made to determine the estimated settlement near the center of grade bearing floor slabs for the proposed guardhouse/administration building (GH) and the power plant (PP) using the soil conditions defined as Profile C (Design Reach PP, SG, GH, WT). Analyses for the warehouse/staging building (WS) were performed using soil conditions defined as Profile D (Design Reach WS). Assuming a uniform pressure intensity of 100 psf from a 6-in. thick grade bearing concrete slab and an assumed sustained live load of 25 psf for the proposed guardhouse/administration building, the results of the analyses indicate settlement near the center of a grade supported slab will be less than ¼ inch. Assuming a uniform pressure intensity of 375 psf from a 6-in. thick grade bearing concrete slab and an assumed separate uniform sustained live load of 300 psf for the proposed power plant and warehouse/staging building, the results of the analyses indicate settlement near the center of these grade supported slabs will be ¼ to ½ inch. Plan dimensions for the GH, PP, and WS slabs used in our analyses are 85’ x 65’, 45’ x 45’, and 230’ x 250’, respectively.

94. Settlement at the corners and midpoint of the sides of grade bearing slabs are estimated to be one-quarter and one-half of these settlement values, respectively. ***Settlement of the slabs should be considered additive of that estimated for footings supporting separate footing loads and settlement estimated for fill.*** The settlement estimates assume the loading intensity is applied by a flexible foundation. As the rigidity of the slab increases, settlement will be more uniform with less settlement at the center and greater settlement at the sides and corners. Uniform settlement of a rigid slab may be estimated as 85% of the center settlement of a flexible slab. If any of our assumptions are not valid,

Eustis Engineering should be contacted to reevaluate our estimates of settlement for grade supported slabs. Note, there is a greater potential for differential settlement across grade bearing slabs where there is a high variation of fill placement beneath the structure.

95. Depth of Footings. We recommend continuous strip footing and isolated square footing foundations for the proposed warehouse/staging building, power plant, and/or guardhouse/administration building be placed to bear at least 2 feet below finished grade on firm undisturbed soil or compacted structural fill. Precautions should be exercised so excavations for footings do not become wet prior to pouring concrete. Foundations should be poured immediately after the completion of the excavations. All footing excavations should be carefully inspected by qualified personnel to verify footings will be placed to bear at the recommended depth, and the excavation is in a dry condition prior to pouring concrete. Eustis Engineering should be retained to observe the condition within footing excavations prior to concrete placement.

96. Allowable Soil Bearing Values. Analyses have been made to estimate the net allowable soil bearing values for continuous strip footing and isolated square footing foundations for the guardhouse and power plant (Profile C, Design Reach PP,SG,GH,WT) and for the warehouse/storage area (Profile D, Design Reach WS.) The results of our analyses are presented in Table 9.

TABLE 9 : ESTIMATES OF ALLOWABLE SOIL BEARING VALUES

FEATURE	ALLOWABLE SOIL BEARING VALUE IN PSF	
	STRIP FOOTING (MAXIMUM WIDTH = 3 FEET)	SPREAD FOOTING (MAXIMUM WIDTH = 5 FEET)
Guardhouse (GH) and Power Plant (PP)	1,350	1,650
Warehouse/Storage Area (WS)	1,250	1,350

97. These allowable soil bearing values contain an estimated factor of safety of 3 against a soil shear failure. Eustis Engineering should be contacted if larger footings will be incorporated into the final foundation design for the proposed warehouse/staging building, power plant, and guardhouse/administration building or if footings are proposed for other areas of the site.

98. <u>Estimated Settlement of Footings.</u> Assuming a long term dead load pressure intensity equal to 80% of the allowable soil bearing values, estimates of settlement were made for continuous strip footing and isolated square footing foundations. We estimate the settlement, due to sustained structural loads for both a continuous grade beam footing having a width of 1 to 3 feet or an isolated square footing having a maximum side width of 5 feet for these structures with Profiles C and D in Soil Design Reaches PP, SG, GW, WT, and WS, will be ¼ to ½ inch.

99. Our estimates of settlement assume the center-to-center spacing between continuous strip footings is not less than three times the footing width or twice the largest side dimension for spread footings. We have also assumed the site has been prepared and drained as recommended in this report, and the foundation soils are not degraded or exposed to excess moisture prior to placing concrete for the footings. To decrease the potential of differential movements, concrete for footings should be placed integrally with the slab and grade beams. If any of our assumptions are not met, Eustis Engineering should be notified to reevaluate potential settlement. ***These settlement estimates should be considered additive to the settlement estimated for a slab supporting a separate uniform dead load and for the placement of fill at the site.***

Deep Foundations

100. Estimated Vertical Pile Load Capacities. Analyses have been made to determine the estimated allowable single load capacities for treated ASTM D25 quality timber piles; square, precast concrete piles; augercast piles; and open end steel pipe piles used to support the project features. The results of these analyses our shown in Figures 4 through 7. Our analyses considered variable subsurface conditions for different project features/areas of the site. Capacities and pile cutoffs for the land based features are assumed based on the estimated average ground surface elevations presented in Table 5. In addition, our analyses considered different pile cutoff/mudline conditions other than the average ground surface for piles used for the proposed bulkhead anchorage system and for mooring dolphin piles. Analyses presented in Figures 4 through 7 for piles installed from the ground surface neglect the capacity within the upper 2 feet below the ground surface for embedment within a pile cap and assume pile installation is vertical. This adjustment also applies for anchorage piles assumed to be installed within a deadman anchor with a pile cutoff between el 4 and el 6.5. Table 10 provides a summary of the figures.

TABLE 10: SUMMARY OF ALLOWABLE LOAD CAPACITY FIGURES

DEEP FOUNDATION TYPE	PROJECT FEATURE	PROFILE (DESIGN REACH)	FIGURE NO. (SHEET NO.)
Timber	Water Tower, Scale, Guardhouse/Administration, Power Plant	C (PP, SG, GH, WT)	4 (1 of 3)
	Warehouse/Manufacturing	B (WM)	4 (2 of 3)
	Warehouse/Staging	D (WS)	4 (3 of3)
Square, Prestressed, Precast Concrete	Water Tower, Scale, Guardhouse/Administration, Power Plant	C (PP, SG, GH, WT)	5 (1 of 5)
	Warehouse/Manufacturing	B (WM)	5 (2 of 5)
	Warehouse/Staging	D (WS)	5 (3 of 5)
	Tank Farm	E and F (TF-East and TF-West)	5 (4 of 5)
	Barge Slip/Crushed Stone Area (Apron, Bulkhead Anchorage)	A (BS/CS)	5 (5 of 5)
Augercast Concrete Pile	Water Tower, Scale, Guardhouse/Administration, Power Plant	C (PP, SG, GH, WT)	6 (1 of 5)
	Warehouse/Manufacturing	B (WM)	6 (2 of 5)
	Warehouse/Staging	D (WS)	6 (3 of 5)
	Tank Farm	E and F (TF-East and TF-West)	6 (4 of 5)
	Barge Slip/Mooring and Crushed Stone Area (Apron)	A (BS/CS)	6 (5 of 5)
Open End Steel Pipe Pile	Water Tower, Scale, Guardhouse/Administration, Power Plant	C (PP, SG, GH, WT)	7 (1 of 6)
	Warehouse/Manufacturing	B (WM)	7 (2 of 6)
	Warehouse/Staging	D (WS)	7 (3 of 6)
	Tank Farm	E and F (TF-East and TF-West)	7 (4 of 6)
	Barge Slip/Mooring and Crushed Stone Area (Apron, Bulkhead Anchorage)	A (BS/CS)	7 (5 of 6)
	Barge Slip/Mooring and Crushed Stone Area (Mooring Structures)	A (BS/CS)	6 (6 of 6)

101. A one-third increase in allowable tension load capacity can be used for temporary loads as allowed by applicable codes and appropriate testing. A test pile program with static load testing is required to verify our estimates of allowable load capacities if a factor of safety of 2 is utilized for design. As an alternative, dynamic pile testing may be performed on driven piles to confirm the estimated load capacities for driven piles and a factor of safety of 2.5 should be used. A factor of safety of 3 should be implemented when verification testing is not planned (e.g., for non-governing loading condition such as tension capacity).

102. Timber Piles. We recommend the treatment of timber piles meet the current American Wood Preservers Association Standards as outlined in Section 719.02 of the MSSRBC for both preservative and quality assurance. Treatment should also follow Section 719.02.2, where applicable. Furthermore, we recommend the timber piles meet the quality (clean peeled, straightness, etc.) and size requirements outlined in ASTM D25. The pile dimensions assumed in our analyses are provided in Figure 4, Sheets 1 through 3.

103. Square, Prestressed, Precast Concrete Piles. We recommend the precast concrete piles meet the requirements outlined in Section 719.03 of the MSSRBC. Precast concrete piles should be designed to have a strand prestress that is structurally sufficient to facilitate handling and driving the piles without damage. Our analyses are based on a solid, square pile. Alternate precast pile configurations (pipe, triangle, etc.) should be further evaluated. The estimated capacities shown in Figure 5, Sheets 1 through 5, do not include the weight of the pile.

104. Augercast Piles. The estimated capacities shown in Figure 6, Sheets 1 through 5, for augercast piles assume the actual grout volume placed is at least 130% of the theoretical volume of the pile and standard installation methods will be utilized. Standard installation

methods are defined here as the use of a single continuous flight, hollow shaft auger to produce the pile such that the spoils are removed. Should positive displacement augercast piles (those where the drilled materials remain in the hole) be selected for the project, additional analyses should be performed to estimate the allowable pile capacities for the reduced overtake volumes anticipated. Additionally, the embedment depths of positive displacement piles in dense to very dense sands will be limited. The estimated capacities shown in Figure 6, Sheets 1 through 5, do not include the weight of the pile. We do not recommend augercast piles be designed or installed with a batter.

105. <u>Steel Piles.</u> We recommend the open end steel pipe piles meet the requirements outlined in Section 719.04 of the MSSRBC. The steel piles should be designed to have wall thicknesses that are structurally sufficient to withstand handling and driving stresses. The pile dimensions assumed in our analyses for the pipe piles shown in Figure 7, Sheets 1 through 6, are based primarily on the outside pile diameter. The actual wall thickness selected for the open end pile will generally not affect our estimated vertical capacities. However, if a closed end pile is considered, Eustis Engineering should evaluate the effect on the capacity and driveability. The estimated capacities shown in Figure 7 do not include the weight of the pile. As noted previously, we evaluated various mudline/cutoff elevations for these piles. Eustis Engineering should be contacted if alternative configurations are needed.

106. <u>Protrusion of Connections or Welds.</u> Pile connections that protrude beyond the surface of the outside wall of the pile reduce the frictional resistance acting on the pile surface above the protrusion. For this reason, your specifications should require all welds be machined and/or ground such that no more than a ⅛-in. protrusion is present. This includes machined bead thicknesses for spiral weld piles and manual full penetration segmented pile section welds as well as splice collars. If the welds or splice collars

protrude past the pile's outside dimensions, the soil-pile adhesion is disturbed during installation of the pile and the pile's capacity may be reduced. Likewise, precast concrete pile splices should be contained within the pile section.

107. Pile Weight. The allowable load capacities presented in this report do not include the weight of the pile. When computing the resultant net compressive capacity of square, precast concrete piles or augercast piles, a net load of 30 pcf may be used for the weight of the concrete (weight of the concrete minus the weight of soil removed). The full buoyant weight of the concrete of 87 pcf may be used to determine the tensile capacity of the pile provided they are designed to transmit these loads. Similar procedures may be used to evaluate the net capacity of timber and steel piles.

108. Structural Capacity. Analyses for pile capacities are based only on a soil-pile relationship. The structural capacity of individual piles and their connections to transmit these loads, and any connections with the piles and the structures, especially in tension, should be determined by a structural engineer.

109. Soil Response of Single Piles to Lateral Loads. We have performed geotechnical analyses to evaluate the effects of lateral loads on individual 24-in. diameter open end steel pipe piles installed vertically for the mooring structure within the existing keyway slip. Our analyses were performed using the computer program LPILE, Version 2016-09.003, by Ensoft, Inc. Analyses were performed assuming free pile head conditions. A pile head that is free to rotate is sometimes referred to as a pinned condition. This model provides restraints against translational movements but none against rotational movements.

110. Lateral Load Analyses - Design Assumptions. For these analyses, we have assumed cyclic loading conditions apply. Our analyses applied a lateral load acting at the top of the pile

at el 4 for the furnished height of 20 feet above the mudline (el -16), with a vertical compressive load of 80 kips, and an estimated pile tip embedment depth of 50 feet below the existing mudline. The vertical loads are based on our estimated allowable pile capacities using a factor of safety of 3. A wall thickness of ⅜ inch and an elastic modulus of 29,000 ksi were also utilized. Initial analyses for the furnished lateral load of 50 tons essentially failed the pile due to excess deflection. We subsequently ran analyses to determine the lateral loads that would result in deflections of 2, 4, 6, and 8 inches. The results are summarized in Table 11. Plots of bending moment and deflection for these analyses are provided in Appendix VI.

TABLE 11: RESULTS OF SOIL-PILE INTERACTION ANALYSES FOR
SINGLE 24-IN. DIAMETER OPEN END STEEL PIPE PILES

DEFLECTION IN INCHES	CORRESPONDING ESTIMATED LATERAL LOAD IN KIPS	ESTIMATED MAXIMUM BENDING MOMENT IN KIPS-IN.	ESTIMATED DEPTH OF MAXIMUM BENDING MOMENT BELOW PILE HEAD IN FEET
2	5.7	1,752	25.2
4	10.0	3,187	25.9
6	13.9	4,545	26.6
8	17.6	5,859	27.3

111. The analyses presented in Table 11 are for laterally loaded vertical, single piles or piles installed in groups widely spaced relative to the direction of lateral load. Limitations to our results are necessary because we referenced unfactored soil parameters (friction angles, undrained shear strengths, and lateral strain factors) in our analyses. Appropriate factors of safety should be applied by the structural engineer. If other loading conditions or complex group configurations, including batter piles, are planned, or if the assumptions made for these analyses are invalid, Eustis Engineering should be consulted to provide additional lateral load analyses. The analytical tools used for analyses of laterally loaded

piles are intended to predict the natural soil response to laterally loaded piles and are not to be used independently for structural analyses of the piles.

112. Reduction Factors for Group Effects. The LPILE program does not account for the effects of interactions between piles that are placed in groups. Group effects (sometimes referred to as shadowing) will reduce lateral pile capacity if piles are spaced closer than six diameters center-to-center in the direction of loading or closer than five diameters perpendicular to the direction of loading. For pile group spacings closer than six pile diameters, reduction factors should be applied to the lateral loads. These reduction factors are known as p-multipliers and are provided in Table 12. For pile spacings greater than six diameters, a reduction factor is not necessary. The p-multipliers from Table 12 should be multiplied together for a given pile layout in a group where both parallel and perpendicular spacings are less than indicated. Linear interpolation should be used for intermediate spacings. If spacings parallel and/or perpendicular to the loading are less than three diameters, Eustis Engineering should be contacted for additional recommendations regarding both lateral and vertical loading impacts.

TABLE 12: REDUCTION FACTORS FOR PILE/SHAFT GROUPS SUBJECTED TO LATERAL LOADS

PILE/SHAFT SPACING		P-MULTIPLIERS (REDUCTION FACTORS FOR PILES)		
		ROW 1	ROW 2	ROW 3 OR HIGHER
In Direction of Lateral Loading	>6B	1.00	1.00	1.00
	6B	1.00	0.93	0.93
	5B	1.00	0.85	0.70
	3B	0.80	0.40	0.30
Perpendicular to Direction of Lateral Loading	5B	1.00		
	3B	0.80		

113. Complex loading conditions or irregular pile group layouts should be evaluated with additional analyses using GROUP, Version 8.0 or newer, by Ensoft, once pile configurations and loadings are finalized. Eustis Engineering is available to further evaluate lateral single pile and pile group response to lateral loads upon request.

114. Batter Piles. The estimated pile capacities provided in Figures 4, 5, and 7 are for piles installed vertically and may be used to determine the load capacities for batter piles. ***We do not recommend augercast piles (Figure 6) be installed on a batter.*** The vertical component of a batter pile will be equal to the capacity for a vertical pile installed to the same tip elevation. From this relationship, geometry may be used to determine the axial capacity and horizontal component of the batter piles. This geometric relationship is shown on Figure 8.

115. Vertical Pile Group Capacity. For driven piles firmly embedded within the sand deposits, shown on the subsurface profiles presented In Figure 2, the majority of their supporting capacity will be derived from end bearing and no reduction in vertical group capacity will be required when evaluating compressive capacities. For piles not embedded in the sand deposits as estimated In Figure 2, augercast piles, or any piles subjected to tension loads, the piles will derive the majority of their supporting capacity through skin friction. Therefore, it is necessary to consider the effect of group action. In this regard, the supporting value of the friction piles driven in groups should be investigated on the basis of group perimeter shear by the formula shown on Figure 9. When evaluating the tensile capacity of pile groups, the second term of the formula should be neglected.

116. Pile Spacing. The minimum center-to-center pile spacing within a row or group of timber piles should be at least 3 feet. The minimum spacing between individual open end steel pipe piles, precast concrete piles, or augercast piles should be determined by the formula

given on Figure 9. The minimum spacing between rows of piles should also meet the requirements discussed in the “Estimated Settlement due to Structural Loads” section of this report. Pile spacing requirements may not minimize influence of individual piles on each other with respect to lateral load resistance, settlement, or vertical group capacity.

117. Estimated Settlement due to Structural Loads. We estimate pile sizes and embedments presented in this report will settle ¼ to ½ inch due to subsoil consolidation under the influence of sustained structural loads. The settlement estimates do not include elastic deformation of the piles which should be added to the settlement estimates. Elastic deformation of friction piles may be estimated as 67% to 75% of the static column strain of a pile acting as a column. Elastic deformation of end bearing piles may be estimated as 100% of the static column strain of a pile acting as a column.

118. Our estimates of settlement assume piles will be installed in small groups or widely spaced rows. We have assumed the largest group dimension will be no greater than 20% of the pile length, and the center-to-center spacing between groups will be no closer than twice the largest group dimension. We have assumed the center-to-center spacing between rows or single piles will be no closer than 8 feet, or nine pile diameters, whichever is greater. In the event any of our assumptions are not met, Eustis Engineering should be contacted to evaluate the potential settlement of the pile foundations.

119. Pile Supported Tanks. We have not been provided tank heights or product unit weights. Thus, we cannot accurately estimate settlement based on the furnished tank layout in the tank farm areas. Depending on the tank base bearing intensity, smaller pile spacing than our recommended pile spacing may be required that will reduce the vertical and lateral support of the pile. Additionally, our estimates of settlement given above are for small pile groups and will be exceeded for a large group supporting a tank. Tank proximity may

also impact settlement of adjacent tanks, even if they are pile supported. Eustis Engineering should be notified once tank base loadings, pile sizes, pile layouts, and tip embedments are finalized to evaluate if our estimates of pile settlement presented are appropriate for the tank farm areas. Note, Eustis Engineering should be contacted to perform additional pile group settlement analyses for any project feature with foundation designs that do not meet our assumptions, or for any other pile group or loading of concern.

120. Effects of Fill Placement on Pile Foundations. As the fill settles from consolidation of the underlying subsoils, negative skin friction (drag loads) is induced on the piles as the soil settles along the piles. These drag loads have the potential to increase settlement of friction piles and increase internal stresses on end bearing piles.

121. We have evaluated pile settlement due to fill placement assuming fill placement to el 14 under the proposed structures and tanks and fill heights derived from the furnished wetland contour map and previously summarized in Table 7. Estimates of settlement of friction piles embedded in clay and silty clay deposits encountered in the borings and CPTs due to the estimated fill placement are presented in Table 13. ***These estimates of settlement should be added to pile settlements due to structural loads.***

TABLE 13: ESTIMATED SETTLEMENT OF PILES DUE TO FILL PLACEMENT

FEATURE	PROFILE (DESIGN REACH)	APPROXIMATE PILE TIP EMBEDMENT ELEVATION (NAVD 88)	ESTIMATED SETTLEMENT DUE TO FILL PLACEMENT IN INCHES
Barge Slip	A (BS/CS)	Below -25	¼ or Less
Guardhouse/ Administration	C (PP, SG, GH, WT)	-20 to -30	½ to ¾
Power Plant		-20 to -30	¼ to ½
Water Tower		-20 to -30	¾ to 1
Scale/Guardhouse		-20 to -30	1¼ to 1½
Warehouse/Staging	D (WS)	-20 to -30	¼ to ½
Warehouse/ Manufacturing	B (WM)	-20 to -30	¼ to ½
West Tank Farm	F (TF-West)	-20 to -40	¾ to 1
East Tank Farm	E (TF-East)	-45 to -50 -60 to -70 Below -70	¾ to 1 ½ to ¾ Less than ¼

122. End bearing piles firmly embedded in the sand deposits as estimated in Figure 2 will have minimal (¼ inch or less) potential for settlement due to fill placement. However, these piles should be evaluated for their structural adequacy. This evaluation should consider an additional compressive load due to the negative skin friction forces on the surface of the piles. The estimated magnitude of this drag load varies with pile type and size. Once pile sizes, types, and depth of embedments have been finalized, Eustis Engineering should be contacted to evaluate drag loads for end bearing piles. ***The drag loads will present as an additional internal stress added to the applied load at the pile butt. The total load should then be used to evaluate the fiber stresses for timber piles using the average pile diameter and internal stresses for precast concrete and steel piles.*** However, the allowable capacities provided in Figures 4 through 7 do not need to be reduced for these loads unless the structural adequacy of the piles are exceeded.

Installation of Driven Piles

123. Quality Control. Close field supervision should be maintained by experienced personnel to ensure proper procedures are followed and accurate records are kept for all pile driving operations. The driving record should include as a minimum the date; pile type; overall length; pile size (tip and butt diameters, side dimensions, or outer diameter and wall thickness); embedment below finished grade or mudline; depth and diameter of predrill; hammer type; and number of blows per foot of penetration. An accurate driving record is especially important to verify the piles are installed to the required tip embedment and to give an indication of any unusual driving characteristics which may indicate pile breakage.

124. Air Hammers. Small treated ASTM D25 quality timber piles, having minimum tip diameters of 6 inches and butt diameters of at least 8 inches, may be driven with a drop hammer or a single acting air hammer having a manufacturer's rated energy of 7,250 ft-lbs per blow. If a drop hammer is used, the ram weight should not exceed 3,000 pounds and the drop should not exceed 3 feet. Larger treated ASTM D25 quality timber piles, having minimum tip diameters of 7 to 8 inches and butt diameters of at least 12 inches, may be driven with a single acting air hammer having a manufacturer's rated energy of 15,000 ft-lbs per blow. Timber piles should be driven no harder than 25 blows per foot with the recommended hammer energies or damage may occur. Additional refusal criteria should be developed during the test pile program.

125. Generally, we recommend a single acting air hammer be used to install the precast concrete piles and steel pipe piles. The manufacturer's rated energy required to drive the piles will depend on the piles' allowable compressive capacity. The estimated hammer energy versus allowable compressive capacity (factor of safety ≈ 2) for piles

recommended in this report is provided in Table 14. For precast concrete piles, we also recommend the ram stroke be limited to 3 feet, and the ram weight be approximately one-half of the pile weight.

TABLE 14: RECOMMENDED PILE HAMMERS

ESTIMATED ALLOWABLE SINGLE PILE LOAD COMPRESSIVE CAPACITY IN TONS FACTOR OF SAFETY ≈ 2	APPROXIMATE RATED SINGLE ACTING HAMMER ENERGY IN FT-LBS PER BLOW
Up to 60	19,500
60 to 120	24,000
120 to 150	32,000

126. <u>Hydraulic or Diesel Hammer.</u> In lieu of an air hammer, a diesel or hydraulic hammer may be used for the installation of the concrete and steel piles. We recommend the diesel hammer have a rated energy of one and one-half times the energy recommended for a comparable installation with a single acting air hammer. Hydraulic hammers should be selected based on an adjusted efficiency to produce similar energy as a corresponding air hammer.

127. <u>Pile Refusal.</u> Refusal criteria for the timber piles may be taken as 25 blows per foot with the recommended hammer driving energies to minimize damage to these piles. Refusal of these piles is likely within loose to dense sands encountered throughout the site as shown in Figure 2. Refusal criteria for the concrete and steel piles should be determined based on the results of the test pile program and dynamic analyses or testing. If the piles are driven with the aid of a follower, or if the pile driving helmet is allowed to impact the ground surface, Eustis Engineering should be consulted to adjust these refusal criteria.

128. <u>Alternate Installation Methods.</u> We do not recommend vibratory methods be utilized for pile installation. If a vibratory hammer is selected for the project, Eustis Engineering

should be contacted to evaluate the reduction in the estimated allowable pile load capacities presented. Also, we do not recommend the use of jetting to aid in the installation of the precast concrete piles. Eustis Engineering should be consulted if these measures are allowed as this will also reduce the estimated capacities presented. If any other alternate installation methods are selected, Eustis Engineering should be contacted to evaluate the effects on our estimates of capacity presented.

129. Predrilling. Predrilling through the surficial fill materials or stiff crust soils may be required at the site. A pilot hole also has the potential of minimizing vibrations resulting from pile driving operations or reducing the potential damage to piles. The predrill bit should be no larger than the tip diameter for timber piles (for shallow predrilling to depths of 6 feet or less) or 75% of the side dimension of precast concrete piles. Predrilling should not be required for open end steel pipe piles. Predrilling through surficial materials may be by dry auger methods. Deeper predrilling should be made by wet rotary methods using a fishtail bit. If deeper predrilling is required for timber piles, the bit diameter should be limited to the tip diameter of the timber piles. Predrilling should extend no deeper than 10 feet above the final tip elevation of the piles. Eustis Engineering should be contacted if a pilot hole deeper than 6 feet is necessary. The use of predrilling should be evaluated and determined, if necessary, during the test pile program.

130. Dynamic Analyses. The precast concrete piles and steel pipe piles should be designed by a structural engineer to have a cross-section which is structurally sufficient to facilitate handling and driving of the piles without damage. Dynamic analyses, or Wave Equation Analysis of Piles (WEAP), can be performed to evaluate driving stresses, pile cushion, and driveability once the hammer and appurtenant equipment have been selected. WEAP is particularly important if a diesel hammer is proposed for installation of precast concrete

piles or if segmented steel piles are selected for the project. Structural requirements can then be verified by a structural engineer and installation criteria can be established.

131. <u>Pile Installation Sequence.</u> Timber piles and square, precast concrete piles are considered displacement piles because soil is displaced laterally when the piles are driven into the subsurface. These piles could cause adjacent piles to refuse at shallower depths or, more likely, could result in heaving of piles that were previously installed. For these reasons, we recommend pile driving take place from the center of large foundations working outward.

<u>Installation of Augercast Piles</u>

132. <u>Quality Control.</u> Successful installation of augercast piles will be a direct function of the quality control used at the site. As a minimum, the quality control should include the checking of the mix, fluidity, temperature, use of additives, and taking of samples for subsequent compressive strength testing. The quality of the grout mix used for the piles should be monitored using ASTM C109 procedures. Fluidity may be checked by using modified USACE Specification CRD C 79 and ASTM C939. Flow rates should be in the range between 10 and 25 seconds using a flow cone modified to have a 0.75-in. opening.

133. Quality control during pile installation should include records for each pile installed. Prior to installation of any grout, the grout pump should be calibrated in units of volume per stroke. During installation, grout pressure and volume should be recorded by flow meter and pressure meter in addition to recording pump strokes. The flow meter should be capable of providing the total and incremental volume of grout pumped. We recommend a maximum increment of 5 feet be implemented for the meters and manual methods. The grout return depth should also be recorded on the installation records and the total

grout volume adjusted based on this return. Finally, the theoretical volume of grout should be compared to this adjusted grout volume for each pile to determine the percent over theoretical volume. Eustis Engineering should be contacted if the volume of grout placed for any 5-ft segment is less than 130% of the theoretical volume. Based on our experience, grout volume overtakes greater than 30% should be anticipated for standard installation methods in soft soils. Deeper grout returns yield higher total grout volumes and subsequent volumetric overtake percentages when based on total grout volume pumped. If the theoretical grout volume is reached after the grout return is observed, additional testing may be recommended.

134. Equipment and Techniques. Augercast piles should be made by rotating a continuous flight hollow shaft auger into the ground to the necessary depth in order to develop the desired load capacity. The auger flighting should be continuous without gaps or breaks and should have a uniform diameter throughout its length. If augercast pile lengths exceed 40 feet, we recommend a middle guide be used. Adjacent piles should not be installed until the piles have set for a sufficient period, as determined by the structural engineer, to withstand earth pressures exerted by the installation process. Normally, a minimum 24-hour set time is recommended.

135. For placement of reinforcing steel, a minimum distance of 3 inches is recommended between the reinforcing bar and outer edge of the piles. If reinforcing cages are used, they should be centered in the bore hole using centralizers or other systems. Cross bracing within the reinforcing cage should not be allowed in order to minimize the potential for void development in the concrete.

136. The grout should be designed to provide an adequate strength to support the anticipated design load. The grout head should be 5 feet higher than the fluid levels in the bore hole

and/or the bottom of the auger at all times. The grout should be placed continuously from bottom to top while the auger rotates during withdrawal.

<u>Integrity Testing for Augercast Piles</u>

137. In order to ensure successful installation of augercast piles, the test piles and a selected percentage of job piles (e.g., 2% to 5%) should be subjected to integrity testing using an instrumented center bar. All piles should have a center bar to further evaluate integrity. For the augercast pile sizes presented in this report, we recommend thermal integrity profile (TIP) testing. If these tests or installation records reveal structural defects or anomalies, additional quality control tests, consisting of single hole sonic logging (SSL) or low strain integrity methods including pile integrity testing (PIT), may be required. The TIP testing should be performed in accordance with ASTM D7949-14 by either the probe method or thermal wire method. The SSL testing should comply with ASTMD 6760-16. PIT testing should be performed in accordance with ASTM D5882-16.

138. The TIP and SSL methods require installation of an access tube (typically PVC) or wires (thermal wire method) and, therefore, must be coordinated prior to construction. We have assumed all augercast piles will have a full length center reinforcing bar that can be used for the testing. These tests must be performed within a specific time frame to maintain their accuracy. The thermal wire method requires cable(s) fitted with uniformly spaced temperature sensors to be installed on the reinforcing cage (or center reinforcing bar) prior to placement into the pile. Temperature data are then collected at selected intervals during the hydration process.

139. For the access tube method for TIP or SSL testing, the access tube(s) should be free of debris and other obstructions, and the outside of the PVC tube(s) should be made coarse

to aid in the bonding with the concrete grout. If SSL testing is planned, immediately upon completion of installing the pile, the PVC tube(s) should be filled with water in order to minimize debonding of the concrete to the surface of the PVC tube(s). Thermal testing (probe method) requires the access tube(s) to be dry during testing. Debonding of concrete to the PVC tube(s) may show false anomalies during SSL testing. Note, PIT is limited to detecting defects to depths of 40 pile diameters and would only be recommended in instances where the other testing is not feasible.

140. We do not recommend forcing the cage or instrumented center bars into place with the use of heavy equipment nor do we recommend twisting the instrumented center bars into place. Should the reinforcing steel not extend to the recommended depth due to anomalies indicative of soil “cave-ins” or grout water loss in cohesionless soils, we recommend the use of a grout intrusion aid admixture such as Intrusion-Aid® MAX by Specrete-IP Incorporated.

Test Piles and Load Tests

141. Eustis Engineering considers a test pile program and load test as an extension of our geotechnical exploration. Therefore, Eustis Engineering should be retained to perform these services. The actual number of test piles should be determined based on the number and type of piles used in the foundations. Eustis Engineering should be consulted to develop a test pile program consistent with the project’s scope. Note, the test pile program will be used to evaluate installation requirements as well as capacity.

142. The test piles should be the same type and embedment anticipated for the job piles and installed with the same equipment and techniques proposed for the job piles. Driven test piles will provide definitive information regarding the anticipated driving resistance,

refusal depths, requirements for predrilling, and vibrations from pile driving. Augercast test piles will provide information relative to the anticipated overtake volumes and can confirm contractor qualifications.

143. Friction test piles should be allowed to set for at least 14 days subsequent to the installation of the reaction system. End bearing test piles should be allowed to set for seven days subsequent to the installation of the reaction system. The structural and geotechnical engineers should select the piles to be used for performance of a static load test(s). The piles should then be load tested to failure in accordance with ASTM D1143 assuming the governing load case is the compressive capacity. A lateral load test or tensile load test may be required depending on the final design. The results should be evaluated by Eustis Engineering to verify the estimated pile load capacities presented in this report.

144. <u>Dynamic Pile Tests.</u> The initial installation of driven precast concrete and steel pipe test piles and job piles may be monitored and evaluated using dynamic pile tests (DPTs) with a Pile Driving Analyzer® (PDA). The performance of DPTs may be used to evaluate actual driving stresses, penetration of resistance, and integrity and capacity of the test piles and/or job piles. The PDA can also monitor energy transferred to the pile from the hammer and evaluate installation efficiency. In order to evaluate pile capacity, a "restrike" DPT should be performed at least 14 days after its initial installation of friction piles or seven days after the initial installation of end bearing piles. Shorter set times may be considered but the ultimate capacity of the pile may not be realized. In addition, an increased hammer energy may be required for the restrike DPT to fully mobilize the pile. Data from this restrike should be further evaluated by CAPWAP® analyses to provide signal matching verification of the estimated capacity. Eustis Engineering is available to perform and evaluate the results of a DPT and CAPWAP analyses. If a PDA is used as the sole

means to evaluate capacity, the factor of safety to establish allowable capacities should be increased from 2 to 2.5 and the capacities shown in Figures 5 and 7 adjusted accordingly.

Pavements

145. Traffic. An aggregate surface pavement is proposed for roads and a parking area at the site. Traffic intensities nor a parking lot layout were provided. Therefore, we have made the following assumptions. We estimate the site will experience fifty passenger cars; fifty pickup trucks, vans, and sports utility vehicles; twenty heavy duty work trucks and twenty 18-wheeler trucks per day; and two garbage trucks per week. The assumed axle weights used in our analyses are provided in Table 15.

TABLE 15: ASSUMED VEHICULAR AXLE WEIGHTS

TYPE OF VEHICLE	AXLE LOAD IN KIPS		
	FRONT	MIDDLE	REAR
Passenger Cars	2(S)	0	2(S)
Pickup Trucks, Vans, or Sport Utility Vehicles	2(S)	0	5(S)
Heavy Duty Work Trucks	12(S)	0	20(S)
Garbage Trucks	20(S)	0	44(T)
18-Wheeler Trucks	12(S)	34(T)	34(T)

(S) = Single; (T) = Tandem

146. Our assumed traffic intensities should be verified prior to implementation of design. If traffic conditions are different than those presented, Eustis Engineering should be contacted to reevaluate the pavement recommendations contained in this report. These traffic data assumptions were converted to equivalent 18-kip single axle loads (E_{18}) using AASHTO equivalency factors for flexible and rigid pavements. A five-year design life was considered for the aggregate surface course pavement.

147. A California Bearing Ratio (CBR) of 3.0 was initially used for the existing, cohesive subgrade in the roadway expansion areas and the railroad embankment area. This represents a worst-case condition based on the available data. A total of seven borings were performed in areas that will either have a new pavement constructed on top of existing unpaved roads or will incorporate new pavement widening the existing unpaved roads. Borings RDB-2, A-1, and A-3 through A-5 were each drilled through existing unpaved roads. Borings RDB-1 and A-2 were each drilled adjacent to existing unpaved roads. DCPTs were performed to the bottom of Borings A-1 through A-5 at depths of 5 feet. The estimated average CBR with depth for these borings are presented in Table 16 based on these data. CBR values were also estimated for Borings RDB-1 and RDB-2 based on soil classifications and natural moisture contents. These results are also presented in Table 16.

TABLE 16: ESTIMATED CBR VALUES FOR THE EXISTING SUBGRADE

BORING	DEPTH IN INCHES	ESTIMATED CBR
RDB-1	0 to 12 12 to 48 48 to 60	3 6 3
RDB-2	0 to 60	10
A-1	0 to 10 10 to 41 41 to 60	10 6 10
A-2	0 to 29 29 to 40 40 to 60	10 7 10
A-3	0 to 10 10 to 36 36 to 60	10 4 9
A-4	0 to 20 20 to 34 34 to 60	10 7 10
A-5	0 to 4 4 to 30 30 to 60	10 7.5 10

148. The subgrade CBR used for analyses should coincide with the estimated CBR value at the base of the pavement section. The results of testing at Borings RDB-1 and A-3 indicate a subgrade CBR value of 3 to 4 is possible if the pavement section is designed to bear concurrently with the existing grade. Note, Boring RDB-1 was positioned in a swale adjacent to an existing unpaved road. The estimated CBR values at this location are indicative of conditions we would anticipate for any roadway constructed where prior roads (or road traffic) have not existed. For roadways constructed on virgin ground, subgrade CBR values can be improved by placement of compacted sand or crushed stone. If at least 6 inches of compacted stone or sand is placed, the CBR value may be increased to 4. If the 12 to 18 inches of compacted stone and/or sand is placed, the CBR value may be increased to 5 and 6, respectively. For areas with existing unpaved roadways, a CBR value of 6 may be used.

149. A pavement design utilizing a CBR value of 3 for the subgrade may generally be utilized across the site without further testing unless weak soils are encountered away from the exploratory locations during site preparation. In this event, our pavement recommendations may need to be revised, or ground improvement may be required to improve the subgrade. In low lying marshy areas where embankment fill is required to raise grade, a CBR of 3 may be assumed for the embankment provided the top 12 inches of embankment comprise structural fill and is compacted to at least 95% of the maximum density per ASTM D1557.

150. Method of Analysis. The components and thicknesses for aggregate pavements were determined using methods presented in the AASHTO Guide for Design of Pavement Structures. The resilient soil modulus (M_r) of the subgrade was estimated based on the type of soil, probable drainage conditions, and engineering experience. For this estimate, we have assumed the subgrade is properly drained during construction and adequate permanent drainage is installed that does not allow the subgrade to become saturated.

151. Aggregate Surface. If the pavement areas are prepared as previously described in this report, we recommend an aggregate surface pavement section consisting of 12 inches of crushed stone aggregate surface material underlain by 16 inches of sand subbase for the assumed traffic intensity. If existing subgrades are verified (or improved) to a CBR value of 4, 5, or 6, the stone surface course may be reduced to 10 inches, 9 inches, or 8 inches, respectively. We recommend a geotextile fabric be used for material separation between the proofrolled subgrade and sand subbase. The geotextile fabric should conform with the requirements of Type IV fabric in Section 714.13 of the MSSRBC.

152. Alternately, a geogrid and/or geosynthetic reinforcement may be considered to reduce the pavement section thicknesses. If a geogrid is used, we recommend the aggregate

surface pavement section have an overall thickness of at least 24 inches. This pavement section should consist of 10 inches of crushed stone aggregate surface material and 14 inches of sand subbase course with a layer of geogrid for additional stabilization of the pavement section. If existing subgrades are verified (or improved) to a CBR value of 4, 5, or 6 and geogrid stabilization is utilized, the crushed stone surface course can be reduced to 8 inches and the sand subbase course should be 14 inches, 12 inches, or 10 inches, respectively. We recommend the geogrid consist of Tensar BX1100 or equivalent. Evaluation of other alternatives can be made as a supplement to this report once traffic intensities are finalized.

153. Rutting. Routine maintenance will be required to fill ruts which develop over the design life as well as to provide for aggregate loss due to erosion and wear. If a geogrid or geosynthetic reinforcement is utilized, ruts should not be regraded for maintenance. Rather, additional crushed stone should be used to fill ruts and the resulting surface leveled with a bulldozer.

154. Our analyses assume the material for the crushed stone surface course conforms to the requirements of Section 703.07 of the MSSRBC for Class I or II, Group A or B course aggregates. The stone surface course should be placed and compacted in accordance with Section 304 of the MSSRBC. We recommend the stone be placed in two lifts with the geogrid being provided at the bottom of the first lift, if utilized. The sand subbase should be compacted to at least 95% of its maximum dry density within ±2% of optimum moisture content using ASTM D1557.

Vibrations

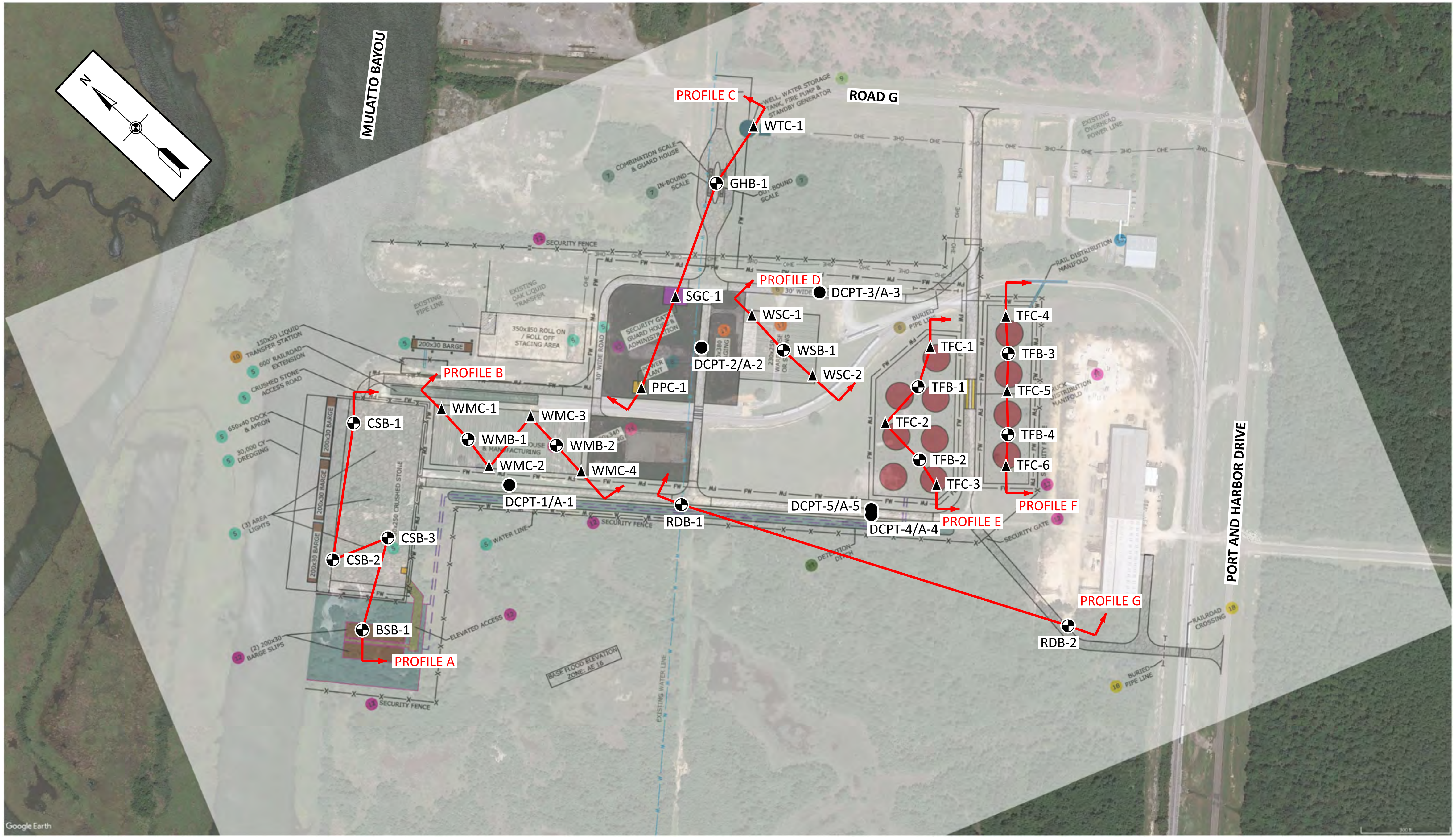
155. Demolition operations, driven pile installations, and other construction activities have the potential to generate vibrations that may affect nearby structures, pavements, and underground utilities. Eustis Engineering recommends vibrations be monitored during the demolition operation, test pile program, and construction activities of concern. This monitoring should evaluate peak particle velocities with a seismograph at critical structures during construction activities generating vibrations (demolition, hauling fill, moving heavy equipment, etc.). The record of peak particle velocities will provide information in assessing potential damage and the need for changes in construction operations.

156. Peak particle velocities (measured at a structure) exceeding 0.5 in./sec may induce damage to the structure, particularly when this structure has been previously stressed by settlement or other movements. Peak particle velocities between 0.25 and 0.5 in./sec may be sensed as being detrimental by human perception. In addition, sustained peak particle velocities of 0.25 in./sec have been documented to densify cohesionless materials such as those encountered by our exploration. Such densification can result in settlement of structures, pavements, and utilities founded over or in these deposits. If sustained vibration levels of 0.25 in./sec are measured at a structure, pavement, or utility of concern, Eustis Engineering should be notified. If sustained peak particle velocities exceed 0.5 in./sec, the construction operations generating these vibrations should be terminated and consideration should be given to altering these procedures.

GEOTECHNICAL SERVICES DURING CONSTRUCTION

157. In order to provide continuity among the exploration, design, and construction phases, Eustis Engineering will prepare a scope of service to review plans and specifications developed for the project and all submittals related to geotechnical issues and foundations. Eustis Engineering will include additional geotechnical services for consultation during design and construction. In our scope of service, we will also include steel and concrete inspection services, and compaction and inplace density determinations on fill materials. We intend to perform appropriate laboratory tests to determine the gradation and quality of material proposed as structural fill or backfill. Eustis Engineering also intends to log the installation of test piles and job piles, perform and evaluate static and pile load tests, and monitor vibrations.

158. Eustis Engineering should be engaged to monitor all geotechnical related work performed by the contractor. This permits the geotechnical engineer that prepared the report to be on hand and quickly evaluate unanticipated conditions; conduct additional tests, if required; and recommend alternative solutions to problems, when necessary. This is recommended to avoid major construction cost overruns or contractual disputes on the project.



SATELLITE IMAGERY DATED: 25 AUGUST 2015

400 200 0 400

SCALE: 1" = 400'

● DENOTES LOCATIONS OF AUGER BORINGS PERFORMED IN CONJUNCTION WITH DYNAMIC CONE PENETRATION TESTS ON 31 JANUARY 2019

◐ DENOTES LOCATIONS OF UNDISTURBED SOIL BORINGS DRILLED BETWEEN 23 OCTOBER AND 6 NOVEMBER 2018

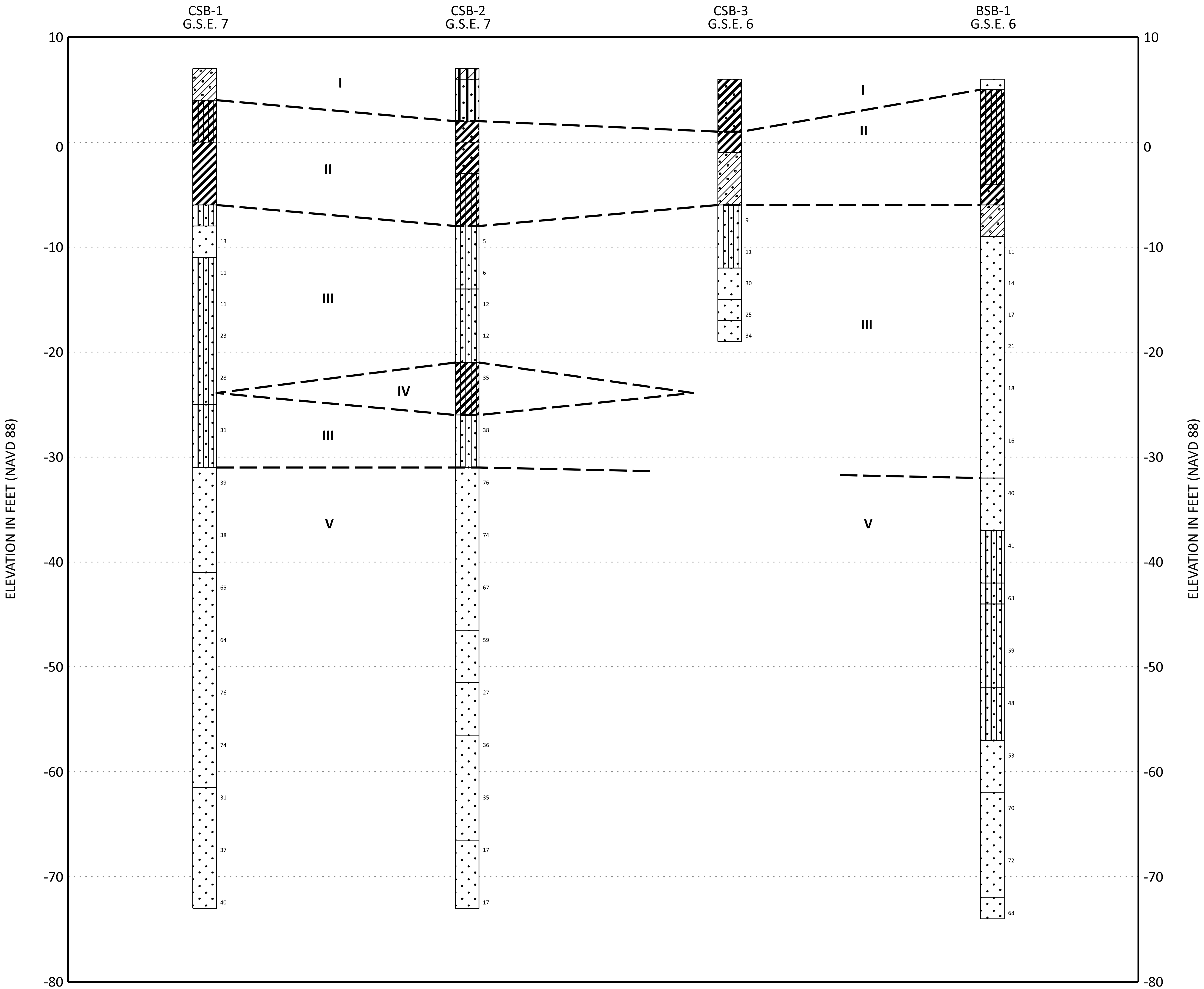
▲ DENOTES LOCATIONS OF CONE PENETRATION TESTS PERFORMED BETWEEN 26 AND 30 OCTOBER 2018

BORING AND CONE PENETRATION TEST LOCATION PLAN
AND SUBSOIL PROFILE DESIGNATIONS

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C.
SINCE 1946

DRAWN BY: J.L.S.	JOB NO.: G0389
CHECKED BY: P.A.T.	DATE: 15 FEB 2019
CADD FILE: B-CPT PLAN.DGN	FIGURE 1



BORING MATERIAL GRAPHICS

- CLAY
- CLAYEY SAND
- SILTY CLAY
- SANDY SILT
- SANDY CLAY
- CLAYEY SILT
- SAND
- SILTY SAND

NOTES:

1. NUMBERS TO THE RIGHT OF BORING LOGS REPRESENT RESULTS OF STANDARD PENETRATION TESTS.
2. ALL GROUND SURFACE ELEVATIONS (G.S.E.) ARE APPROXIMATE AND ESTIMATED FROM A FURNISHED WETLAND CONTOUR MAP (SHEET C0.0)
3. EXISTING AND PROPOSED TOPGOGRAPHY IS NOT SHOWN BUT IS ESTIMATED TO VARY BETWEEN EL 3 AND EL 11 IN THIS PROJECT AREA AS SHOWN IN TABLE 5.
4. INTERPRETED STRATIGRAPHY IS SCHEMATIC ONLY AND DOES NOT DEPICT VARIATIONS THAT SHOULD BE ANTICIPATED BETWEEN AND AWAY FROM EXPLORATORY LOCATIONS.
5. GEOPHYSICAL MAPPING AND GEOLOGIC STUDIES FOR SITE CHARACTERIZATION HAVE NOT BEEN CONDUCTED AND SHOULD NOT BE ASSUMED FROM THE DATA SHOWN.

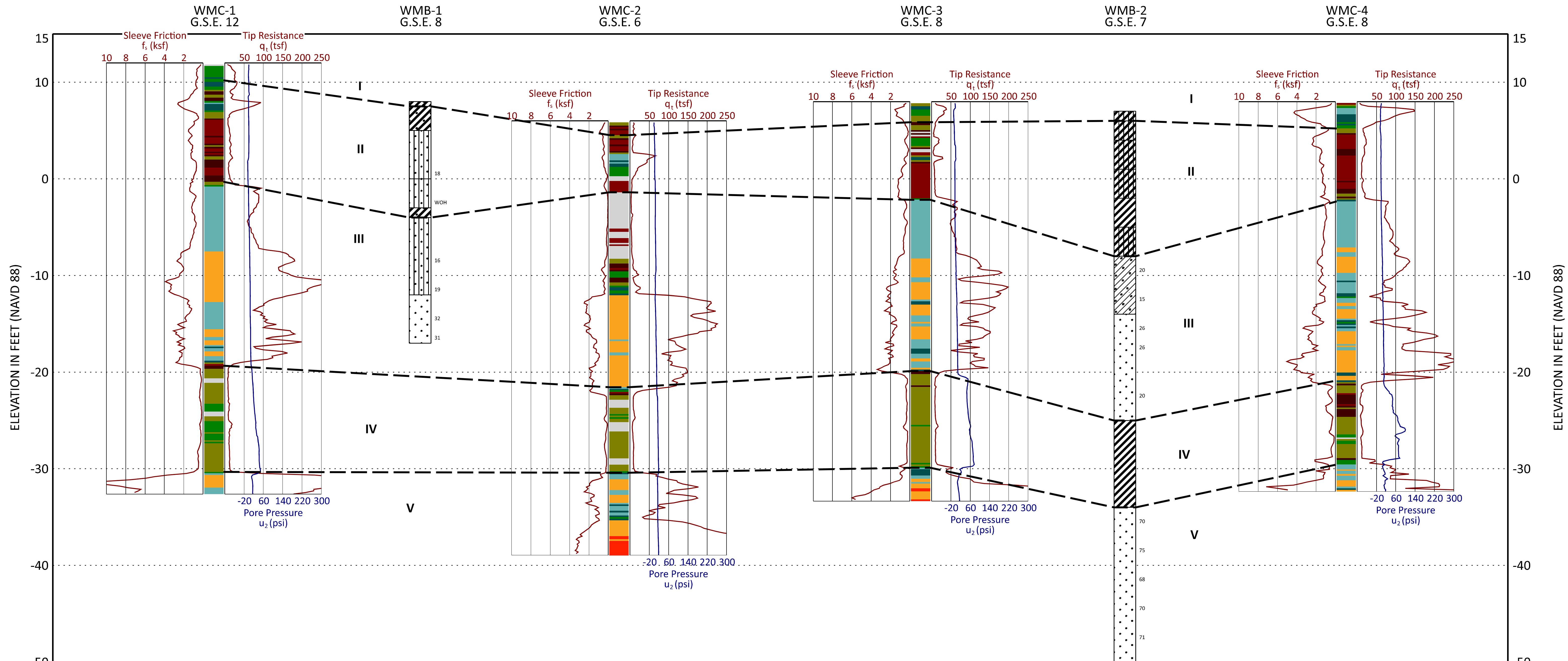
STRATA	GENERAL DESCRIPTION
I	SURFICIAL DEPOSITS OF LOOSE TO MEDIUM DENSE SANDS, SILTS, AND SANDY CLAY
II	MEDIUM STIFF TO STIFF CLAY, SILTY CLAY, AND SANDY CLAY WITH SOME SILTY SAND
III	MEDIUM DENSE TO DENSE SILTY SAND OR FINE SAND
IV	SOFT SILTY CLAY WITH FEW SAND POCKETS
V	DENSE TO VERY DENSE FINE TO COURSE SAND OR SILTY SAND

SUBSURFACE PROFILE A
BARGE SLIP, BARGE MOORING, AND CRUSHED STONE AREA
SOIL DESIGN REACH BS/CS

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS
ENGINEERING L.L.C.
SINCE 1946

DRAWN BY: J.L.S.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 15 FEB 2019
CADD FILE: PROFILES.DGN	FIGURE 2 (SHEET 1 OF 7)



CPT MATERIAL GRAPHICS

- SENSITIVE FINE GRAINED
- ORGANIC SOILS, PEATS
- CLAY
- SILTY CLAY TO CLAY
- CLAYEY SILT TO SILTY CLAY
- SANDY SILT TO CLAYEY SILT
- SILTY SAND TO SANDY SILT
- SAND TO SILTY SAND
- SAND
- GRAVELLY SAND TO SAND
- VERY STIFF FINE GRAINED (*)
- SAND TO CLAYEY SAND (*)

* OVERCONSOLIDATED OR CEMENTED
Robertson et al (1986) q_c vs Rf

BORING MATERIAL GRAPHICS

- CLAY
- SILTY CLAY
- SANDY CLAY
- SAND
- SILTY SAND
- CLAYEY SAND

STRATA	GENERAL DESCRIPTION
I	SURFICIAL DEPOSITS OF LOOSE TO MEDIUM DENSE SANDS, SILTS, AND SANDY CLAY
II	INTERBEDDED MEDIUM STIFF TO STIFF CLAY, SILTY CLAY, AND SANDY CLAY WITH LOOSE TO MEDIUM DENSE SILTY SAND
III	MEDIUM DENSE TO DENSE SILTY SAND, CLAYEY SAND, AND FINE SAND
IV	MEDIUM STIFF TO STIFF CLAY OR SILTY CLAY WITH SOME ORGANICS
V	DENSE TO VERY DENSE FINE TO COURSE SAND OR SILTY SAND

NOTES:

1. NUMBERS TO THE RIGHT OF BORING LOGS REPRESENT RESULTS OF STANDARD PENETRATION TESTS.
2. ALL GROUND SURFACE ELEVATIONS (G.S.E.) ARE APPROXIMATE AND ESTIMATED FROM A FURNISHED WETLAND CONTOUR MAP (SHEET C0.0)
3. EXISTING AND PROPOSED TOPGOGRAPHY IS NOT SHOWN BUT IS ESTIMATED TO VARY BETWEEN EL 5 AND EL 12 IN THIS PROJECT AREA AS SHOWN IN TABLE 5.
4. INTERPRETED STRATIGRAPHY IS SCHEMATIC ONLY AND DOES NOT DEPICT VARIATIONS THAT SHOULD BE ANTICIPATED BETWEEN AND AWAY FROM EXPLORATORY LOCATIONS.
5. GEOPHYSICAL MAPPING AND GEOLOGIC STUDIES FOR SITE CHARACTERIZATION HAVE NOT BEEN CONDUCTED AND SHOULD NOT BE ASSUMED FROM THE DATA SHOWN.

SUBSURFACE PROFILE B
WAREHOUSE/MANUFACTURING AREA
SOIL DESIGN REACH WM

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

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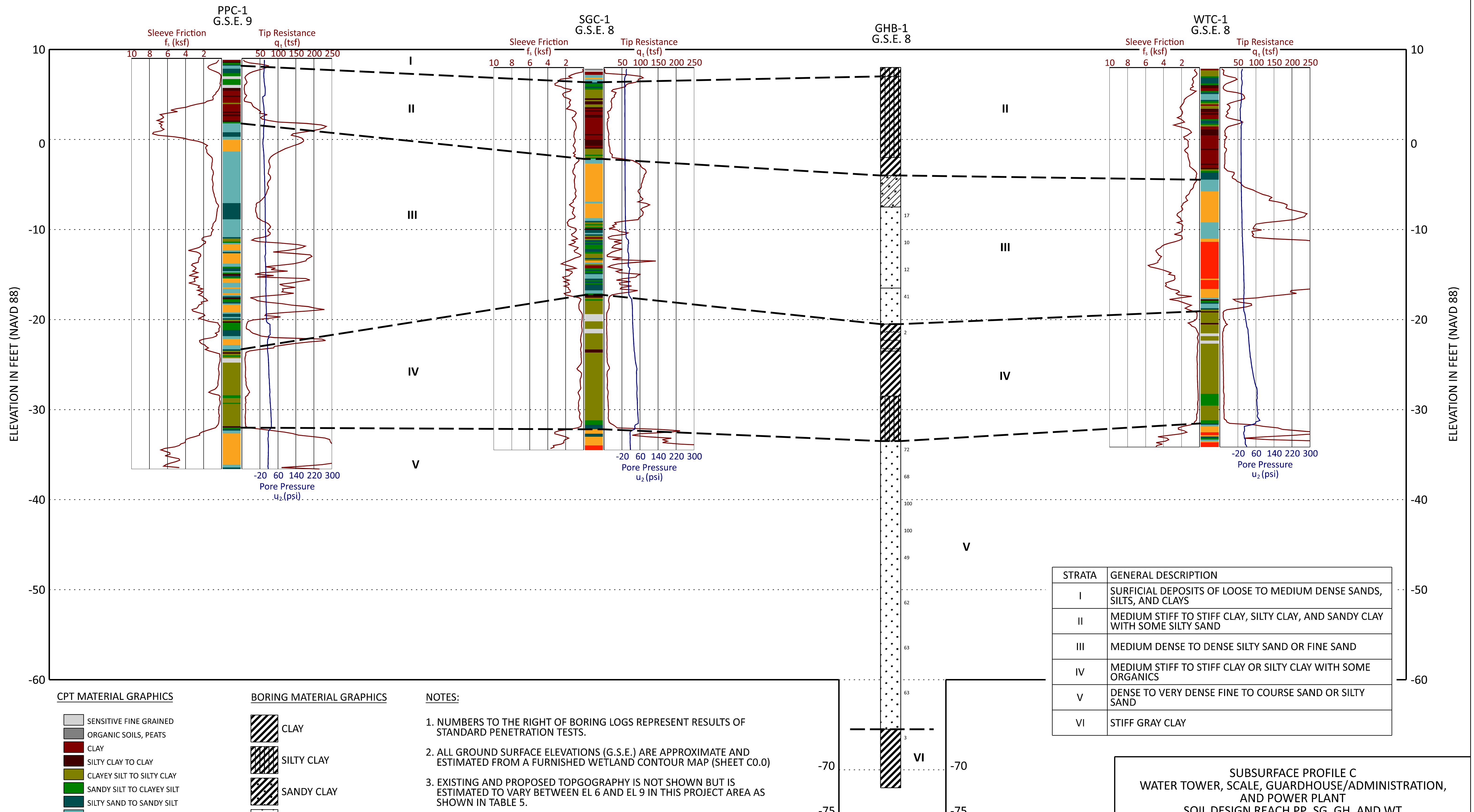
SINCE 1946

DRAWN BY: J.L.S.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 18 FEB 2019
CADD FILE: PROFILES.DGN	FIGURE 2 (SHEET 2 OF 7)



STRATA	GENERAL DESCRIPTION
I	SURFICIAL DEPOSITS OF LOOSE TO MEDIUM DENSE SANDS, SILTS, AND CLAYS
II	MEDIUM STIFF TO STIFF CLAY, SILTY CLAY, AND SANDY CLAY WITH SOME SILTY SAND
III	MEDIUM DENSE TO DENSE SILTY SAND OR FINE SAND
IV	MEDIUM STIFF TO STIFF CLAY OR SILTY CLAY WITH SOME ORGANICS
V	DENSE TO VERY DENSE FINE TO COURSE SAND OR SILTY SAND
VI	STIFF GRAY CLAY

CPT MATERIAL GRAPHICS

- SENSITIVE FINE GRAINED
- ORGANIC SOILS, PEATS
- CLAY
- SILTY CLAY TO CLAY
- CLAYEY SILT TO SILTY CLAY
- SANDY SILT TO CLAYEY SILT
- SILTY SAND TO SANDY SILT
- SAND TO SILTY SAND
- SAND
- GRAVELLY SAND TO SAND
- VERY STIFF FINE GRAINED (*)
- SAND TO CLAYEY SAND (*)

* OVERCONSOLIDATED OR CEMENTED
Robertson et al (1986) q_c vs Rf

BORING MATERIAL GRAPHICS

- CLAY
- SILTY CLAY
- SANDY CLAY
- SAND
- SILTY SAND
- CLAYEY SAND

NOTES:

1. NUMBERS TO THE RIGHT OF BORING LOGS REPRESENT RESULTS OF STANDARD PENETRATION TESTS.
2. ALL GROUND SURFACE ELEVATIONS (G.S.E.) ARE APPROXIMATE AND ESTIMATED FROM A FURNISHED WETLAND CONTOUR MAP (SHEET C0.0)
3. EXISTING AND PROPOSED TOPGOGRAPHY IS NOT SHOWN BUT IS ESTIMATED TO VARY BETWEEN EL 6 AND EL 9 IN THIS PROJECT AREA AS SHOWN IN TABLE 5.
4. INTERPRETED STRATIGRAPHY IS SCHEMATIC ONLY AND DOES NOT DEPICT VARIATIONS THAT SHOULD BE ANTICIPATED BETWEEN AND AWAY FROM EXPLORATORY LOCATIONS.
5. GEOPHYSICAL MAPPING AND GEOLOGIC STUDIES FOR SITE CHARACTERIZATION HAVE NOT BEEN CONDUCTED AND SHOULD NOT BE ASSUMED FROM THE DATA SHOWN.

SUBSURFACE PROFILE C
WATER TOWER, SCALE, GUARDHOUSE/ADMINISTRATION, AND POWER PLANT
SOIL DESIGN REACH PP, SG, GH, AND WT

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: J.L.S.	JOB NO.: G0389
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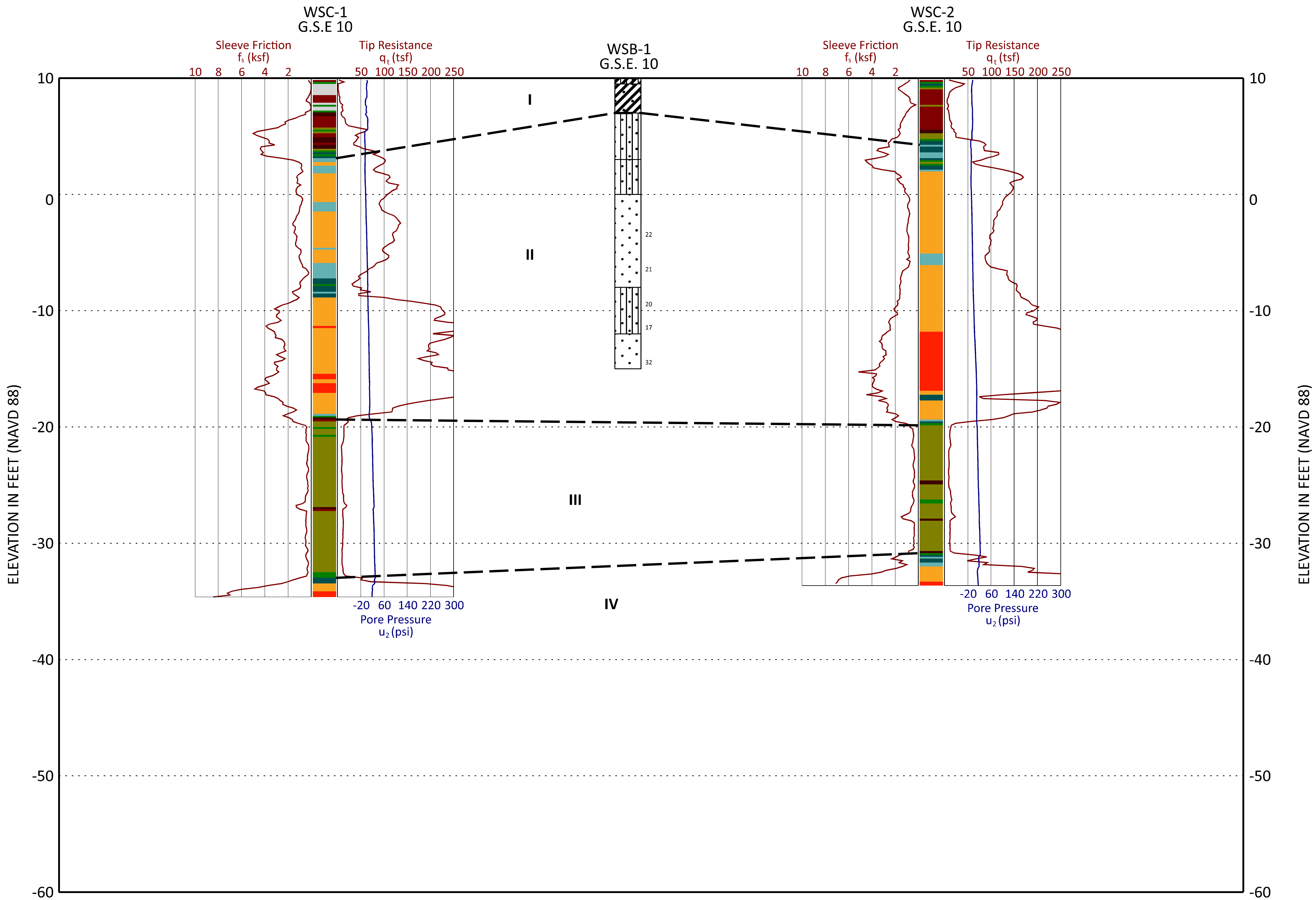
CADD FILE: PROFILES.DGN	FIGURE 2 (SHEET 3 OF 7)



CPT MATERIAL GRAPHICS

- SENSITIVE FINE GRAINED
- ORGANIC SOILS, PEATS
- CLAY
- SILTY CLAY TO CLAY
- CLAYEY SILT TO SILTY CLAY
- SANDY SILT TO CLAYEY SILT
- SILTY SAND TO SANDY SILT
- SAND TO SILTY SAND
- SAND
- GRAVELLY SAND TO SAND
- VERY STIFF FINE GRAINED (*)
- SAND TO CLAYEY SAND (*)

* OVERCONSOLIDATED OR CEMENTED

Robertson et al (1986) q_c vs Rf

BORING MATERIAL GRAPHICS

- SILTY CLAY
- SANDY CLAY
- SAND
- SILTY SAND

NOTES:

1. NUMBERS TO THE RIGHT OF BORING LOGS REPRESENT RESULTS OF STANDARD PENETRATION TESTS.
2. ALL GROUND SURFACE ELEVATIONS (G.S.E.) ARE APPROXIMATE AND ESTIMATED FROM A FURNISHED WETLAND CONTOUR MAP (SHEET C0.0)
3. EXISTING AND PROPOSED TOPGOGRAPHY IS NOT SHOWN BUT IS ESTIMATED TO BE AT EL 10 IN THIS PROJECT AREA AS SHOWN IN TABLE 5.
4. INTERPRETED STRATIGRAPHY IS SCHEMATIC ONLY AND DOES NOT DEPICT VARIATIONS THAT SHOULD BE ANTICIPATED BETWEEN AND AWAY FROM EXPLORATORY LOCATIONS.
5. GEOPHYSICAL MAPPING AND GEOLOGIC STUDIES FOR SITE CHARACTERIZATION HAVE NOT BEEN CONDUCTED AND SHOULD NOT BE ASSUMED FROM THE DATA SHOWN.

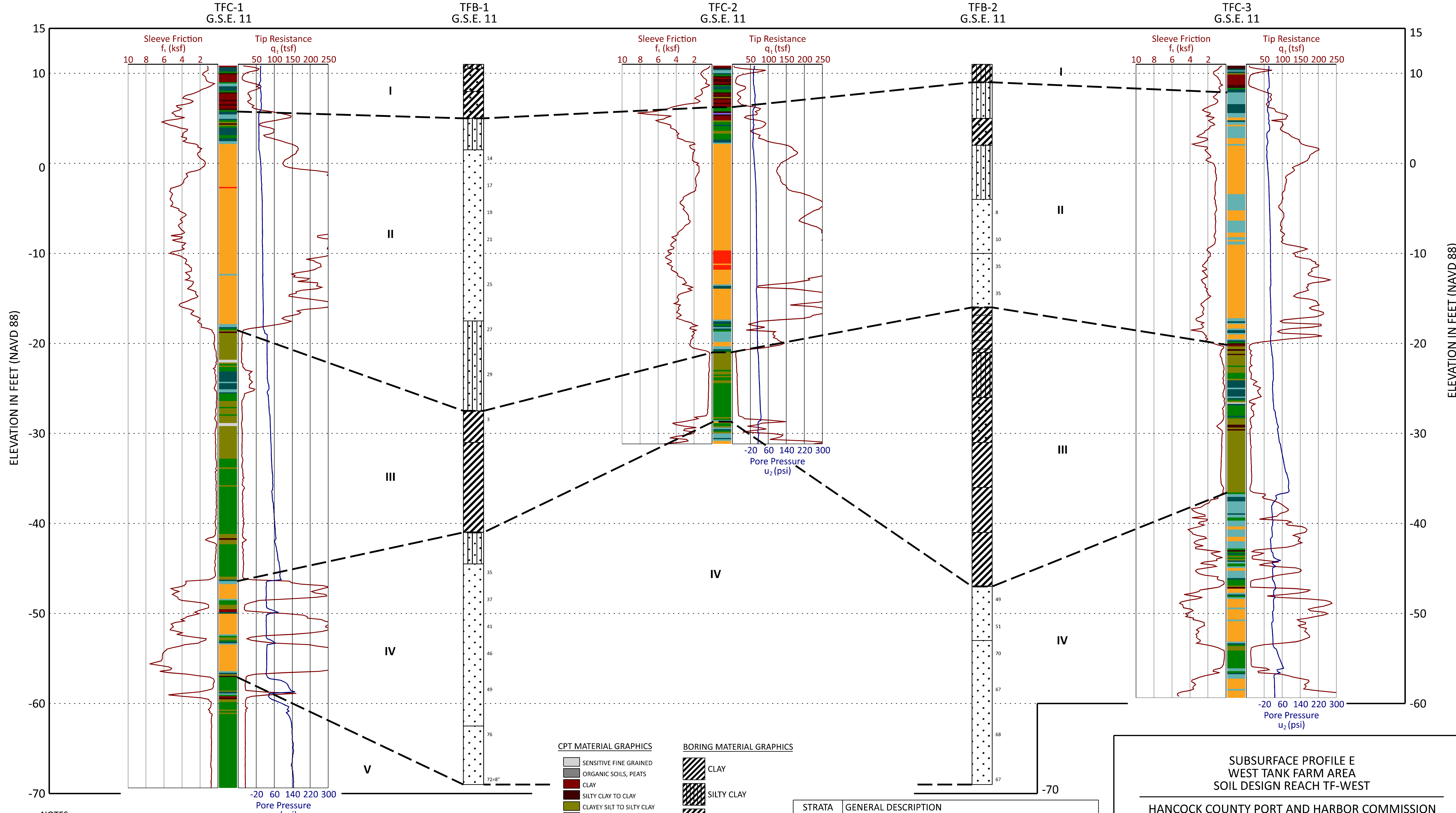
STRATA	GENERAL DESCRIPTION
I	MEDIUM STIFF TO STIFF CLAY, SILTY CLAY, AND SANDY CLAY WITH SOME SILTY SAND
II	MEDIUM DENSE TO DENSE SILTY SAND OR FINE SAND
III	MEDIUM STIFF TO STIFF CLAY OR SILTY CLAY WITH SOME ORGANICS
IV	DENSE TO VERY DENSE FINE TO COURSE SAND OR SILTY SAND

SUBSURFACE PROFILE D
WAREHOUSE/STAGING AREA
SOIL DESIGN REACH WS

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C.
SINCE 1946

DRAWN BY: J.L.S.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 15 FEB 2019
CADD FILE: PROFILES.DGN	FIGURE 2 (SHEET 4 OF 7)



NOTES:

1. NUMBERS TO THE RIGHT OF BORING LOGS REPRESENT RESULTS OF STANDARD PENETRATION TESTS.
2. ALL GROUND SURFACE ELEVATIONS (G.S.E.) ARE APPROXIMATE AND ESTIMATED FROM A FURNISHED WETLAND CONTOUR MAP (SHEET C0.0)
3. EXISTING AND PROPOSED TOPGOGRAPHY IS NOT SHOWN BUT IS ESTIMATED TO BE AT EL 11 IN THIS PROJECT AREA AS SHOWN IN TABLE 5.
4. INTERPRETED STRATIGRAPHY IS SCHEMATIC ONLY AND DOES NOT DEPICT VARIATIONS THAT SHOULD BE ANTICIPATED BETWEEN AND AWAY FROM EXPLORATORY LOCATIONS.
5. GEOPHYSICAL MAPPING AND GEOLOGIC STUDIES FOR SITE CHARACTERIZATION HAVE NOT BEEN CONDUCTED AND SHOULD NOT BE ASSUMED FROM THE DATA SHOWN.

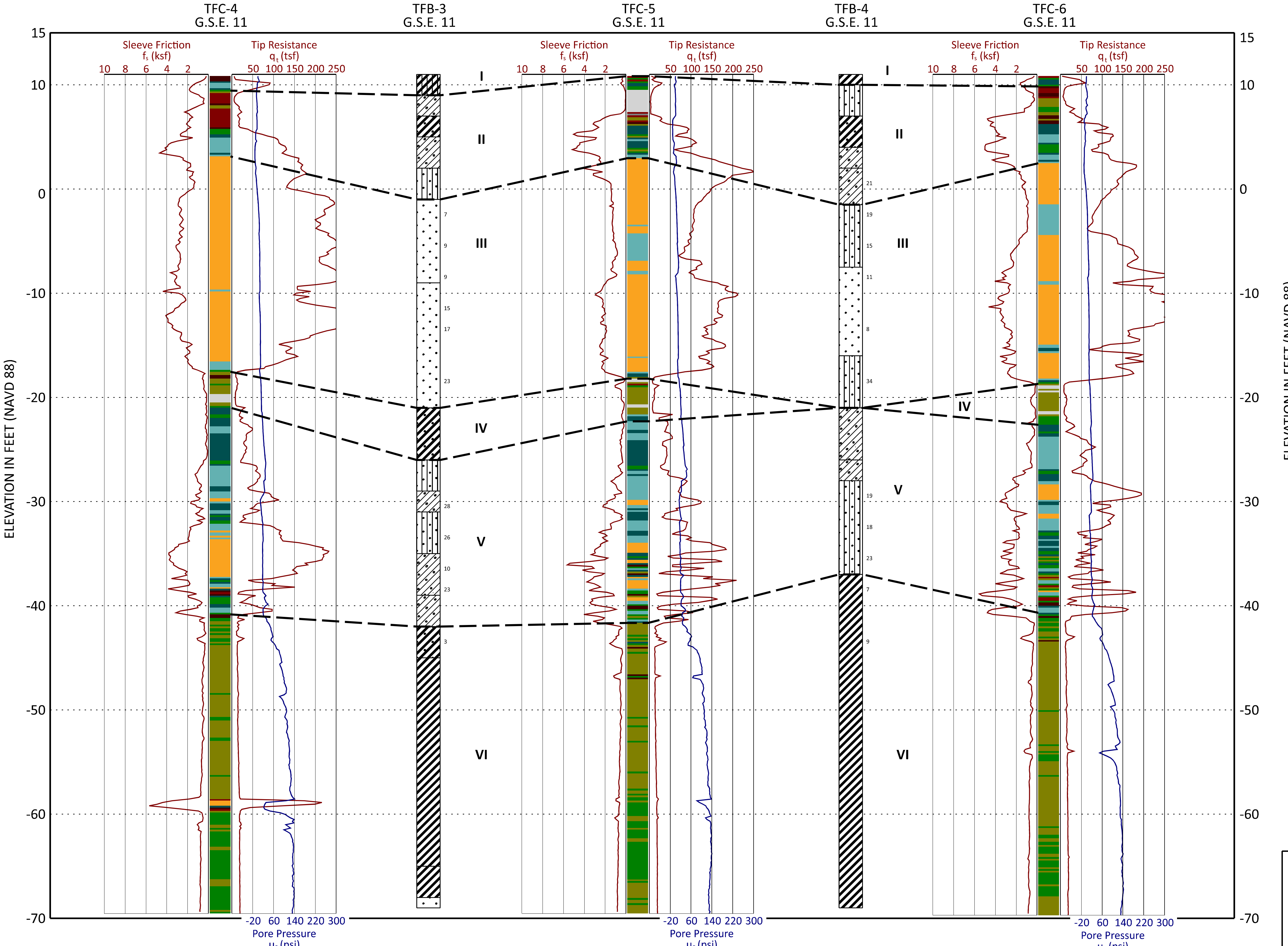
STRATA	GENERAL DESCRIPTION
I	MEDIUM STIFF TO STIFF CLAY, SILTY CLAY, AND SANDY CLAY WITH SOME SILTY SAND
II	MEDIUM DENSE TO DENSE SILTY SAND OR FINE SAND
III	MEDIUM STIFF TO STIFF CLAY OR SILTY CLAY WITH SOME ORGANICS
IV	DENSE TO VERY DENSE FINE TO COURSE SAND OR SILTY SAND
V	STIFF GRAY CLAY

SUBSURFACE PROFILE E
WEST TANK FARM AREA
SOIL DESIGN REACH TF-WEST

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C.
SINCE 1946

	DRAWN BY: J.L.S.	JOB NO.: G0389
	CHECKED BY: B.M.C.	DATE: 15 FEB 2019
	CADD FILE: PROFILES.DGN	FIGURE 2 (SHEET 5 OF 7)



CPT MATERIAL GRAPHICS

- SENSITIVE FINE GRAINED
- ORGANIC SOILS, PEATS
- CLAY
- SILTY CLAY TO CLAY
- CLAYEY SILT TO SILTY CLAY
- SANDY SILT TO CLAYEY SILT
- SILTY SAND TO SANDY SILT
- SAND TO SILTY SAND
- SAND
- GRAVELLY SAND TO SAND
- VERY STIFF FINE GRAINED (*)
- SAND TO CLAYEY SAND (*)

* OVERCONSOLIDATED OR CEMENTED

Robertson et al (1986) q_c vs Rf

BORING MATERIAL GRAPHICS

- SILTY CLAY
- SANDY CLAY
- SAND
- SILTY SAND
- CLAYEY SAND

NOTES:

1. NUMBERS TO THE RIGHT OF BORING LOGS REPRESENT RESULTS OF STANDARD PENETRATION TESTS.
2. ALL GROUND SURFACE ELEVATIONS (G.S.E.) ARE APPROXIMATE AND ESTIMATED FROM A FURNISHED WETLAND CONTOUR MAP (SHEET C0.0)
3. EXISTING AND PROPOSED TOPGOGRAPHY IS NOT SHOWN BUT IS ESTIMATED TO VARY BETWEEN EL 11 AND EL 12 IN THIS PROJECT AREA AS SHOWN IN TABLE 5.
4. INTERPRETED STRATIGRAPHY IS SCHEMATIC ONLY AND DOES NOT DEPICT VARIATIONS THAT SHOULD BE ANTICIPATED BETWEEN AND AWAY FROM EXPLORATORY LOCATIONS.
5. GEOPHYSICAL MAPPING AND GEOLOGIC STUDIES FOR SITE CHARACTERIZATION HAVE NOT BEEN CONDUCTED AND SHOULD NOT BE ASSUMED FROM THE DATA SHOWN.

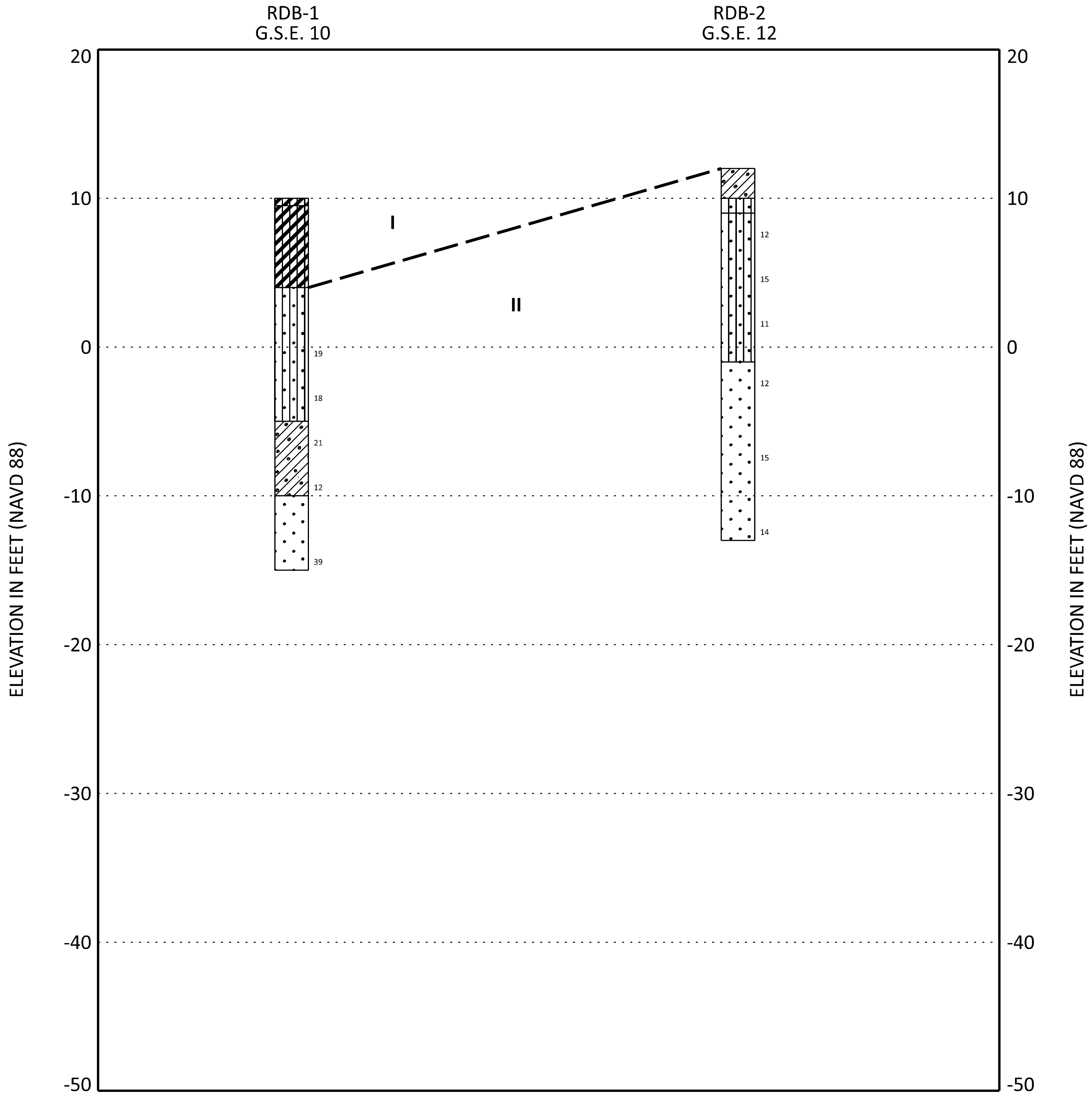
STRATA	GENERAL DESCRIPTION
I	SURFICIAL DEPOSITS OF LOOSE TO MEDIUM DENSE SANDS, SILTS, AND CLAYS
II	INTERBEDDED MEDIUM STIFF TO STIFF SANDY CLAY AND MEDIUM DENSE CLAYEY SAND AND SILTY SAND
III	LOOSE TO MEDIUM DENSE SILTY SAND OR FINE SAND
IV	MEDIUM STIFF TO STIFF CLAY OR SILTY CLAY WITH SOME ORGANICS
V	DENSE TO VERY DENSE FINE TO COURSE SAND OR SILTY SAND
VI	STIFF GRAY CLAY

SUBSURFACE PROFILE F
EAST TANK FARM AREA
SOIL DESIGN REACH TF-EAST

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: J.L.S.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 15 FEB 2019
CADD FILE: PROFILES.DGN	FIGURE 2 (SHEET 6 OF 7)



BORING MATERIAL GRAPHICS

- SILTY CLAY
- SAND
- SILTY SAND
- CLAYEY SAND

NOTES:

1. NUMBERS TO THE RIGHT OF BORING LOGS REPRESENT RESULTS OF STANDARD PENETRATION TESTS.
2. ALL GROUND SURFACE ELEVATIONS (G.S.E.) ARE APPROXIMATE AND ESTIMATED FROM A FURNISHED WETLAND CONTOUR MAP (SHEET C0.0)
3. EXISTING AND PROPOSED TOPGOGRAPHY IS NOT SHOWN BUT IS ESTIMATED TO VARY BETWEEN EL 10 AND EL 12 IN THIS PROJECT AREA AS SHOWN IN TABLE 5.
4. INTERPRETED STRATIGRAPHY IS SCHEMATIC ONLY AND DOES NOT DEPICT VARIATIONS THAT SHOULD BE ANTICIPATED BETWEEN AND AWAY FROM EXPLORATORY LOCATIONS.
5. GEOPHYSICAL MAPPING AND GEOLOGIC STUDIES FOR SITE CHARACTERIZATION HAVE NOT BEEN CONDUCTED AND SHOULD NOT BE ASSUMED FROM THE DATA SHOWN.

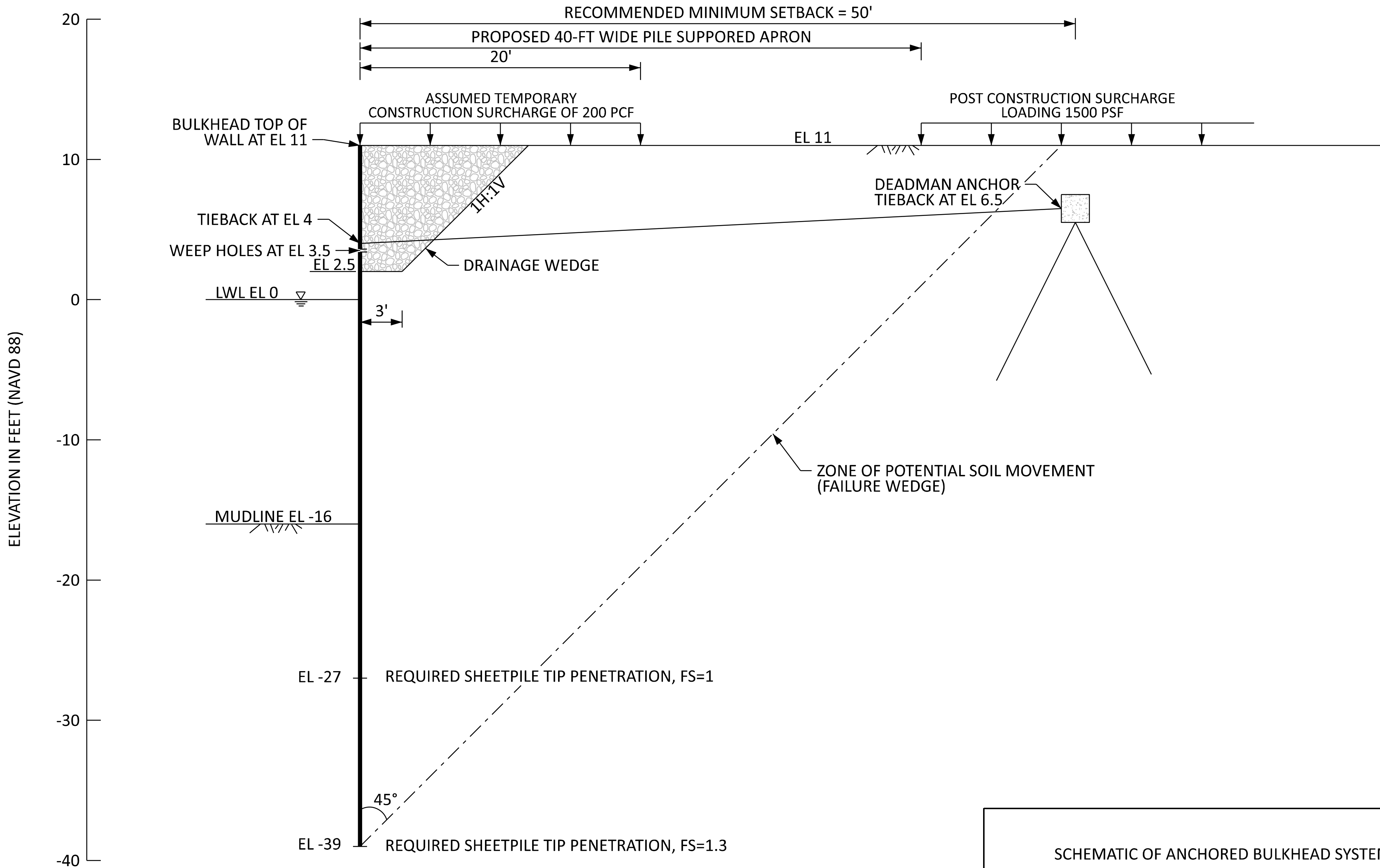
STRATA	GENERAL DESCRIPTION
I	MEDIUM STIFF TO STIFF CLAY, SILTY CLAY, AND SANDY CLAY WITH SOME SILTY SAND
II	MEDIUM DENSE TO DENSE SILTY SAND OR FINE SAND

SUBSOIL PROFILE G
RAILROAD BED BORINGS
SOIL DESIGN REACH RD

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: J.L.S.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 15 FEB 2019
CADD FILE: PROFILES.DGN	FIGURE 2 (SHEET 7 OF 7)



NOTES:

1. POST-CONSTRUCTION SURCHARGE LOADS ADJACENT TO THE BULKHEAD ASSUMED TO BE SUPPORTED BY A 40-FT WIDE PILE SUPPORTED APRON SYSTEM (NOT SHOWN HERE) ISOLATED FROM THE TIEBACK ANCHOR SYSTEM.

2. POST-CONSTRUCTION LOADS THAT ARE GRADE SUPPORTED BEYOND 40 FEET WILL NOT AFFECT THE NEW BULKHEAD BUT MAY IMPACT GLOBAL STABILITY OF EXISTING SLOPES AND SHOULD BE EVALUATED.

3. VERTICAL PILES SUPPORTING THE APRON MUST BE EMBEDDED TO A MINIMUM TIP AT EL -25 TO MINIMIZE LATERAL LOADING ON THE BULKHEAD.

SCHEMATIC OF ANCHORED BULKHEAD SYSTEM

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: J.L.S.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 15 FEB 2019
CADD FILE: BULKHEAD.DGN	FIGURE 3

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI
EUSTIS ENGINEERING PROJECT NO. G0389

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
TREATED ASTM D25 QUALITY TIMBER PILES

WATER TOWER, SCALE, GUARDHOUSE/ADMINISTRATION, AND POWER PLANT
PROFILE C - SOIL DESIGN REACH PP, SG, GH, WT

PILE DIAMETER	PILE LENGTH IN FEET[1][2]	PILE TIP ELEVATION (NAVD 88)[3]	ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES IN TONS[4][5] FACTOR OF SAFETY $\approx$ 2[6][7]	
			COMPRESSION	TENSION
6-In. Tip 8-In. Butt	20 25	-12 -17	5½ 6½[3]	3½ 4
8-In. Tip 12-In. Butt	30 35 40	-22 -27 -32	11½ 14 16	7½ 9½ 11
7-In. Tip 12-In. Butt	45	-37	23½	13½

[1]Selection of pile tip embedment should also consider settlement potential.
[2]Capacities neglect friction in the upper 2 feet for embedment within a pile cap.
[3]Pile capacities assume average grade and a pile cutoff at el 8.
[4]These estimated capacities do not include limitations on structural capacity as imposed by some building codes.
[5]Piles assumed to be installed vertically by impact driving equipment without assistance from vibratory equipment.
[6]Use of a factor of safety of 2 assumes a static pile load test will be performed.
[7]Capacities shown on this figure derived from stratigraphy as seen in Borings WSB-1 and GHB-1, and CPTS PPC-1, SGC-1, and WTC-1 as shown on Figure 2, Sheet 3.

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI
EUSTIS ENGINEERING PROJECT NO. G0389

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
TREATED ASTM D25 QUALITY TIMBER PILES

WAREHOUSE/MANUFACTURING AREA

PILE DIAMETER	PILE LENGTH IN FEET[1][2]	PILE TIP ELEVATION (NAVD 88)[3]	ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES IN TONS[4][5] FACTOR OF SAFETY ≈ 2[6][7]	
			COMPRESSION	TENSION
6-In. Tip 8-In. Butt	20 25	-12 -17	4 5½	2½ 3
8-In. Tip 12-In. Butt	30 35 40	-22 -27 -32	9½ 12 13	6½ 8 9
7-In. Tip 12-In. Butt	45	-37	21½	10½

[1]Selection of pile tip embedment should also consider settlement potential.
[2]Capacities neglect friction in the upper 2 feet for embedment within a pile cap.
[3]Pile capacities assume a pile cutoff at el 8.
[4] These estimated capacities do not include limitations on structural capacity as imposed by some building codes.
[5]Piles assumed to be installed vertically by impact driving equipment without assistance from vibratory equipment.
[6]Use of a factor of safety of 2 assumes a static pile load test will be performed.
[7]Capacities shown on this figure derived from stratigraphy as seen in Borings WMB-1 and WMB-2, and CPTS WMC-1 through WMC-4 as shown on Figure 2, Sheet 2.

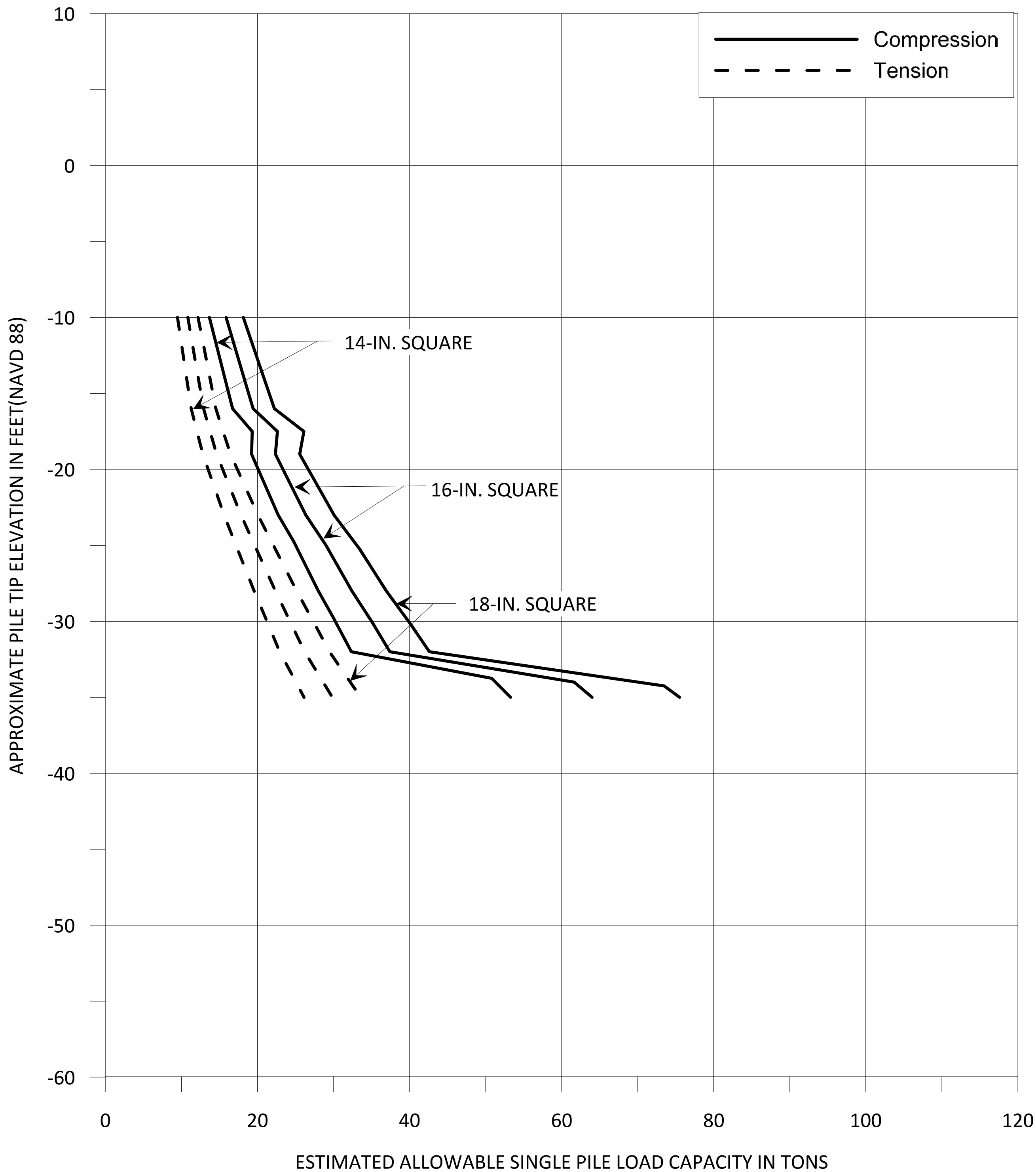
HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI
EUSTIS ENGINEERING PROJECT NO. G0389

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
TREATED ASTM D25 QUALITY TIMBER PILES

WAREHOUSE/STAGING AREA

PILE DIAMETER	PILE LENGTH IN FEET[1][2]	PILE TIP ELEVATION (NAVD 88)[3]	ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES IN TONS[4][5] FACTOR OF SAFETY ≈ 2[6][7]	
			COMPRESSION	TENSION
6-In. Tip 8-In. Butt	20 25	-10 -15	3½ 5½	2 3
8-In. Tip 12-In. Butt	30 35 40	-20 -25 -30	10 12½ 14½	6½ 8½ 10
7-In. Tip 12-In. Butt	45 50	-35 -40	23 25	12½ 16

[1]Selection of pile tip embedment should also consider settlement potential.
[2]Capacities neglect friction in the upper 2 feet for embedment within a pile cap.
[3]Pile capacities assume a pile cutoff at el 10.
[4]These estimated capacities do not include limitations on structural capacity as imposed by some building codes.
[5]Piles assumed to be installed vertically by impact driving equipment without assistance from vibratory equipment.
[6]Use of a factor of safety of 2 assumes a static pile load test will be performed. If dynamic pile testing is used in lieu of static pile testing, a factor of safety of 2.5 should be used. A factor of safety of 3 should be used if load testing will not be performed.
[7]Capacities shown on this figure derived from stratigraphy as seen in Boring WSB-1, and CPTS WSC-1 AND WSC-2 as shown on Figure 2, Sheet 4.



NOTES:

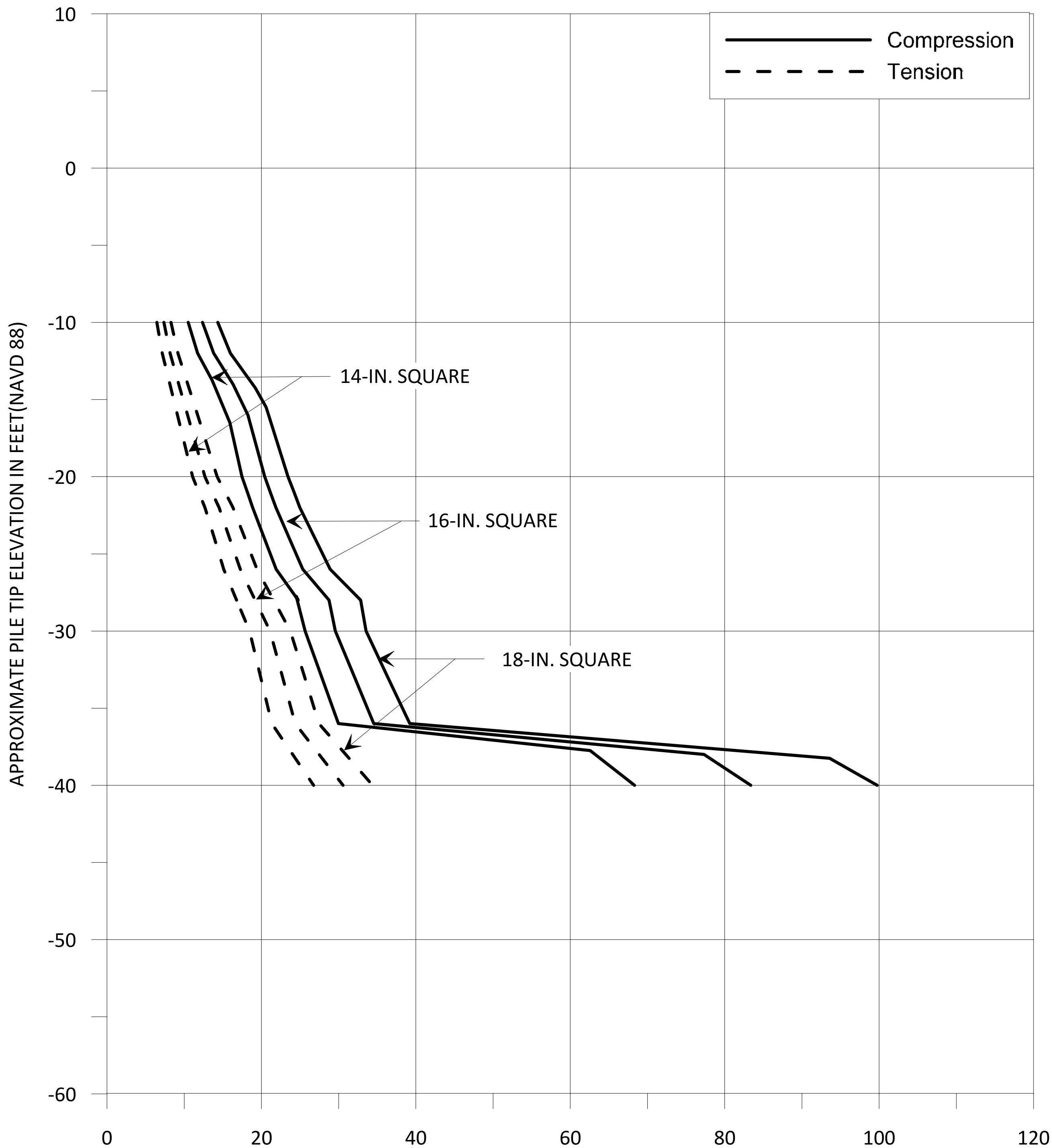
1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.
2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.
3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.
4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS WSB-1 AND GHB-1 AND CPTS PPC-1, SGC-1, AND WTC-1 AS SHOWN ON FIGURE 2, SHEET 3
5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.
6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 8.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
SQUARE, PRECAST CONCRETE PILES
WATER TOWER, SCALE, GUARDHOUSE/ADMINISTRATION, AND POWER PLANT

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 5 (SHEET 1 OF 5)



ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY IN TONS

FACTOR OF SAFETY ≈ 2

NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS WMB-1 AND WMB-2 AND CPTS WMC-1 THROUGH WMC-4 AS SHOWN ON FIGURE 2, SHEET 2.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

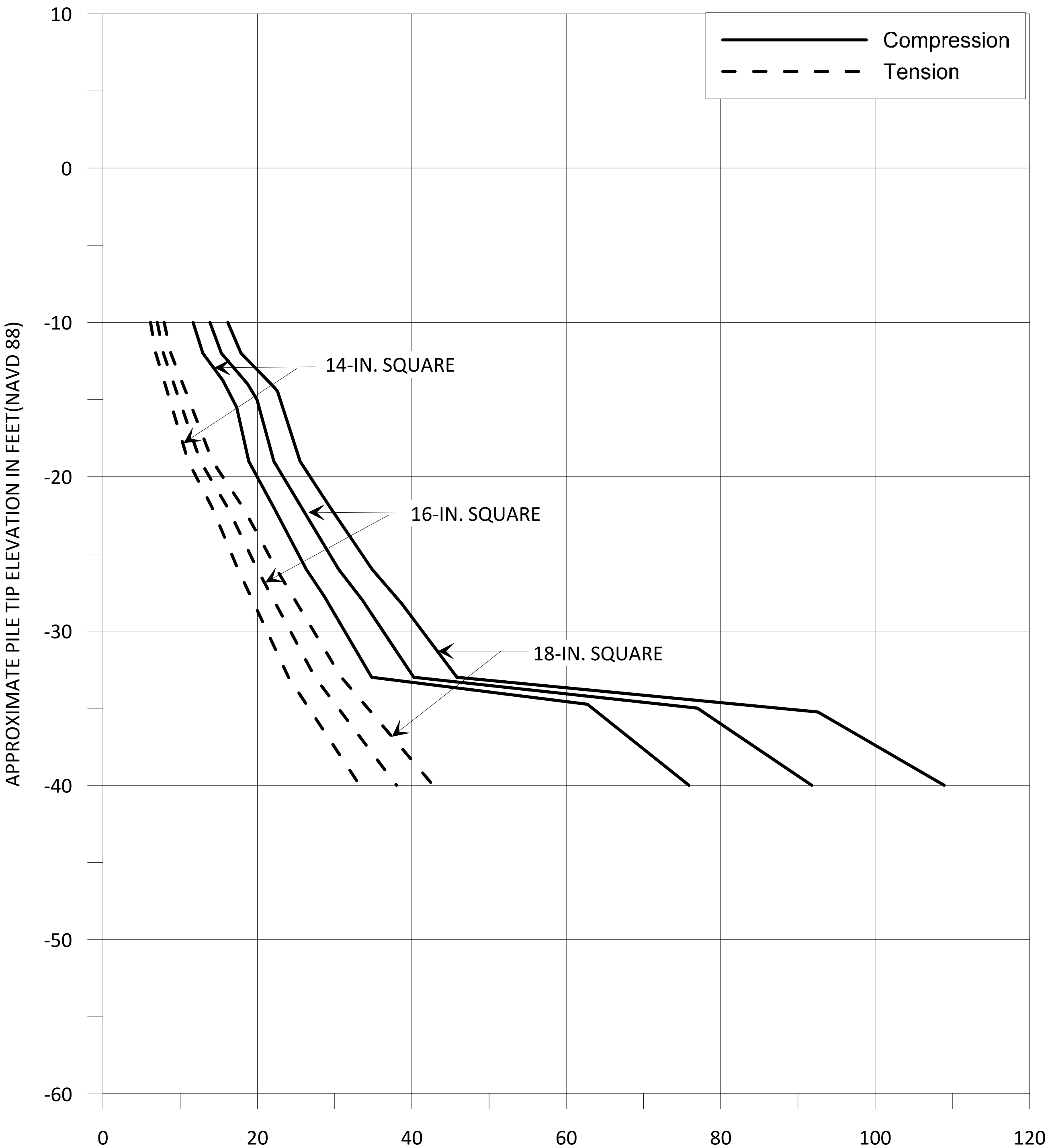
6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 8.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
SQUARE, PRECAST CONCRETE PILES
WAREHOUSE/MANUFACTURING AREA

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 5 (SHEET 2 OF 5)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORING WSB-1 AND CPTS WSC-1 AND WSC-2 AS SHOWN ON FIGURE 2, SHEET 4.

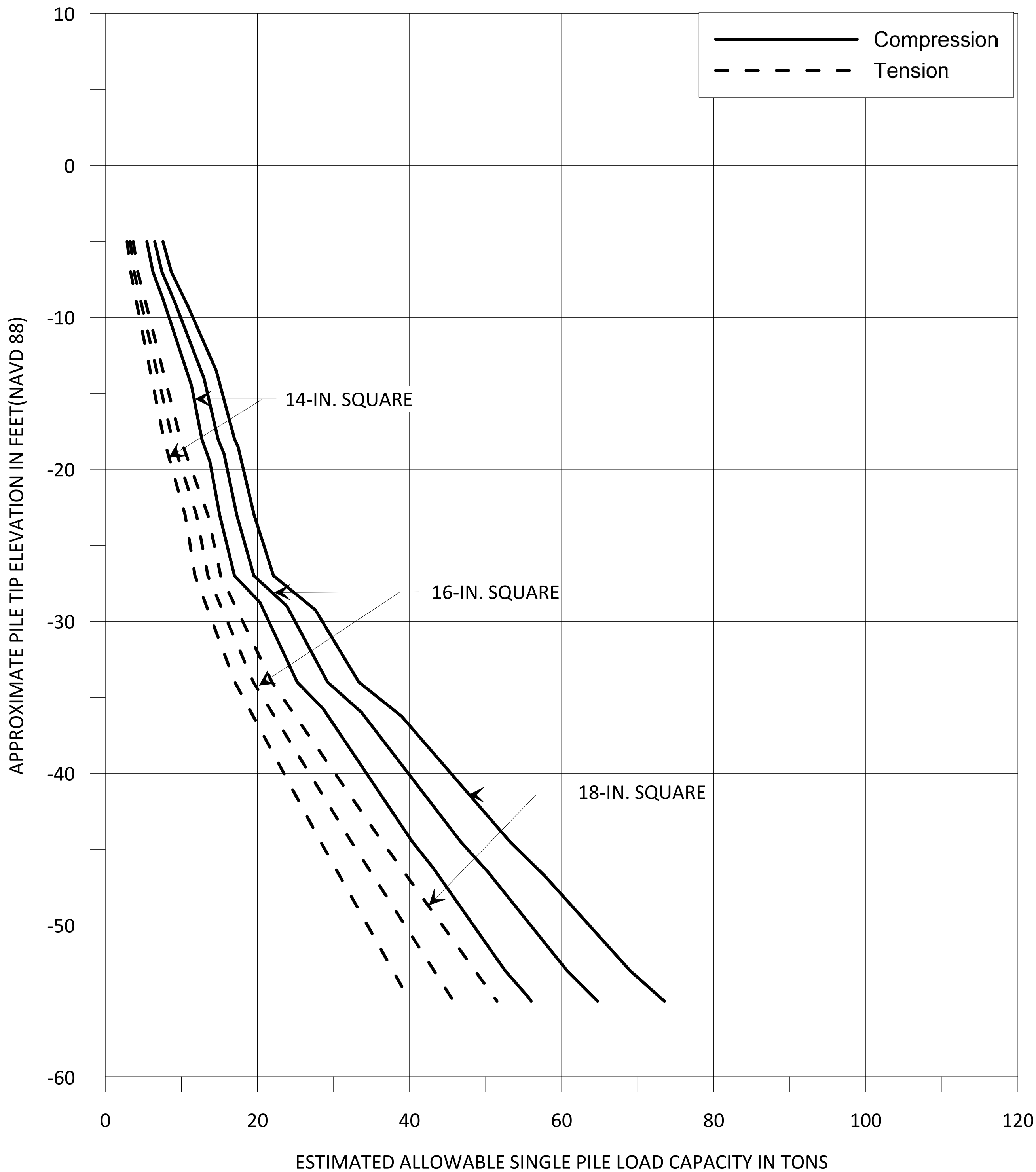
5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 10.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
SQUARE, PRECAST CONCRETE PILES
WAREHOUSE/STAGING AREA

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946	DRAWN BY: H.C.W.	JOB NO.: G0389
	CHECKED BY: B.M.C.	DATE: 27 DEC 2018
	FILE NAME: SPC.GRF	FIGURE 5 (SHEET 3 OF 5)



NOTES:

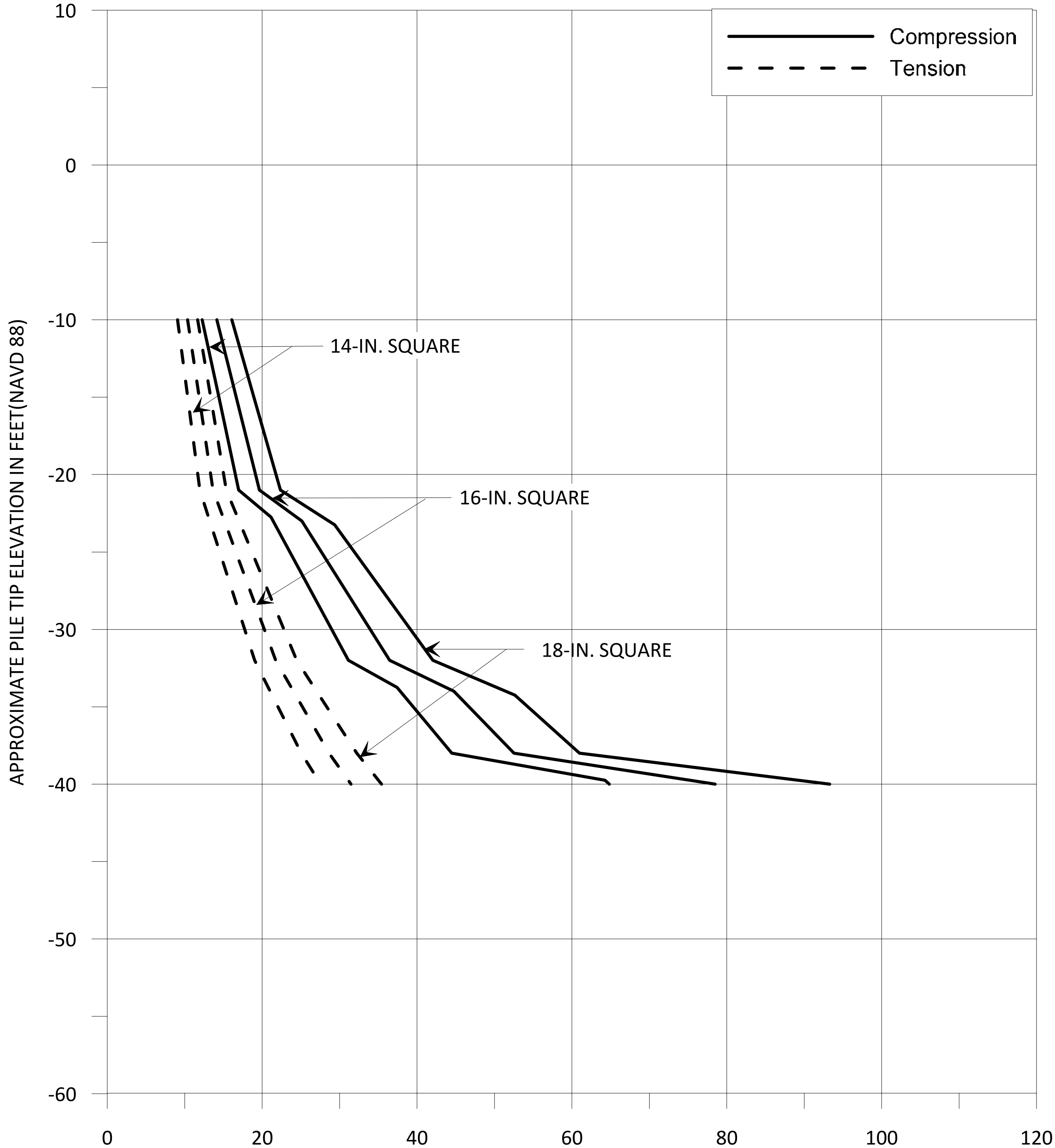
1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.
2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.
3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.
4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS TFB-1 THROUGH TFB-4 AND CPTS TFC-1 THROUGH TFC-6 AS SHOWN IN FIGURE 2, SHEETS 5 AND 6.
5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.
6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 11

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
SQUARE, PRECAST CONCRETE PILES
TANK FARM AREA (EAST AND WEST)

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 5 (SHEET 4 OF 5)



ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY IN TONS
FACTOR OF SAFETY ≈ 2

<u>NOTES</u>:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS BSB-1 AND CSB-1 THROUGH CSB-3 AS SHOWN ON FIGURE 2, SHEET 1.

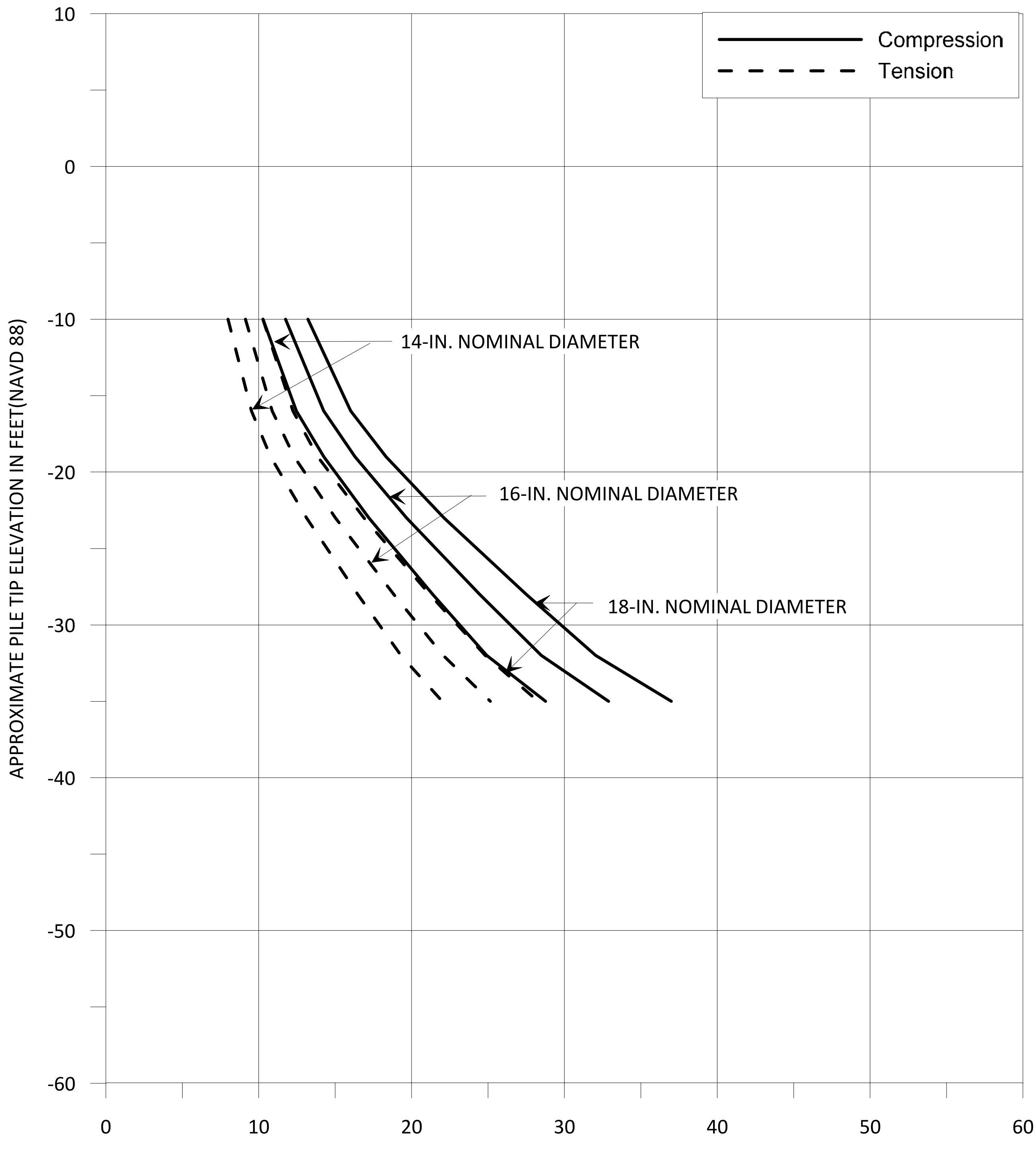
5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 4.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
SQUARE, PRECAST CONCRETE PILES
BARGE SLIP/CRUSHED STONE AREA, APRON, AND BULKHEAD ANCHORAGE

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946	DRAWN BY: H.C.W.	JOB NO.: G0389
	CHECKED BY: B.M.C.	DATE: 27 DEC 2018
	FILE NAME: SPC.GRF	FIGURE 5 (SHEET 5 OF 5)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. ESTIMATED CAPACITIES ASSUME ACTUAL GROUT VOLUME IS AT LEAST 130% OF THE THEORETICAL VOLUME.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS WSB-1 AND GHB-1 AND CPTS PPC-1, SGC-1, AND WTC-1 AS SHOWN ON FIGURE 2, SHEET 3.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

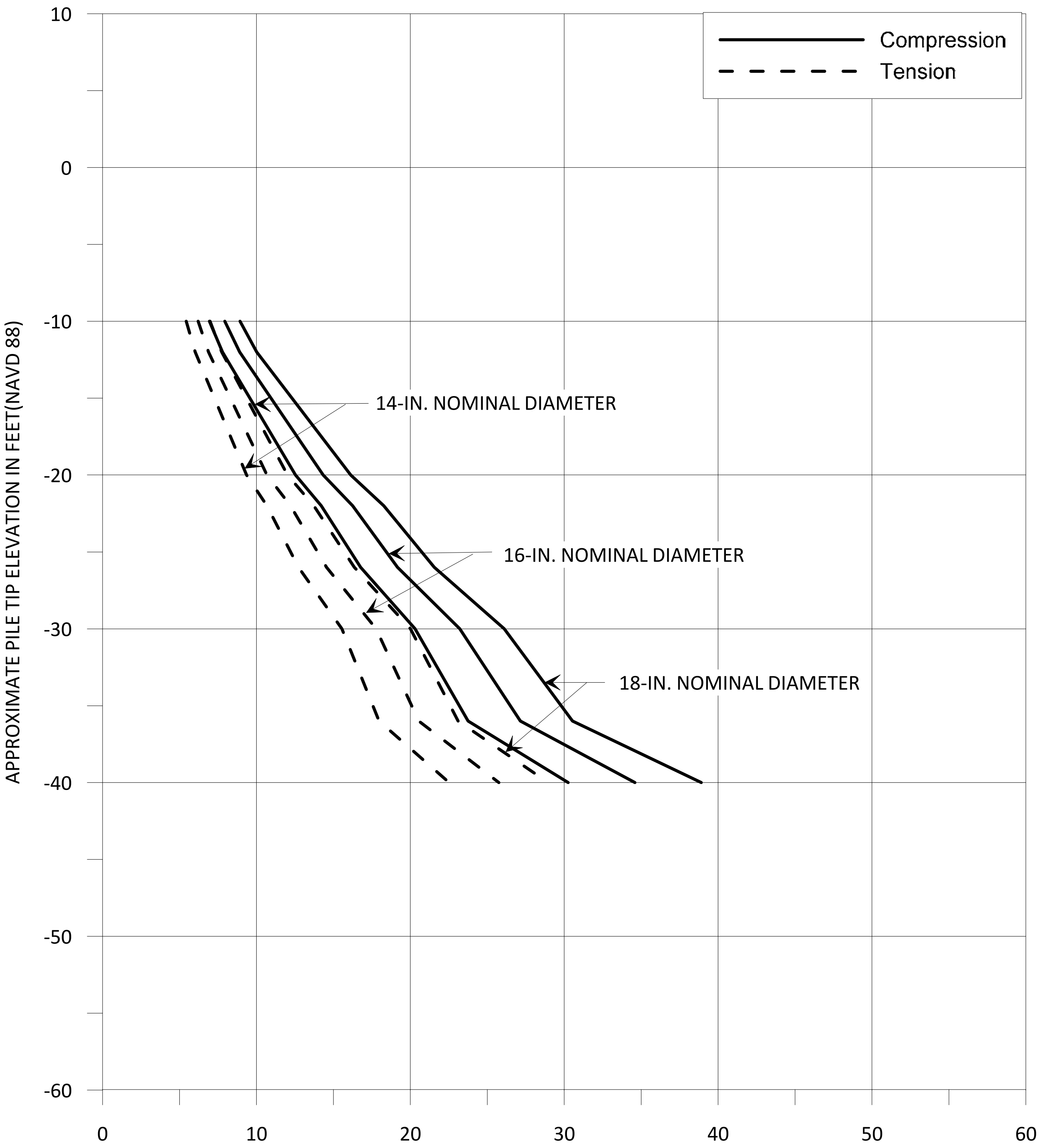
6. PILE CAPACITES ASSUME A PILE CUTOFF AT EL 8.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
AUGERED CAST-IN-PLACE PILES
WATER TOWER, SCALE, GUARDHOUSE/ADMINISTRATION, AND POWER PLANT

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 6 (SHEET 1 OF 5)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. ESTIMATED CAPACITIES ASSUME ACTUAL GROUT VOLUME IS AT LEAST 130% OF THE THEORETICAL VOLUME.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS WMB-1 AND WMB-2 AND CPTS WMC-1 THROUGH WMC-4 AS SHOWN ON FIGURE 2, SHEET 2.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

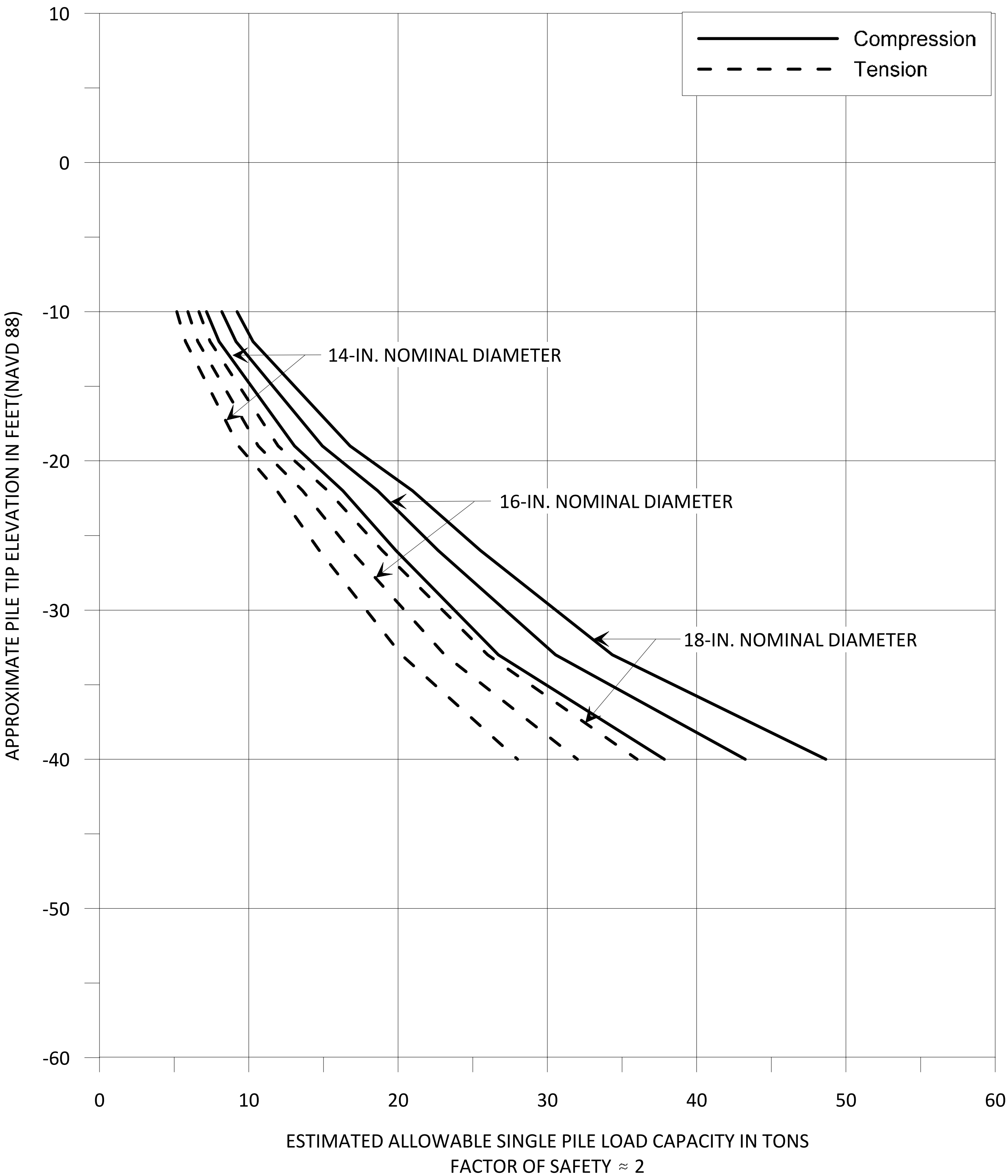
6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 8.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
AUGERED CAST-IN-PLACE PILES
WAREHOUSE/MANUFACTURING AREA

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 6 (SHEET 2 OF 5)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. ESTIMATED CAPACITIES ASSUME ACTUAL GROUT VOLUME IS AT LEAST 130% OF THE THEORETICAL VOLUME.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORING WSB-1 AND CPTS WSC-1 AND WSC-2 AS SHOWN ON FIGURE 2, SHEET 4.

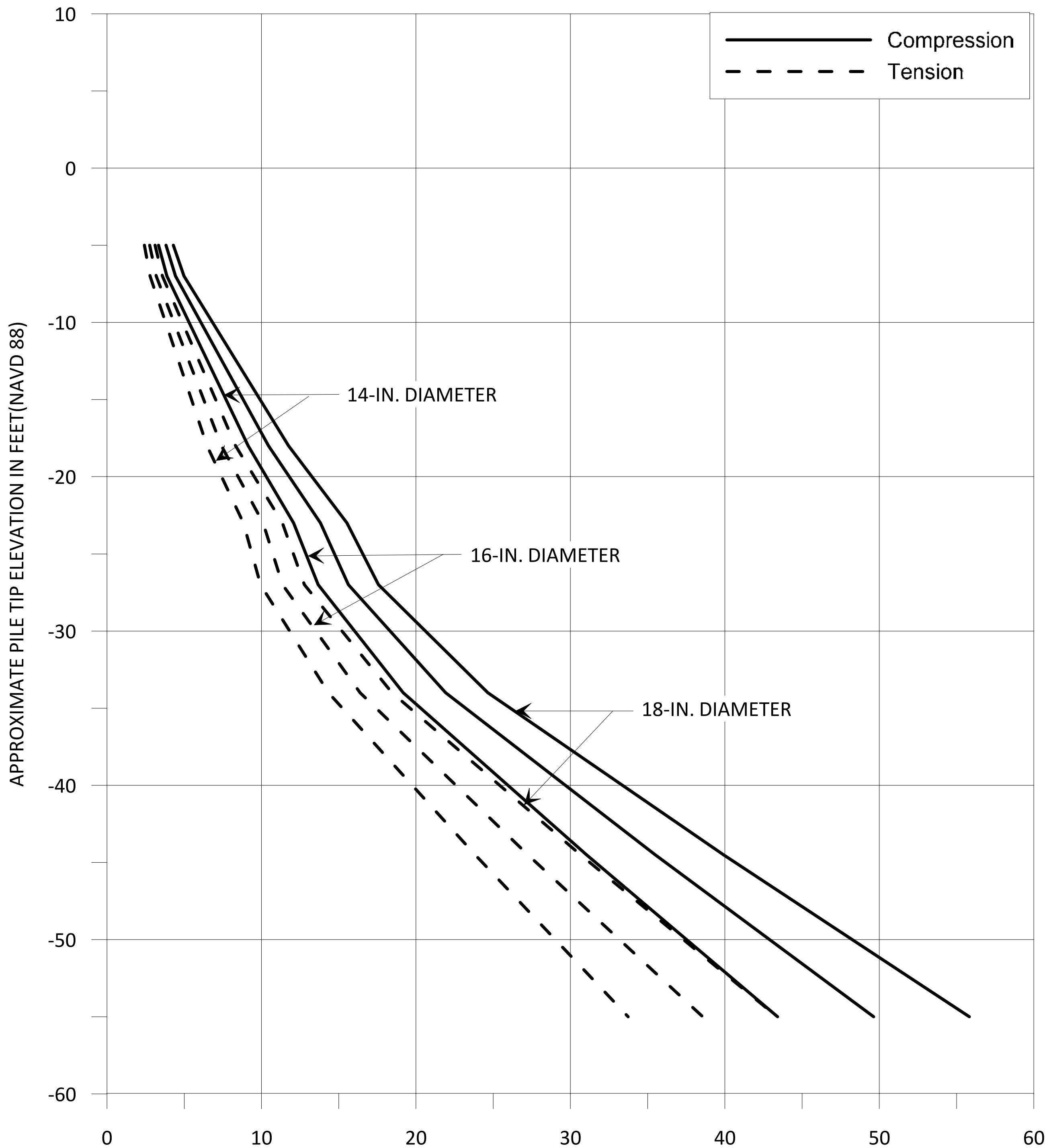
5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 10.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
AUGERED CAST-IN-PLACE PILES
WAREHOUSE/STAGING AREA

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946	DRAWN BY: H.C.W.	JOB NO.: G0389
	CHECKED BY: B.M.C.	DATE: 27 DEC 2018
	FILE NAME: SPC.GRF	FIGURE 6 (SHEET 3 OF 5)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. ESTIMATED CAPACITIES ASSUME ACTUAL GROUT VOLUME IS AT LEAST 130% OF THE THEORETICAL VOLUME.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS TFB-1 THROUGH TFB-4 AND CPTS TFC-1 THROUGH TFC-6 AS SHOWN IN FIGURE 2, SHEETS 5 AND 6.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

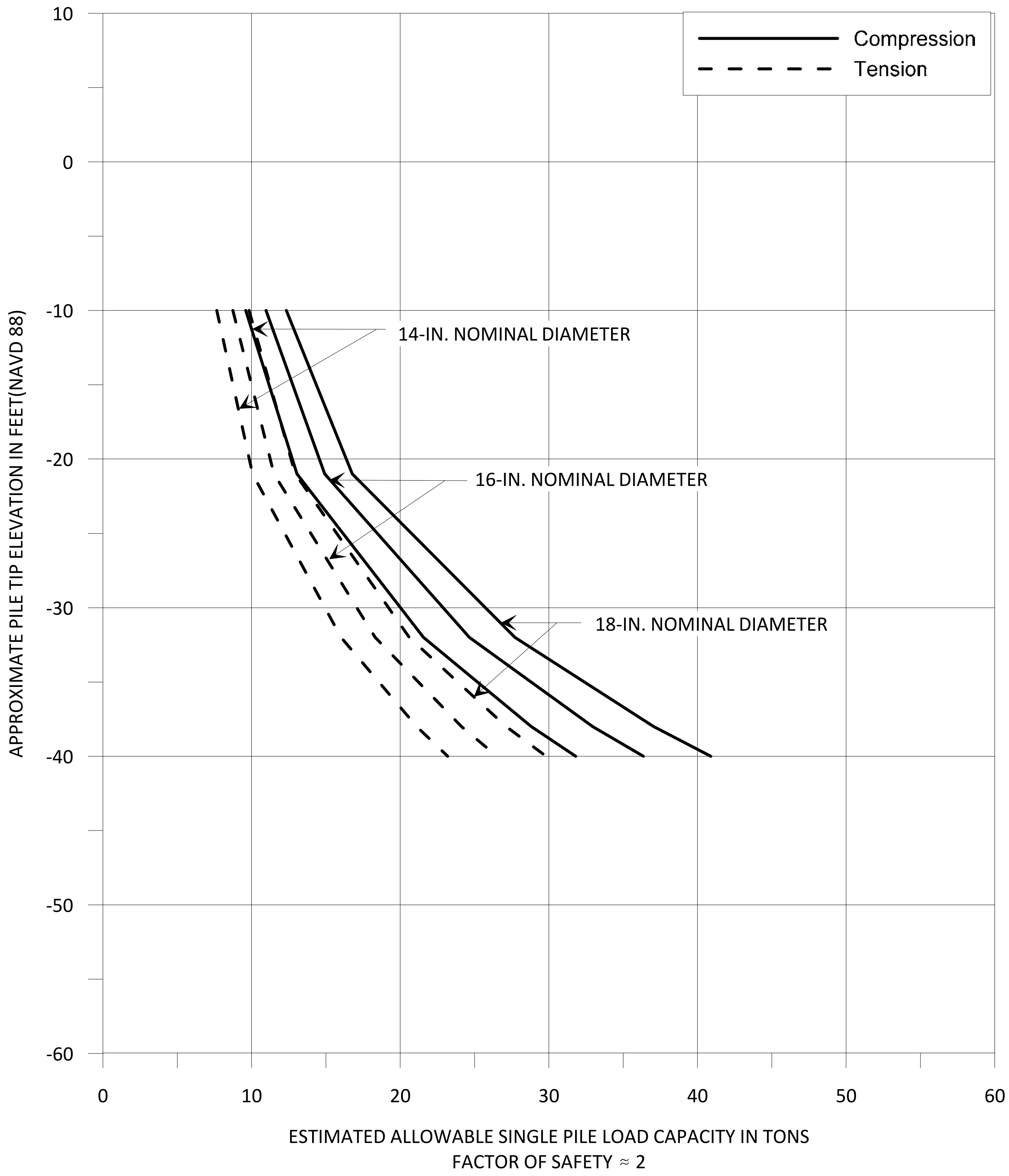
6. PILE CAPACITIES ASSUME PILE A CUTOFF AT EL 11.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
AUGERED CAST-IN-PLACE PILES
TANK FARM AREA (EAST AND WEST)

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 6 (SHEET 4 OF 5)



<u>NOTES</u>:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. ESTIMATED CAPACITIES ASSUME ACTUAL GROUT VOLUME IS AT LEAST 130% OF THE THEORETICAL VOLUME.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS BSB-1 AND CSB-1 THROUGH CSB-3 AS SHOWN ON FIGURE 2, SHEET 1.

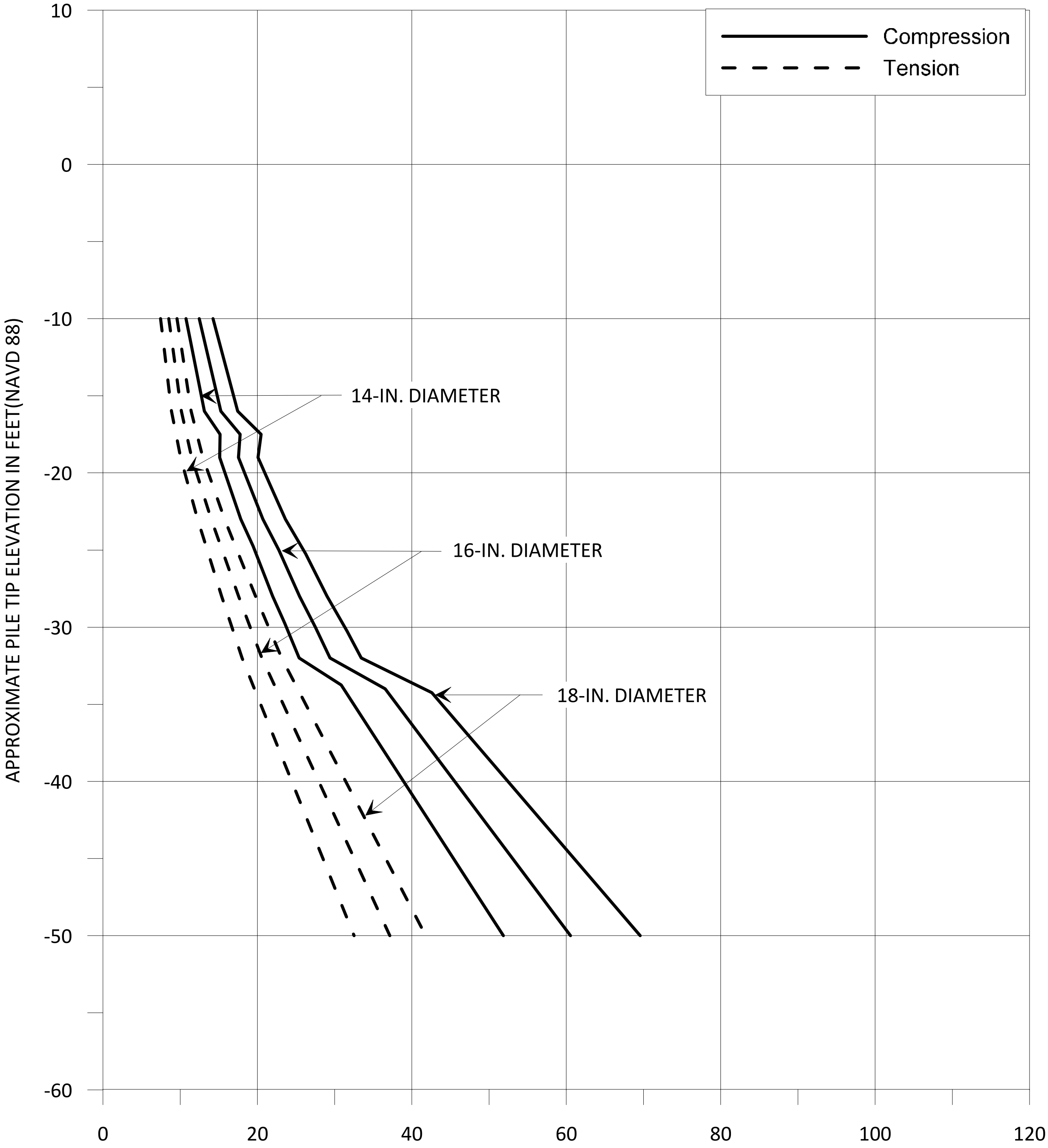
5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 4.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
AUGERED CAST-IN-PLACE PILES
BARGE SLIP/CRUSHED STONE AREA, APRON

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946		
	DRAWN BY: H.C.W.	JOB NO.: G0389
	CHECKED BY: B.M.C.	DATE: 27 DEC 2018
	FILE NAME: SPC.GRF	FIGURE 6 (SHEET 5 OF 5)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS WSB-1 AND GHB-1 AND CPTS PPC-1, SGC-1, AND WTC-1 AS SHOWN ON FIGURE 2, SHEET 3.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

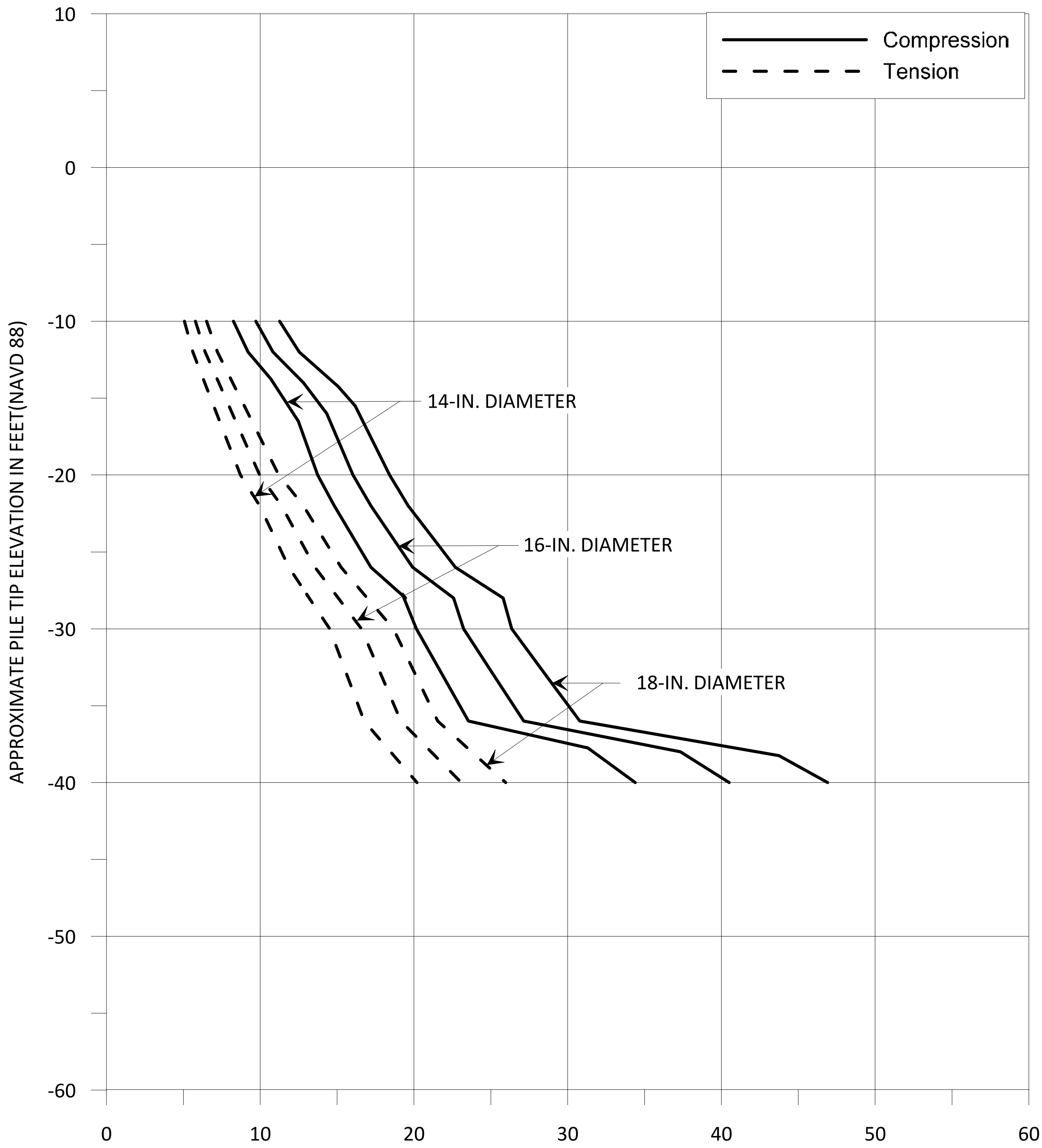
6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 8.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
OPEN END STEEL PIPE PILES
WATER TOWER, SCALE, GUARDHOUSE/ADMINISTRATION, AND POWER PLANT

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C.
SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 7 (SHEET 1 OF 6)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS WMB-1 AND WMB-2 AND CPTS WMC-1 THROUGH WMC-4 AS SHOWN ON FIGURE 2, SHEET 2.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

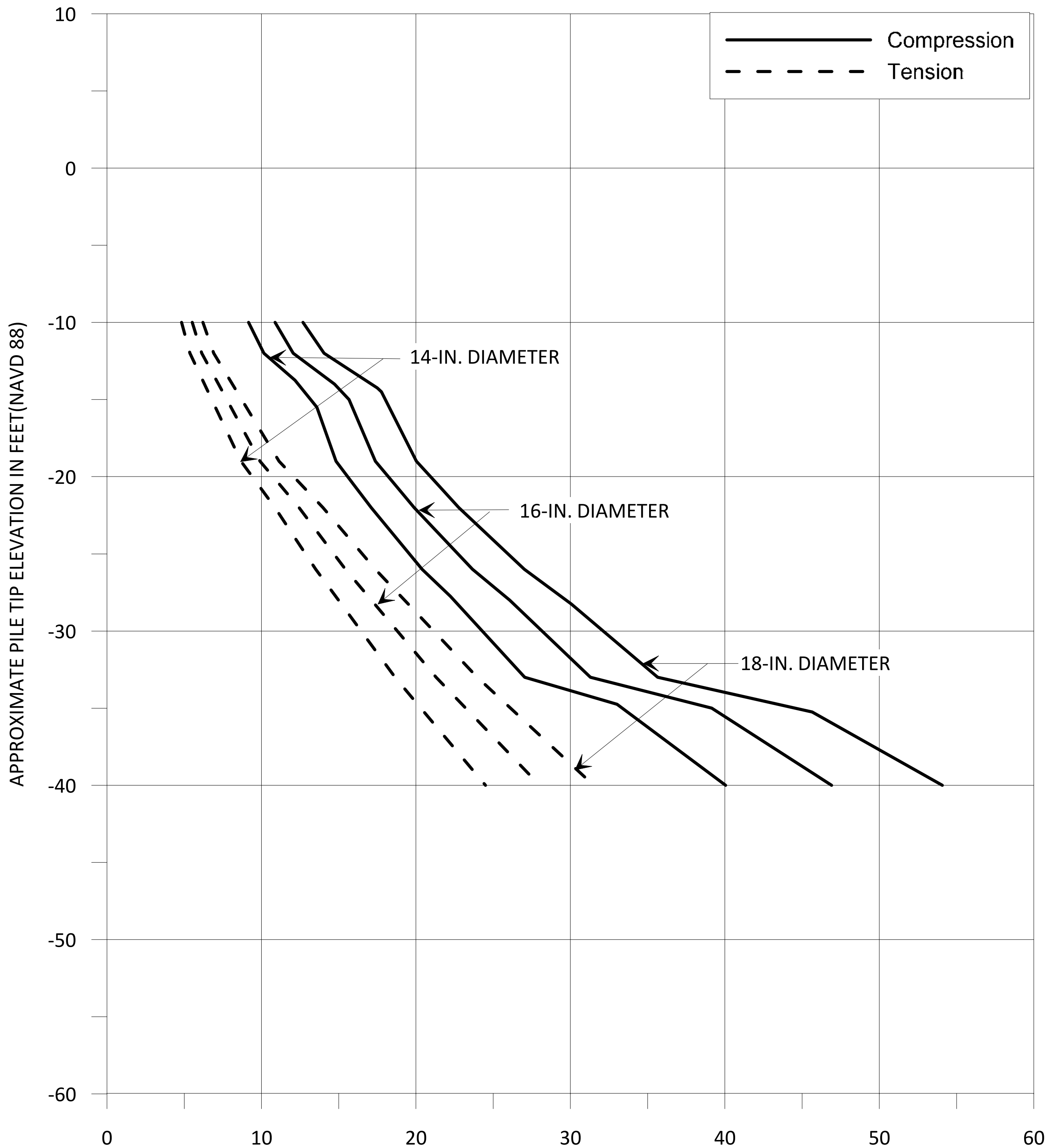
6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 8.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
OPEN END STEEL PIPE PILES
WAREHOUSE/MANUFACTURING AREA

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 7 (SHEET 2 OF 6)



ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY IN TONS
FACTOR OF SAFETY ≈ 2

NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORING WSB-1 AND CPTS WSC-1 AND WSC-2 AS SHOWN ON FIGURE 2, SHEET 4.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

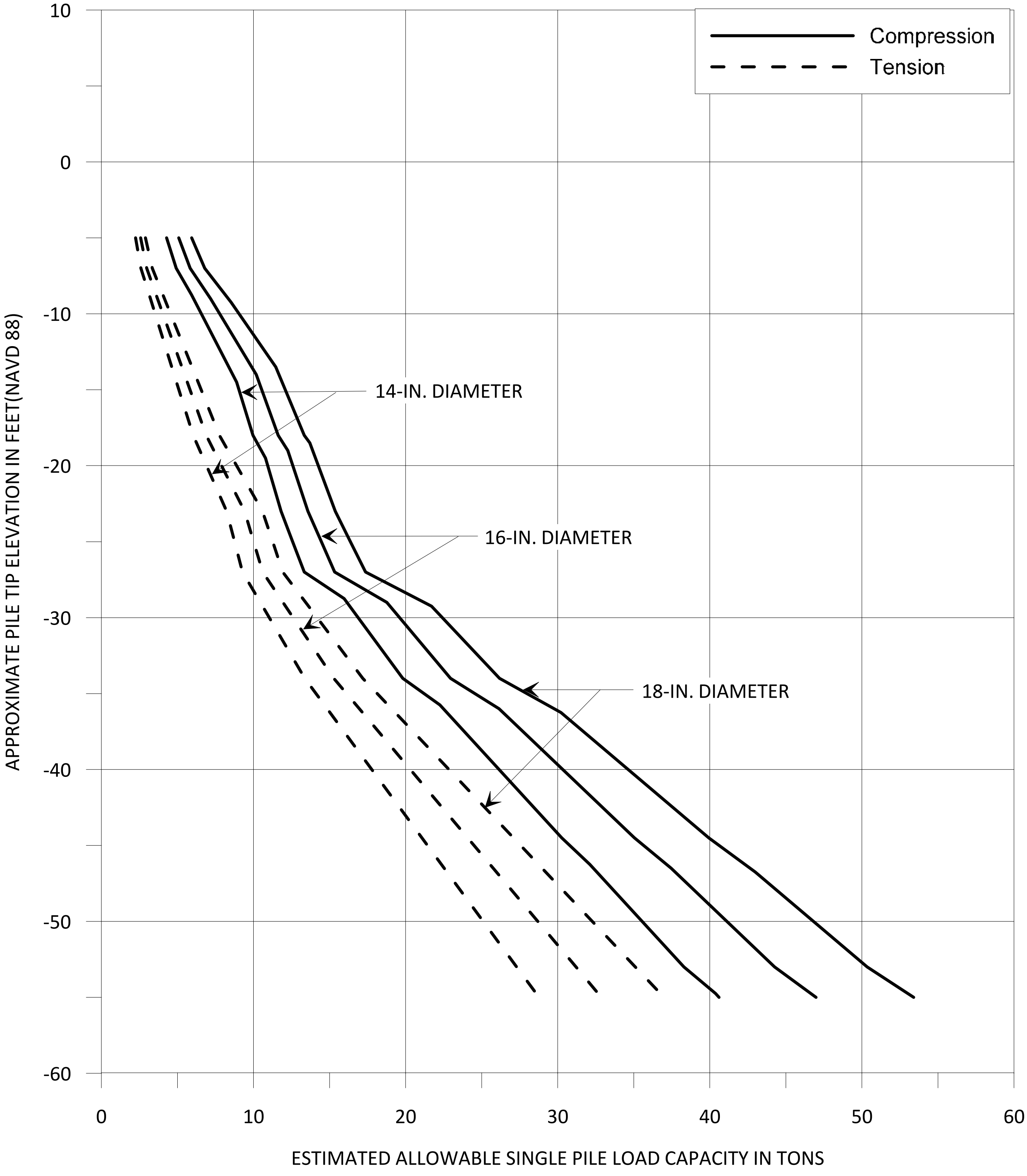
6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 10.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
OPEN END STEEL PIPE PILES
WAREHOUSE/STAGING AREA

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 7 (SHEET 3 OF 6)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS TFB-1 THROUGH TFB-4 AND CPTS TFC-1 THROUGH TFC-6 AS SHOWN IN FIGURE 2, SHEETS 5 AND 6.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 2 FEET FOR EMBEDMENT WITHIN THE PILE CAP.

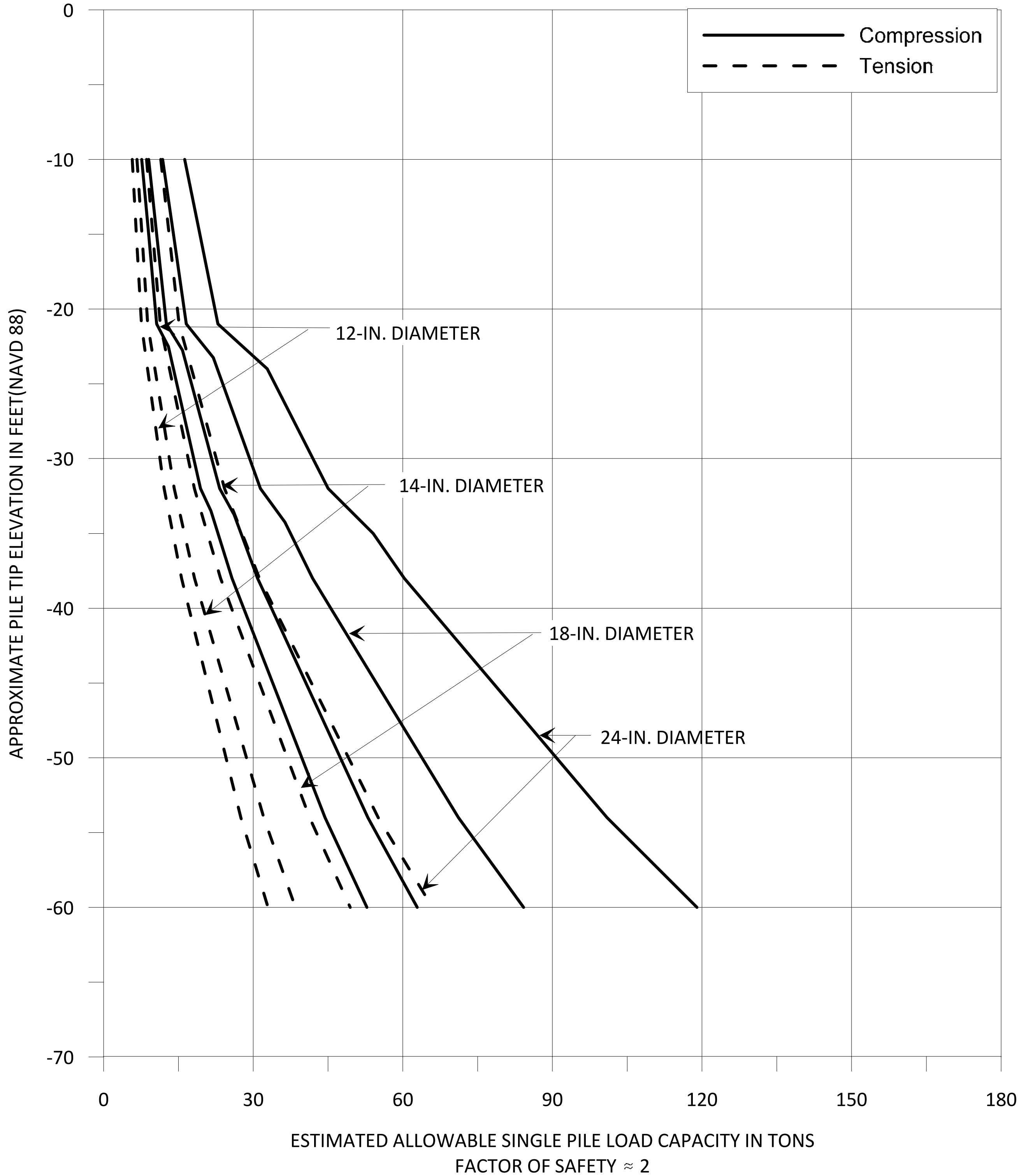
6. PILE CAPACITIES ASSUME A PILE CUTOFF AT EL 11.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
OPEN END STEEL PIPE PILES
TANK FARM AREA (EAST AND WEST)

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C.
SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 7 (SHEET 4 OF 6)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 2 ASSUMES A STATIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED IN LIEU OF STATIC PILE TESTING, A FACTOR OF SAFETY OF 2.5 SHOULD BE USED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS BSB-1 AND CSB-1 THROUGH CSB-3 AS SHOWN ON FIGURE 2, SHEET 1.

5. CAPACITIES NEGLECT FRICTION IN THE UPPER 7 FEET DUE TO TIEBACK FROM THE BULKHEAD.

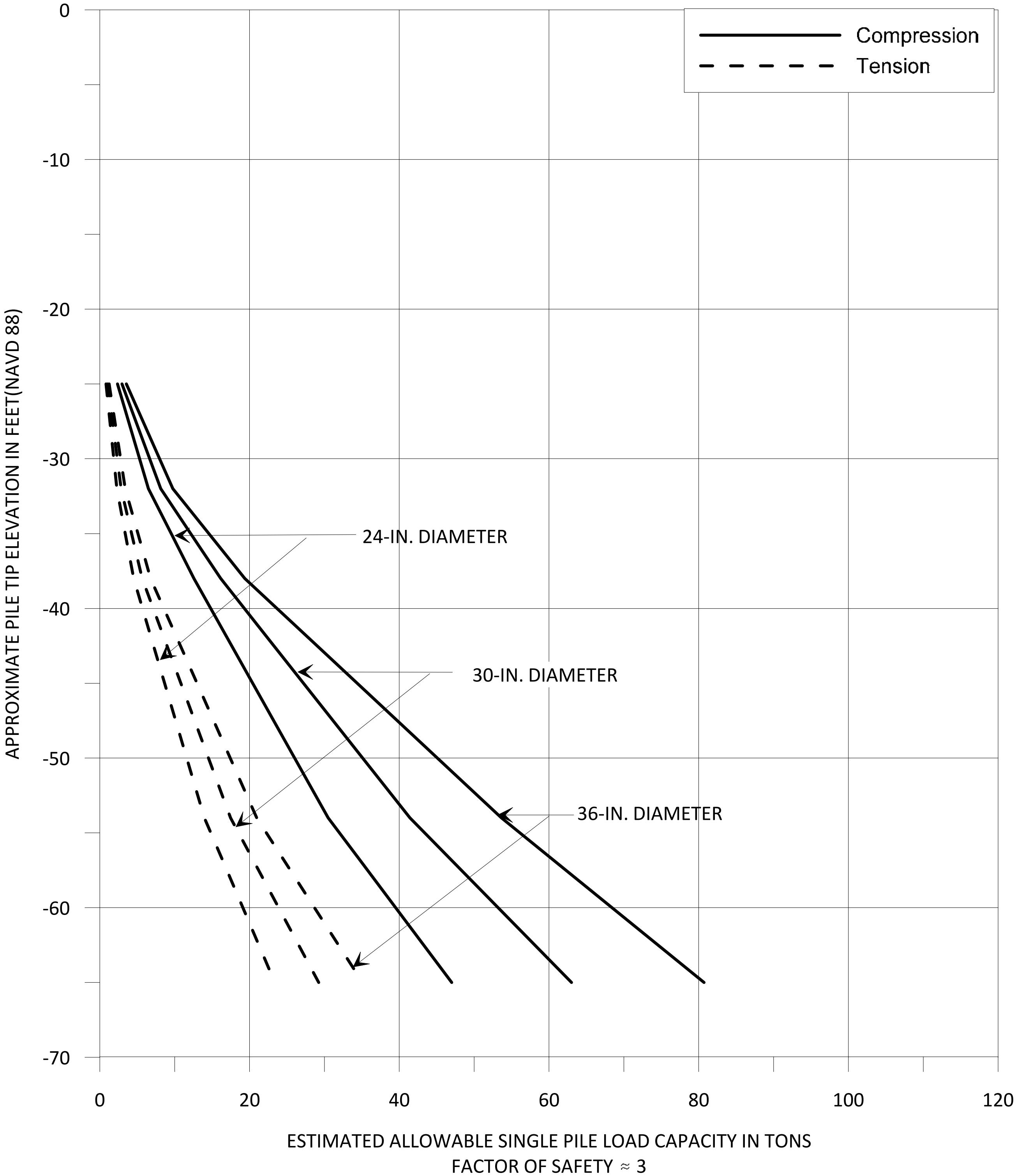
6. PILES CAPACITIES ASSUME A PILE CUTOFF AT EL 4.

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
OPEN END STEEL PIPE PILES
BARGE SLIP/CRUSHED STONE AREA, APRON, AND BULKHEAD ANCHORAGE

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946

DRAWN BY: H.C.W.	JOB NO.: G0389
CHECKED BY: B.M.C.	DATE: 27 DEC 2018
FILE NAME: SPC.GRF	FIGURE 7 (SHEET 5 OF 6)



NOTES:

1. ALLOWABLE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.

2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.

3. USE OF A FACTOR OF SAFETY OF 3 ASSUMES NEITHER A STATIC NOR DYNAMIC PILE LOAD TEST WILL BE PERFORMED. IF DYNAMIC PILE TESTING IS USED, A FACTOR OF SAFETY OF 2.5 MAY BE APPLIED.

4. CAPACITIES SHOWN ON THIS FIGURE DERIVED FROM STRATIGRAPHY AS SEEN IN BORINGS BSB-1 AND CSB-1 THROUGH CSB-3 AS SHOWN ON FIGURE 2, SHEET 1.

5. MONOPOLES ASSUMED TO BE INSTALLED AT A MUDLINE OF EL -16.

6. CAPACITIES NEGLECT FRICTION IN THE UPPER 5 FEET FOR SCOUR.

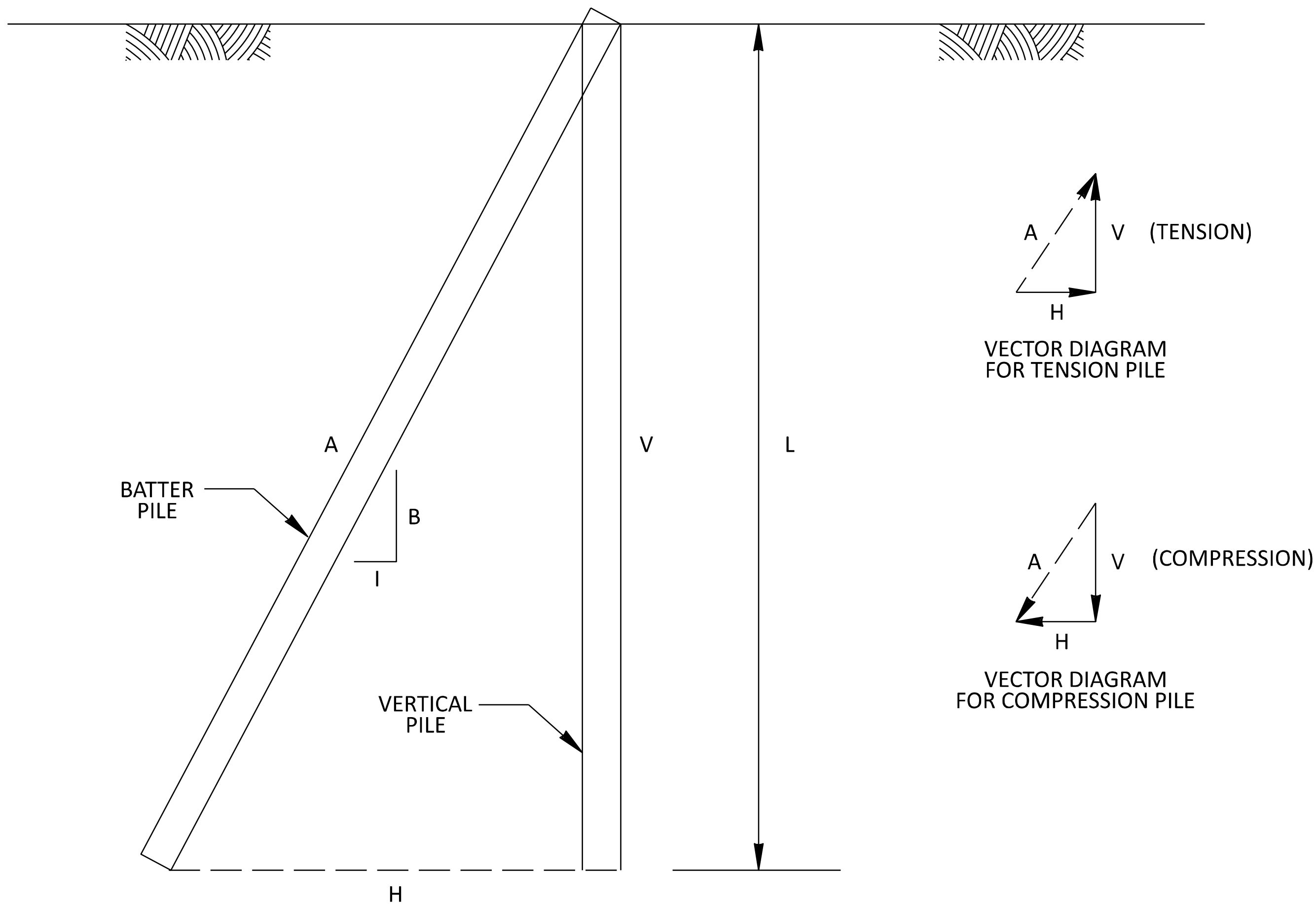
ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
OPEN END STEEL PIPE PILES
BARGE SLIP/CRUSHED STONE AREA, MOORING STRUCTURE

HANCOCK COUNTY PORT AND HARBOR COMMISSION
RESTORE PROJECT
PROPOSED FACILITY
PORT BIENVILLE, MISSISSIPPI

EUSTIS ENGINEERING L.L.C. SINCE 1946	DRAWN BY: H.C.W.	JOB NO.: G0389
	CHECKED BY: B.M.C.	DATE: 27 DEC 2018
	FILE NAME: SPC.GRF	FIGURE 7 (SHEET 6 OF 6)

EUSTIS ENGINEERING SINCE 1946

AXIAL AND HORIZONTAL RESISTANCE OF BATTER PILES ESTIMATED FROM ALLOWABLE VERTICAL LOAD CAPACITIES



L = VERTICAL COMPONENT OF BATTER PILE EMBEDMENT LENGTH.

V = ESTIMATED ALLOWABLE SINGLE LOAD CAPACITY OF A PILE DRIVEN VERTICALLY WITH EMBEDMENT LENGTH, L.

B = BATTER OF PILE EXPRESSED AS A RATIO OF VERTICAL DISTANCE TO ONE FOOT HORIZONTAL DISTANCE.

H = HORIZONTAL RESISTANCE OF BATTER PILE ESTIMATED AS FOLLOWS: $H = \frac{V}{B}$

A = ALLOWABLE AXIAL PILE LOAD CAPACITY OF A SINGLE BATTER PILE ESTIMATED AS FOLLOWS: $A = \sqrt{V^2\left(1+\frac{1}{B^2}\right)}$

NOTE: THE AXIAL LOAD RESISTANCE OF A VERTICAL PILE, V, IS DEPENDENT ON THE TYPE OF LOADING - TENSION OR COMPRESSION. CAUTION SHOULD BE EXERCISED TO ENSURE THE CORRECT VERTICAL CAPACITY IS USED.

CAPACITY OF PILE GROUPS

The maximum allowable load carrying capacity of a pile group is no greater than the sum of the single pile load capacities, but may be limited to a lower value if so indicated by the result of the following formula.

$$Q_a = \frac{P \ x \ L \ x \ c}{(FSF)} + \frac{2.6 \ q_u \ (1 + 0.2 \ \frac{w}{b}) \ A}{(FSB)}$$

In Which:

Q_a	=	Allowable load carrying capacity of pile group, lb
P	=	Perimeter distance of pile group, ft
L	=	Length of pile, ft
c	=	Average (weighted) cohesion or shear strength of material between surface and depth of pile tip, psf
q_u	=	Average unconfined compressive strength of material in the zone immediately below pile tips, psf (unconfined compressive strength = cohesion x 2)
w	=	Width of base of pile group, ft
b	=	Length of base of pile group, ft
A	=	Base area of pile group, sq ft
(FSF)	=	Factor of safety for the friction area = 2
(FSB)	=	Factor of safety for the base area = 3

The values of c and q_u used in this formula should be based on applicable soil data shown on the Log of Boring and Test Results for this report. In the application of this formula, the weight of the piles, pile caps and mats, considering the effect of buoyancy, should be included.

SPACING WITHIN PILE GROUPS

SPAC = $0.05 (L_1) + 0.025 (L_2) + 0.0125 (L_3)$

In Which:

SPAC	=	Center-to-center of piles, feet
L_1	=	Pile penetration up to 100 feet
L_2	=	Pile penetration from 101 to 200 feet
L_3	=	Pile penetration beyond 200 feet

NOTE: Minimum pile spacing = 3 feet or 3 pile diameters, whichever is greater

APPENDIX I

LEGEND AND NOTES FOR LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

PP Pocket penetrometer: Resistance in tons per square foot

SPT Standard Penetration Test: Number of blows of a 140-lb hammer dropped 30 inches required to drive 2-in. O.D., 1.4-in. I.D. sampler a distance of 1 foot into the soil after first seating it 6 inches. Values shown have not been corrected.

SPLR Type of Sampling Shelby SPT Auger Vibracore Geoprobe No sample

SYMBOL Clay Silt Sand Peat/Humus Shells Stone/Gravel

Predominant type shown heavy; modifying type shown light

USC Unified Soil Classification

DENSITY Unit weight in pounds per cubic foot

SHEAR TESTS

TYPE

UC Unconfined compression shear

OB Unconsolidated undrained triaxial compression shear on one specimen confined at the approximate overburden pressure

UU Unconsolidated undrained triaxial compression shear

ϕ Angle of internal friction in degrees

c Cohesion in pounds per square foot

ATTERBERG LIMITS

LL Liquid Limit

PL Plastic Limit

PI Plasticity Index

OTHER TESTS

CON Consolidation

-#200 Percent passing a U.S. No. 200 sieve

SV Particle size distribution (sieve only)

PD Particle size distribution (sieve and hydrometer)

k Coefficient of permeability in centimeters per second

SP Swelling pressure in pounds per square foot

Other laboratory test results reported on separate figures

GENERAL NOTES

(1) If a ground water depth is shown on the boring log, these observations were made at the time of drilling and were measured below the existing ground surface. These observations are shown on the boring logs. However, ground water levels may vary due to seasonal fluctuations and other factors. If important to construction, the depth to ground water should be determined by those persons responsible for construction immediately prior to beginning work.

(2) While the individual logs of borings are considered to be representative of subsurface conditions at their respective locations on the dates shown, it is not warranted that they are representative of subsurface conditions at other locations and times.

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: BSB-1

Project No: G0389
Date: 10/26/2018
Latitude: 30.23806°
Longitude: -89.56956°

Elevation: 6.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Medium dense brown fine sand w/roots & gravel	SP	PB-1	0	27										
	1.00				Medium stiff tan & light gray silty clay w/trace of concretions	CL	2	2											
5	1.00						3	5	42										
	2.00						4	8											
10																			
	0.50				Medium stiff tan & light gray sandy clay	CL	5	11	24										
					Medium dense tan & gray clayey sand	SC													
	1.00						6	14											
15					Medium dense tan fine sand	SP													
		11					PB-7	15.5	29										
		14					PB-8	18.5											
20																			
		17			w/few clay pockets		PB-9	21.5	23										-#200 = 7.7%
		21					PB-10	24.5											
25																			
		18			w/few silt, trace of clay pockets, concretions, & organic matter		PB-11	28.5	23										-#200 = 7.7%
30																			
		16					PB-12	33.5											
35																			
		40			Dense gray fine sand	SP	PB-13	38.5	21										
40																			
		41			Dense gray silty sand w/trace of clay pockets & organic matter	SM	PB-14	43.5	19										-#200 = 20.2%
45																			
		63			Very dense brown silty sand w/gravel	SM	PB-15	48.5											
50																			

NOTES:

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: BSB-1

Project No: G0389
Date: 10/26/2018
Latitude: 30.23806°
Longitude: -89.56956°

Elevation: 6.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50					Very dense brown silty sand	SM													
55		59					PB-16	53.5											
60		48			Dense gray silty sand w/coarse to fine gravel	SM	PB-17	58.5	16										SV
65		53			Very dense tan & gray fine sand w/trace of coarse to fine sand	SP	PB-18	63.5											
70		70			Very dense gray fine sand w/trace of gravel	SP	PB-19	68.5											
75		72					PB-20	73.5	14										
80		68			Very dense gray & tan fine sand w/trace of gravel	SP	PB-21	78.5											
85																			
90																			
95																			
100																			

NOTES:

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LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: CSB-1

Project No: G0389
Date: 10/23/2018
Latitude: 30.23939°
Longitude: -89.56839°

Elevation: 7.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Loose gray & tan clayey sand w/silty sand	SC-SM													
	2.75						1	2	11						19	12	7		-#200 = 43.0%
5					Stiff tan & light gray silty clay w/trace of fine sand, concretions, & organic matter	CL	2	5	21	103	125	UC	--	1244					
	1.75				Medium stiff tan & light gray clay w/trace of silty clay lenses, silty sand pockets, & roots	CH	3	8	33	87	116	UC	--	747					
10	1.75						4	11	41	77	108	UC	--	573					
					Loose tan & gray silty sand	SM	PB-5	14	24										-#200 = 14.5%
15		13			Medium dense tan & gray fine sand w/silt	SP-SM	PB-6	15.5	23										-#200 = 11.3%
		11			Medium dense tan & gray silty sand	SM	PB-7	18.5											
20		11					PB-8	21.5											
25		23					PB-9	24.5	22										
30		28					PB-10	28.5											
		31			Dense tan & gray silty sand	SM	PB-11	33.5	22										
35		39			Dense tan fine sand w/trace of silt & clay pockets	SP	PB-12	38.5	20										-#200 = 7.1%
40		38					PB-13	43.5											
45		65			Very dense tan coarse to fine sand w/coarse to fine gravel & trace of silt	SP	PB-14	48.5	11										SV
50																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: CSB-1

Project No: G0389
Date: 10/23/2018
Latitude: 30.23939°
Longitude: -89.56839°

Elevation: 7.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50					Very dense tan coarse to fine sand w/coarse to fine gravel & trace of silt	SP													
		64					PB-15	53.5											
55																			
		76					PB-16	58.5	14										
60																			
		74					PB-17	63.5											
65																			
		31			Dense tan coarse to fine sand w/few gravel & trace of silt	SP	PB-18	68.5	13										
70																			
		37					PB-19	73.5											
75																			
		40					PB-20	78.5											
80																			
85																			
90																			
95																			
100																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: CSB-2

Project No: G0389
Date: 10/24/2018
Latitude: 30.23853°
Longitude: -89.56944°

Elevation: 7.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0			Loose brown clayey silt w/roots	ML	PB-1	0	14										
	3.00		Loose brown & tan clayey sandy silt w/trace of fine gravel, organic matter, & concretions	ML	2	2	20										-#200 = 56.2%
5	1.25		Stiff tan sandy clay w/trace of roots	CL	3	5	17	115	135	UC	--	1141					ORG = 3.6%
	1.50		Very stiff tan sandy clay w/fine sand pockets	CL	4	8	17	107	125	UC	--	2078					
10	1.25		Stiff light gray & tan silty clay w/fine sand pockets	CL	5	11											
	1.50				6	14	25	99	123	UC	--	1628					
15		5	Loose tan & light gray silty sand w/trace of clay	SM	PB-7	15.5	26										
		6			PB-8	18.5	24										-#200 = 12.2%
20		12	Medium dense tan silty sand	SM	PB-9	21.5											
25		12			PB-10	24.5											
		35	Stiff gray silty clay w/few fine sand pockets	CL	PB-11	28.5	27						46	14	32		
30		38	Dense tan silty sand w/trace of clay pockets	SM	PB-12	33.5	22										-#200 = 22.2%
35		76	Very dense tan fine sand w/gravel	SP	PB-13	38.5	18										
40		74			PB-14	43.5											
45		67			PB-15	48.5											
50																	

NOTES:

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LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: CSB-2

Project No: G0389
Date: 10/24/2018
Latitude: 30.23853°
Longitude: -89.56944°

Elevation: 7.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50					Very dense tan fine sand w/gravel	SP													
55		59			Very dense tan coarse to fine sand w/trace of fine gravel & silt	SP	PB-16	53.5	15										
60		27			Medium dense tan coarse to fine sand w/trace of fine gravel	SP	PB-17	58.5											
65		36			Dense tan coarse to fine sand w/trace of fine gravel	SP	PB-18	63.5	13										
70		35					PB-19	68.5											
75		17			Medium dense tan fine sand	SP	PB-20	73.5											
80		17					PB-21	78.5	19										
85																			
90																			
95																			
100																			

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NOTES:

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: CSB-3

Project No: G0389
Date: 10/29/2018
Latitude: 30.23844°
Longitude: -89.56881°

Elevation: 6.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 25.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Medium stiff tan sandy clay w/trace of silty sand pockets & concretions	CL	NS	0											
	1.00						1	2	19	101	120	UC	--	683					
5	0.50				Soft gray & reddish-brown clay w/trace of silt pockets	CH	2	5	36	86	117	OB	0	471					
	1.00				Medium dense greenish-gray clayey sand w/silty sand & sandy clay pockets	SC	3	8	23	98	121	OB	0	695					
10							NS	11											
		9			Loose to medium dense tan & light gray silty sand w/trace of clay pockets	SM	PB-4	12.5	30										
15		11					PB-5	15.5											
		30			Dense tan & light gray fine sand w/silt	SP-SM	PB-6	18.5	24										
20		25			Medium dense tan & light gray fine sand w/silt	SP-SM	PB-7	21.5											
		34			Dense tan & light gray fine sand w/trace of silt & roots	SP	PB-8	23.5	22										
25																			
30																			
35																			
40																			
45																			
50																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

LOG OF BORING AND TEST RESULTS

Boring: GHB-1

Project No: G0389
Date: 11/06/2018
Latitude: 30.23900°
Longitude: -89.56372°

Elevation: 8.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Soft brown silty clay w/gravel, fine sand, roots, & vegetation	CL	PB-1	0	31										
	3.50				Stiff tan & gray silty clay w/fine sand pockets & trace of concretions (brittle)	CL	2	2	15	112	129	UC	--	1781					
5	3.50						3	5	21	104	126	UC	--	1764					
	3.50						4	8											
10	3.25				Medium stiff gray & tan clay w/few silt pockets & concretions (flocculated)	CH	5	11	30	88	114	UC	--	949					
					Loose tan & light gray clayey sand	SC	PB-6	12	33										
15		17			Medium dense tan fine to medium sand	SP	PB-7	15.5											
		10					PB-8	18.5											
20		12					PB-9	21.5	23										
25		41			Dense tan fine sand	SP	PB-10	24.5											
		2			Very soft dark gray & brown organic clay w/fine sand pockets & decayed wood	OH	PB-11	28.5	113						186	77	109		
30	0.50				Medium stiff dark gray clay w/decayed wood	CH	12	33	70	56	95	UC	--	936					
35	0.50				Medium stiff light gray silty clay	CL	13	38	21	100	121	UC	--	864					
40		72			Very dense light gray fine to medium sand w/trace of gravel	SP	PB-14	41.5											
45		68					PB-15	44.5	16										
		100					PB-16	47.5											
50																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: GHB-1

Project No: G0389
Date: 11/06/2018
Latitude: 30.23900°
Longitude: -89.56372°

Elevation: 8.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50		64			Very dense light gray fine to medium sand w/trace of gravel	SP	PB-17	50.5											
		49					PB-18	53.5	13										
55																			
		62					PB-19	58.5											
60																			
		63					PB-20	63.5	15										
65																			
		63					PB-21	68.5											
70																			
		3			Stiff gray clay w/trace of silt pockets (flocculated)	CH	PB-22	73.5	41						67	19	48		
75																			
	3.75						23	78	44	76	109	UC	--	1131					
80																			
85																			
90																			
95																			
100																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: RDB-1

Project No: G0389
Date: 10/26/2018
Latitude: 30.23688°
Longitude: -89.56618°

Elevation: 10.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 25.0 ft

Scale in Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	LL	PL	PI		Other Tests
0			Soft brown silty clay w/trace of roots & vegetation	CL	PB-1	0	22										
	1.75		Stiff light gray & tan silty clay w/some fine sand pockets (brittle)	CL	2	2	13	103	117	UC	--	1209					
5	1.50		w/trace of concretions & roots		3	5	23	101	124	UC	--	1561					
			Medium dense tan & reddish-brown silty sand w/trace of clay pockets	SM	4	8											
10		19			PB-5	9.5	18										
		18			PB-6	12.5											
15		21	Medium dense tan & reddish-brown clayey sand	SC	PB-7	15.5	19										
		12			PB-8	18.5											
20			Dense tan fine sand	SP													
25		39			PB-9	23.5	23										
30																	
35																	
40																	
45																	
50																	

NOTES:

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LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: RDB-2

Project No: G0389
Date: 10/24/2018
Latitude: 30.23367°
Longitude: -89.56397°

Elevation: 12.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 25.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density: Dry pcf	Density: Wet pcf	Shear Tests: Type	Shear Tests: ϕ	Shear Tests: C psf	Atterberg Limits: LL	Atterberg Limits: PL	Atterberg Limits: PI		Other Tests
0					Loose tan & light brown clayey sand	SC	PB-1	0	16										
	0.25				Loose tan silty sand w/trace of clay	SM	2	2	13										
					Medium dense tan silty sand	SM	PB-3	3.5	13										
5		12																	
		15					PB-4	6.5											
10		11					PB-5	9.5											
					Medium dense tan fine sand w/silt, trace of gravel, & organic matter	SP-SM	PB-6	13.5	22										
15		12																	
							PB-7	18.5											
20		15																	
							PB-8	23.5											
25		14																	
30																			
35																			
40																			
45																			
50																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: TFB-1

Project No: G0389
Date: 10/29/2018
Latitude: 30.23633°
Longitude: -89.56352°

Elevation: 11.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0			Soft brown silty clay w/fine sand pockets & roots	CL	PB-1	0	35						32	21	11		
	0.50		Medium stiff brown sandy clay	CL	2	2	16	113	131	OB	0	535					
			Stiff light gray, tan, & reddish-brown sandy clay (brittle)	CL													
5	4.25				3	5	18	106	125	UC	--	1114					
			Medium dense reddish-brown silty sand & trace of clay pockets	SM													
	1.25				4	8	20										
10		14	Medium dense tan fine sand w/trace of silt	SP	PB-5	9.5	24										
		17			PB-6	12.5											
15		19			PB-7	15.5											
20		21	w/trace of medium sand		PB-8	18.5	23										-#200 = 6.3%
25		25			PB-9	23.5											
30		27	Medium dense tan & gray silty sand	SM	PB-10	28.5											
35		29			PB-11	33.5	23										
40		3	Soft gray clay	CH	PB-12	38.5											
	1.25		Stiff gray clay w/few fine sand pockets (fissured, flocculated)	CH	13	43	58	63	99	UC	--	1315	104	20	84		CONS
45																	
50	1.00				14	48											

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: TFB-1

Project No: G0389
Date: 10/29/2018
Latitude: 30.23633°
Longitude: -89.56352°

Elevation: 11.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50			Stiff gray clay w/few fine sand pockets (fissured, flocculated)	CH													
			Loose gray silty sand	SM	PB-15	53	25										
55			Dense light gray & gray fine sand w/silt & clay pockets	SP-SM	PB-16	55.5											
		35															
		37			PB-17	58.5	23										-#200 = 9.7%
60		41			PB-18	61.5	22										-#200 = 16.4%
65		46			PB-19	64.5											
70		49			PB-20	68.5											
75		76	Very dense gray medium to fine sand w/trace of gravel, organic matter, & wood	SP	PB-21	73.5	21										
80		72=8"			PB-22	78.5											
85																	
90																	
95																	
100																	

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

EUSTIS
ENGINEERING
SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

LOG OF BORING AND TEST RESULTS

Boring: TFB-2

Project No: G0389
Date: 10/30/2018
Latitude: 30.23589°
Longitude: -89.56400°

Elevation: 11.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density: Dry pcf	Density: Wet pcf	Shear Tests: Type	Shear Tests: ϕ	Shear Tests: C psf	Atterberg Limits: LL	Atterberg Limits: PL	Atterberg Limits: PI		Other Tests
0					Soft brown silty clay w/trace of roots	CL	PB-1	0	24										
	1.00				Medium dense tan silty sand w/few clay pockets	SM	2	2	12	117	131	OB	0	1724					
5	1.50						3	5	14	110	126	OB	0	919					-#200 = 43.5%
					Stiff tan, reddish-brown, & gray sandy clay w/few concretions	CL													
	2.00						4	8	17	109	128	UC	--	1194					
					Medium dense tan & brown silty sand w/few clay pockets	SM													
10	1.00						5	11	18										-#200 = 17.7%
	1.50						6	14	16										
15		8			Loose tan & brown fine sand w/few clay pockets, trace of silt, & medium sand	SP-SC	PB-7	15.5											
20		10					PB-8	18.5	24										-#200 = 7.2%
		35			Dense tan fine sand	SP	PB-9	21.5											
25		35					PB-10	24.5	23										
	0.50				Medium stiff dark gray organic clay w/few decayed wood	OH	11	28	104	42	86	UC	--	616					
30																			
	0.50				Medium stiff gray silty clay w/decayed wood & few fine sand pockets	CL	12	33	23	100	123	OB	0	664					
35																			
	0.50				Loose dark gray clayey sand w/trace of roots	SC	13	38	31	90	117	UC	--	284					-#200 = 32.5
40																			
	0.50				Medium stiff gray clay w/trace of fine sand pockets & layers	CH	14	43	43	72	103	OB	0	576					
45																			
	1.00				Stiff gray clay w/trace of silt pockets	CH	15	48	60	62	99	UC	--	1310					
50																			

NOTES:

LOG OF BORING AND TEST RESULTS

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: TFB-2

Project No: G0389
Date: 10/30/2018
Latitude: 30.23589°
Longitude: -89.56400°

Elevation: 11.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50			Stiff gray clay w/trace of silt pockets	CH													
	1.00		Medium stiff gray clay	CH	16	53	61	62	100	UC	--	674					
55					17	57											
		49	Dense gray fine sand	SP	PB-18	58.5											
60		51			PB-19	61.5	23										
65		70	Very dense tan fine sand	SP	PB-20	64.5											
70		67			PB-21	68.5	18										
75		68			PB-22	73.5	16										
80		67			PB-23	78.5											
85																	
90																	
95																	
100																	

NOTES:

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LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: TFB-3

Project No: G0389
Date: 11/02/2018
Latitude: 30.23606°
Longitude: -89.56258°

Elevation: 11.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Other Tests
		Medium stiff brown silty clay w/few silty sand pockets & vegetation	CL	PB-1	0	37						48	14	34	
0.25		Medium dense tan & gray clayey sand w/trace of concretions & roots	SC	2	2	12	125	140	OB	0	2273				
0.50		Stiff light gray & tan sandy clay	CL	3	5	19	109	129	UC	--	1281				
1.50		Medium dense light gray & tan clayey sand	SC	4	8	15	111	128	UC	--	1879				
1.00		Loose to medium dense tan & gray silty sand w/trace of clay	SM	5	11	17									
	7	Loose tan fine sand w/silt	SP-SM	PB-6	12.5										
	9			PB-7	15.5	25									
	9			PB-8	18.5										
	15	Medium dense tan fine sand	SP	PB-9	21.5										
	17			PB-10	23.5	26									
	23			PB-11	28.5										
1.00		Soft gray sandy clay w/few decayed wood	CL	12	33	30	88	114	UC	--	407				
0.50		Loose to medium dense silty sand w/trace of clay pockets & decayed wood	SM	13	38	28									-#200 = 16.4%
	28	Medium dense gray clayey sand	SC	PB-14	40.5	27									
	26	Medium dense gray silty sand	SM	PB-15	43.5	22									
	10	Medium dense gray clayey sand	SC	PB-16	46.5										
	23	w/clay layers		PB-17	48.5	31									

NOTES:

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: TFB-3

Project No: G0389
Date: 11/02/2018
Latitude: 30.23606°
Longitude: -89.56258°

Elevation: 11.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50			Medium dense gray clayey sand w/clay layers	SC													
			Soft gray sandy clay	CL													
55		3			PB-18	53.5	41						46	22	24		
			Stiff gray clay w/trace of silty sand pockets (flocculated)	CH													
	1.00				19	58	44	74	106	UC	--	1263	61	18	43		
60																	
	1.00				20	63											
65																	
	1.00				21	68	41	80	113	UC	--	1104					
70																	
	1.25				22	73											
75																	
			Medium stiff gray clay w/trace of silt pockets (flocculated)	CH													
	1.00				23	78	46	73	107	UC	--	921					
			Medium dense gray fine sand	SP													
80																	
85																	
90																	
95																	
100																	

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

LOG OF BORING AND TEST RESULTS

Boring: TFB-4

Project No: G0389
Date: 11/01/2018
Latitude: 30.23536°
Longitude: -89.56328°

Elevation: 11.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Other Tests
0			Medium dense gray & brown fine sand w/organic matter & roots	SP	PB-1	0	35									
	0.50		Medium dense tan silty sand w/trace of organic matter	SM	2	2	14	114	129	OB	0	1367				
5	0.75		Soft tan & light gray sandy clay	CL	3	5	18	108	127	UC	--	334				
	1.50		Loose gray clayey sand w/trace of decayed wood, roots, & organic matter	SC	4	8	17									-#200 = 42.1%
10		21	Medium dense tan & reddish-brown clayey sand	SC	PB-5	9.5	17									
		19	Medium dense tan & light gray silty sand	SM	PB-6	12.5										
15		15			PB-7	15.5	20									
20		11	Loose to medium dense tan fine sand w/silt & trace of shell fragments	SP-SM	PB-8	18.5	26									-#200 = 6.9%
25		8			PB-9	23.5										
30		34	Dense tan & gray silty sand	SM	PB-10	28.5	23									
35	0.50		Medium dense gray clayey sand	SC	11	33	23	104	128	OB	0	509				
	1.00		Loose gray clayey sand w/trace of decayed wood, roots, & organic matter	SC	12	38	31	93	121	OB	0	447				-#200 = 25.8%
40		19	Medium dense gray silty sand w/trace of clay pockets	SM	PB-13	39.5	24									
		18			PB-14	42.5										
45		23			PB-15	45.5										
50		7	Medium stiff gray clay w/silty sand pockets	CH	PB-16	48.5	40									

NOTES: 3 inches of standing water at the location of TFB-4.

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: TFB-4

Project No: G0389
Date: 11/01/2018
Latitude: 30.23536°
Longitude: -89.56328°

Elevation: 11.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50					Medium stiff gray clay w/silty sand pockets	CH													
		9					PB-17	53.5											
55																			
	1.00				w/trace of shell fragments		18	58	41	80	113	UC	--	856					
60																			
	1.25				w/trace of silt pockets		19	63	47	74	108	UC	--	1019					
65																			
	1.50				w/trace of silt pockets (flocculated)		20	68	46	74	108	UC	--	1686					
70																			
	1.50						21	73											
75																			
	1.00						22	78	47	75	110	UC	--	1510					
80																			
85																			
90																			
95																			
100																			

NOTES: 3 inches of standing water at the location of TFB-4.

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: WMB-1

Project No: G0389
Date: 10/29/2018
Latitude: 30.23853°
Longitude: -89.56760°

Elevation: 8.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 25.0 ft

Scale in Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	ϕ	C psf	LL	PL	PI	Other Tests
0			Medium stiff tan sandy clay w/roots & organic matter	CL	PB-1	0										
	1.25		Soft tan sandy clay w/trace of clayey sand pockets, concretions, shell fragments, & roots	CL	2	2	15	110	126	UC	--	449				
5			Medium dense tan & gray silty sand w/few clay pockets	SM	3	5	23									
		18			PB-4	6.5										
			Very loose gray & tan clayey silty sand	SC-SM	PB-5	9.5	30									
10		WOH														
			Medium stiff gray & tan sandy clay w/few fine sand pockets, roots, & trace of clay lenses	CL	6	11	21	100	121	UC	--	831	54	21	33	
			Medium dense tan & greenish-gray silty sand w/few clay pockets	SM	7	13	20									
15		16			PB-8	15.5										
		19			PB-9	18.5	20									-#200 = 13.9%
20			Dense tan fine sand w/silt, trace of medium sand, & clay pockets	SP-SM												
		32			PB-10	21.5	24									-#200 = 6.6%
		31			PB-11	23.5										
25																
30																
35																
40																
45																
50																

NOTES:

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: WMB-2

Project No: G0389
Date: 10/31/2018
Latitude: 30.23808°
Longitude: -89.56683°

Elevation: 7.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
		Soft brown silty clay w/trace of organic matter, roots, & gravel	CL	PB-1	0	43										
3.00		Very stiff tan silty clay w/few fine sand pockets & trace of concretions	CL	2	2	14	116	132	UC	--	3242					
1.50		Stiff tan & reddish-brown silty clay w/trace of concretions	CL	3	5	17	108	126	UC	--	1893					
1.50		Stiff light gray & tan silty clay	CL	4	8	23	104	128	UC	--	1012					
0.50		Medium stiff tan & light gray clay w/trace of silt pockets	CH	5	11	30	93	121	OB	0	557	65	19	46		CONS
1.50		Medium stiff gray & tan silty clay w/trace of fine sand pockets	CL	6	14	41	80	113	UC	--	531					
	20	Medium dense tan clayey sand w/trace of shell fragments & organic matter	SC	PB-7	15.5	22										-#200 = 21.4%
	15			PB-8	18.5											
	26	Medium dense tan fine sand	SP	PB-9	21.5	23										
	26			PB-10	23.5											
	20			PB-11	28.5	23										
0.75		Medium stiff gray clay w/trace of silt pockets & organic matter	CH	12	33	23	103	126	UC	--	680					
0.50				13	38	24	102	127	OB	0	626					
	70	Very dense tan fine sand	SP	PB-14	41.5											
	75			PB-15	44.5	21										
	68			PB-16	47.5											

NOTES:

LOG OF BORING AND TEST RESULTS

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: WMB-2

Project No: G0389
Date: 10/31/2018
Latitude: 30.23808°
Longitude: -89.56683°

Elevation: 7.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 80.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50		70			Very dense tan fine sand	SP	PB-17	50.5	21										
55		71					PB-18	53.5											
60		76			w/trace of gravel		PB-19	58.5	20										
65		72					PB-20	63.5											
70		66					PB-21	68.5											
75		64					PB-22	73.5	18										
80		67					PB-23	78.5											
85																			
90																			
95																			
100																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: WSB-1

Project No: G0389
Date: 10/29/2018
Latitude: 30.23738°
Longitude: -89.56428°

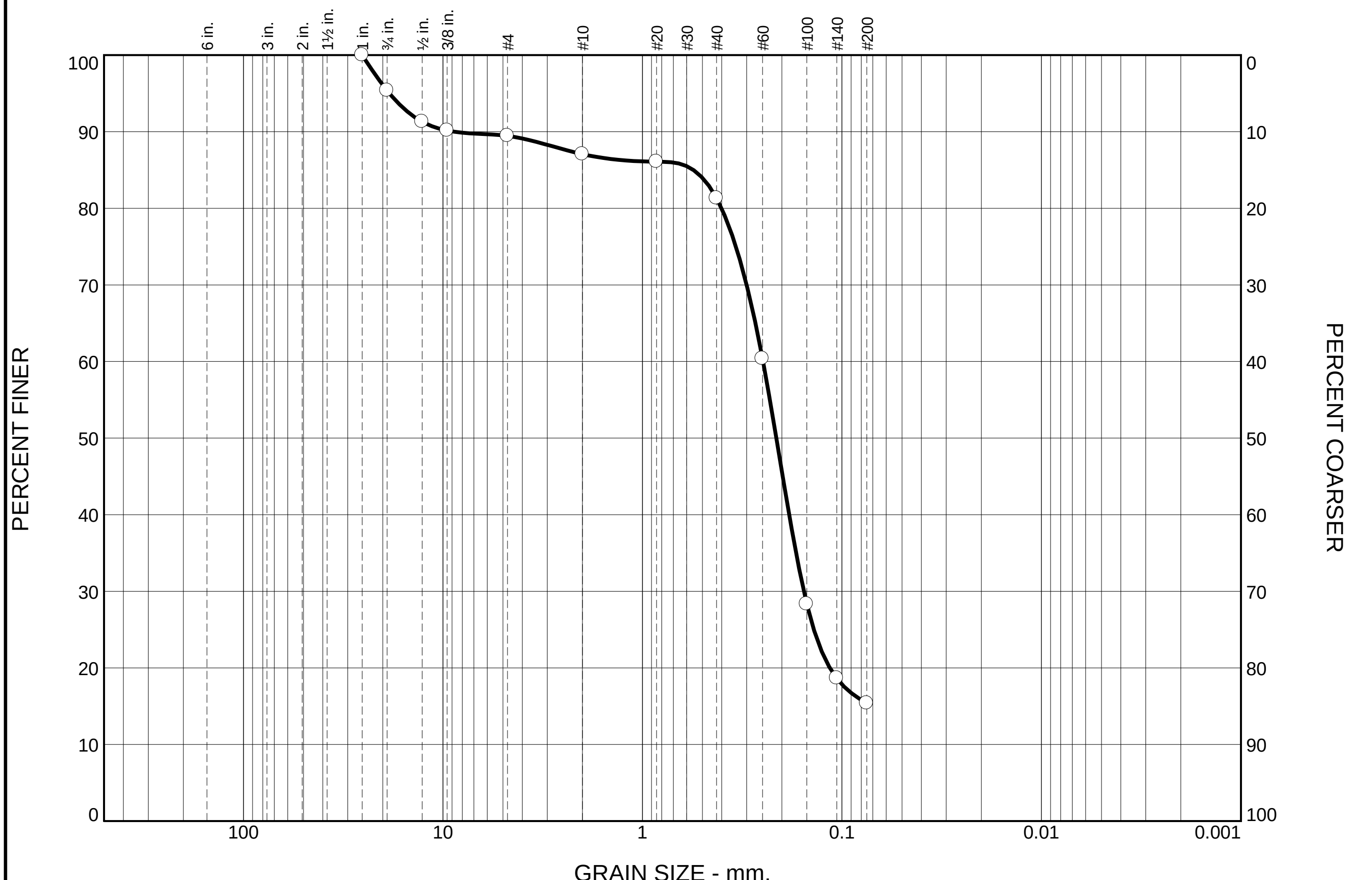
Elevation: 10.0
Datum: NAVD88
Water Depth: See Text
Total Depth: 25.0 ft

PP	SPT	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
		Medium stiff brown silty clay w/fine sand pockets, roots, organic matter, & trace of shell fragments	CL	PB-1	0	50										
1.25		Medium stiff brown sandy clay w/silty sand pockets & trace of roots	CL	2	2	15	113	129	UC	--	718					
		Medium dense gray silty sand w/clay pockets & roots	SM	3	5	11										-#200 = 39.9%
		Medium dense tan & reddish-brown silty sand w/clay pockets	SM	4	8	16										
		Medium dense tan & reddish-brown fine sand w/few clay pockets	SP	5	11	18										
	22			PB-6	12.5											
	21			PB-7	15.5											
	20	Medium dense tan silty sand w/clay pockets	SM	PB-8	18.5	23										
	17			PB-9	20.5											
	32	Dense tan fine sand w/silt & trace of medium sand	SP-SM	PB-10	23.5	20										

Scale in Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

NOTES:

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	4.6	6.0	2.3	5.8	65.9	15.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.5546	0.2484	0.2131	0.1552				

Material Description	USCS	AASHTO
○ Dense Gray Silty Sand W/ Coarse to Fine Gravel	SM	

Project No. G0389 **Client:** COMPTON ENGINEERING, INC., BAY ST. LOUIS,

Project: HANCOCK COUNTY PORT AND HARBOR COMMISSION - RESTORE PROJECT, PROPOSED FACILITY,

○ **Source of Sample:** BSB-1 **Depth:** 58.5 **Sample Number:** PB-17

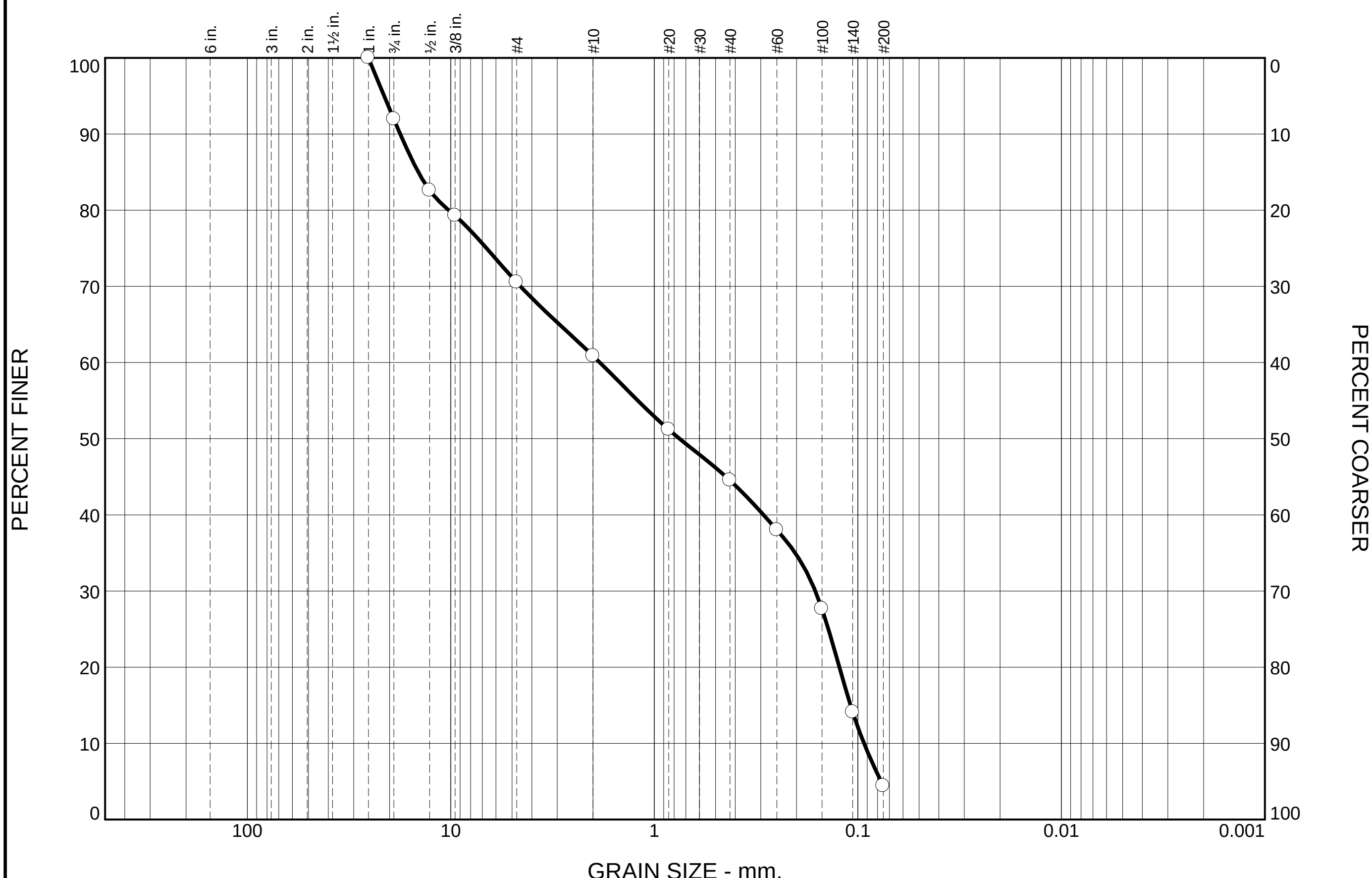
EUSTIS ENGINEERING SINCE 1946

Remarks:

Figure

Tested By: BH **Checked By:** CD

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	8.0	21.5	9.6	16.4	40.1	4.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			14.4413	1.8527	0.7519	0.1618	0.1087	0.0933	0.15	19.86

Material Description	USCS	AASHTO
○ Very Dense Tan Coarse to Fine Sand W/ Coarse to Fine Gravel, Trace of Silt	SP	

Project No. G0389 **Client:** COMPTON ENGINEERING, INC., BAY ST. LOUIS,
Project: HANCOCK COUNTY PORT AND HARBOR COMMISSION - RESTORE PROJECT, PROPOSED FACILITY,
○ **Source of Sample:** CSB-1 **Depth:** 48.5 **Sample Number:** PB-14

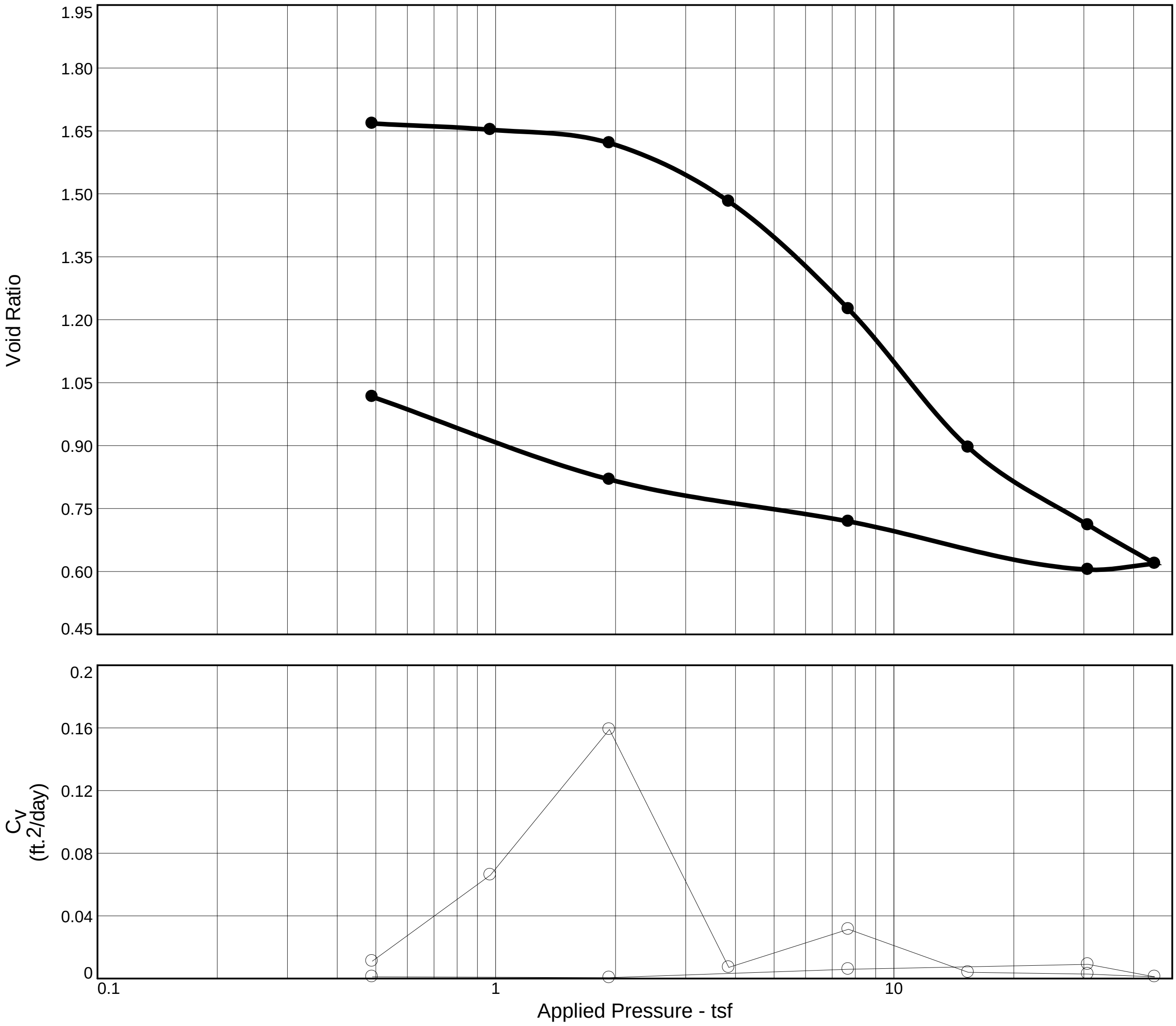
EUSTIS ENGINEERING SINCE 1946

Remarks:

Figure

Tested By: BH **Checked By:** CD

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture							
98.1 %	60.3 %	63.6	104	84	2.72	4.1	1.17	1.672

MATERIAL DESCRIPTION	USCS	AASHTO
ST G CL W/ FEW FISA POC (FISS, FLOC)	CH	

Project No. G0389 **Client:** COMPTON ENGINEERING, INC., BAY ST. LOUIS,

Project: HANCOCK COUNTY PORT AND HARBOR COMMISSION - RESTORE PROJECT, PROPOSED FACILITY,

Source of Sample: TFB-1 **Depth:** 40 **Sample Number:** 13

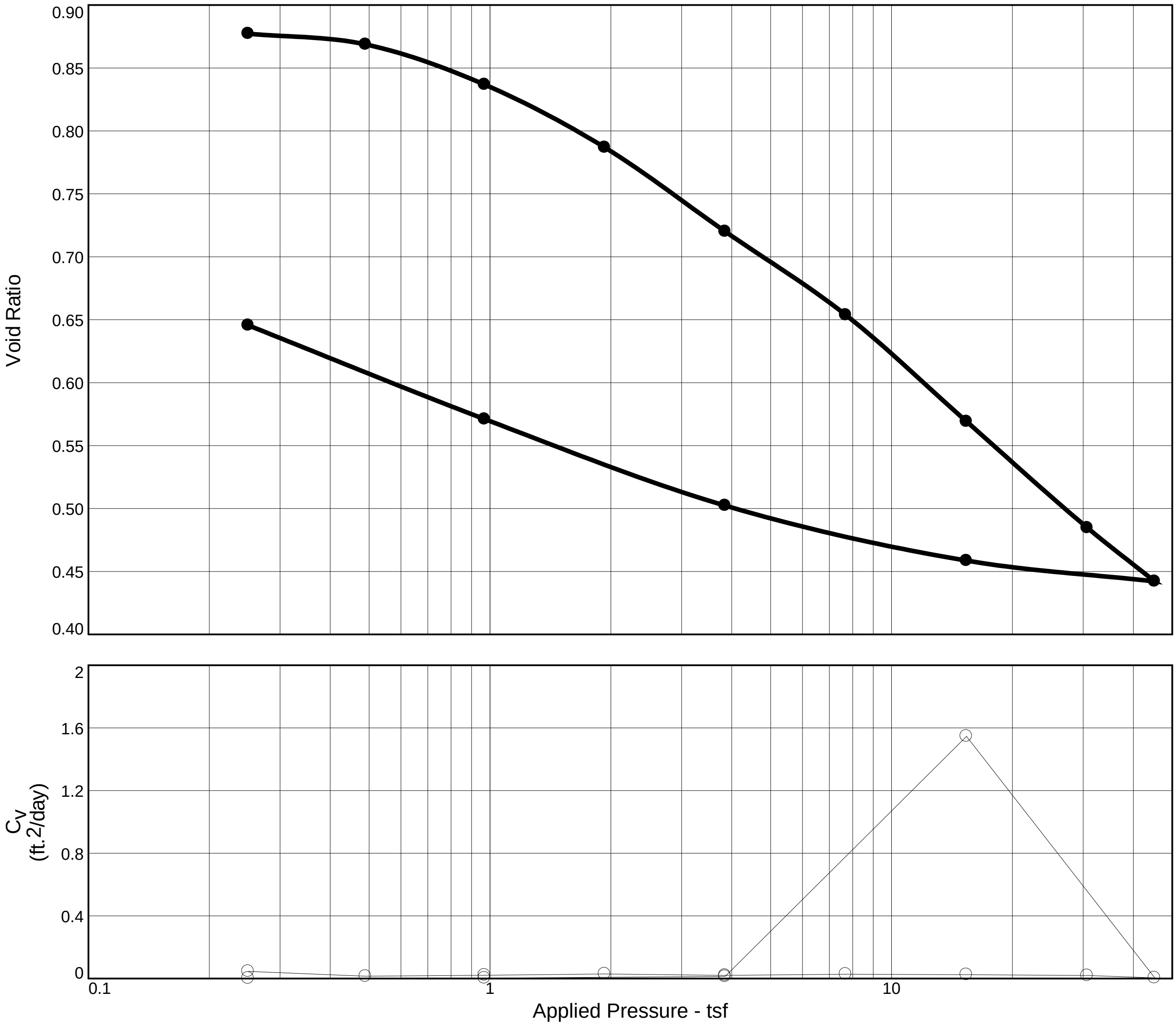
EUSTIS ENGINEERING SINCE 1946

Remarks:

Figure

Tested By: ASR **Checked By:** RR

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture							
95.6 %	30.7 %	90.6	65	46	2.72	2.2	0.28	0.874

MATERIAL DESCRIPTION	USCS	AASHTO
MST T & LT G CL W/ TR-SI POC	CH	

Project No. G0389 **Client:** COMPTON ENGINEERING, INC., BAY ST. LOUIS,

Project: HANCOCK COUNTY PORT AND HARBOR COMMISSION - RESTORE PROJECT, PROPOSED FACILITY,

Source of Sample: WMB-2 **Depth:** 12 **Sample Number:** 5

EUSTIS ENGINEERING SINCE 1946

Remarks:

Figure

Tested By: ASR **Checked By:** RR

APPENDIX II

EUSTIS ENGINEERING SINCE 1946

CONE PENETRATION SOIL BEHAVIOR TYPE LEGEND

1 - SENSITIVE FINE GRAINED

2 - ORGANIC MATERIAL

3 - CLAY

4 - SILTY CLAY TO CLAY

5 - CLAYEY SILT TO SILTY CLAY

6 - SANDY SILT TO CLAYEY SILT

7 - SILTY SAND TO SANDY SILT

8 - SAND TO SILTY SAND

9 - SAND

10 - GRAVELLY SAND TO SAND

11 - VERY STIFF FINE GRAINED (*)

12 - SAND TO CLAYEY SAND (*)

*OVERCONSOLIDATED OR CEMENTED

EUSTIS ENGINEERING SINCE 1946

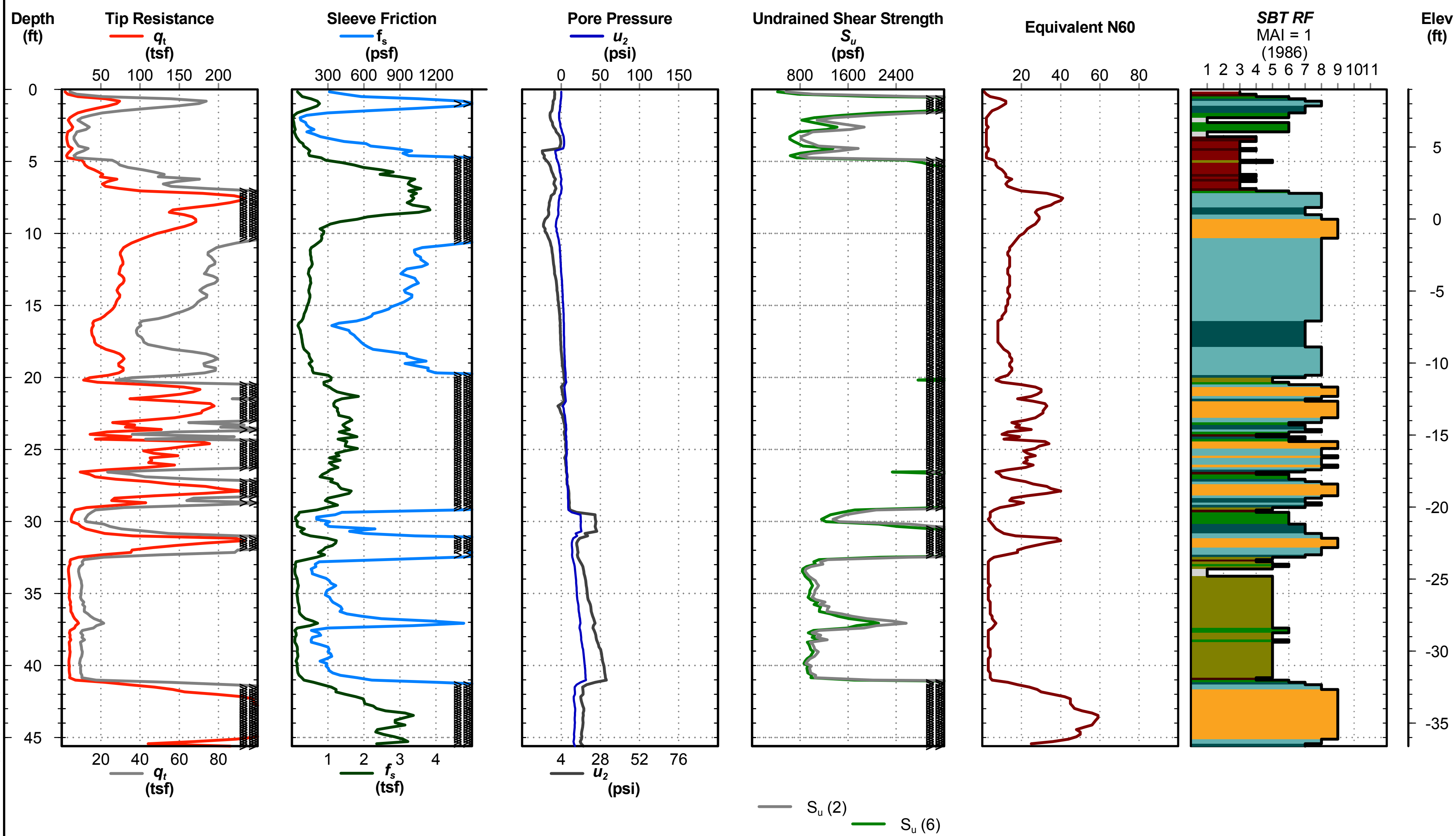
Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

CONE PENETRATION TEST

PPC-1

Project No: G0389
Date: 10/29/2018
Latitude: 30.23720°
Longitude: -89.56570°
CPT ID: DSA1082

Elevation: 9.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 45.6 ft
Operator: G.Reitmeyer



EUSTIS GINT LIBRARY012017.GLB EE 5 GRAPH CPT LOG G0389.GPJ 2/15/19

Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389PPC-1x.cpt

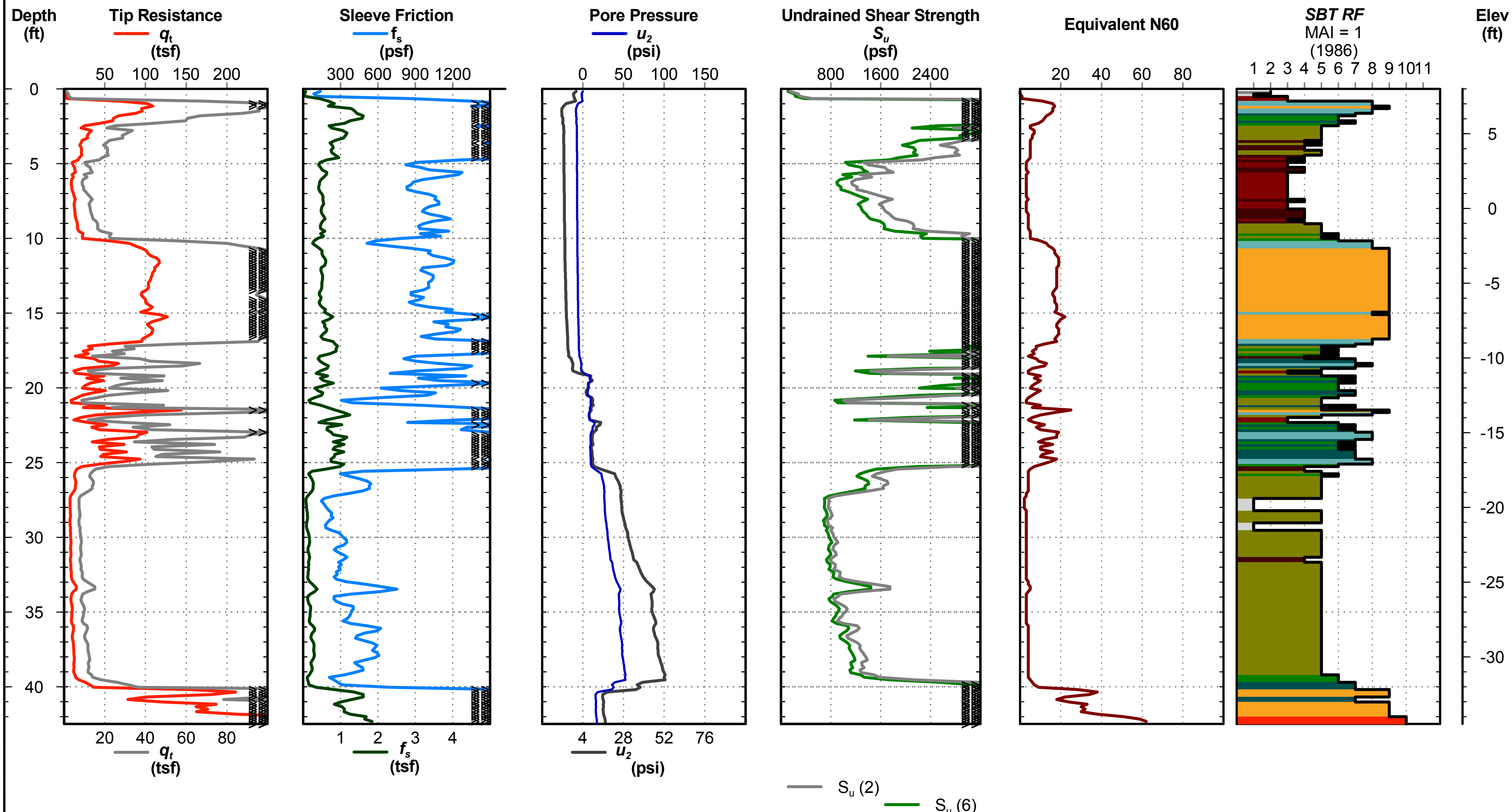
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

SGC-1

Project No: G0389
Date: 10/26/2018
Latitude: 30.23847°
Longitude: -89.56492°
CPT ID: DSG0709

Elevation: 8.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 42.5 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

EUSTIS
ENGINEERING
SINCE 1946

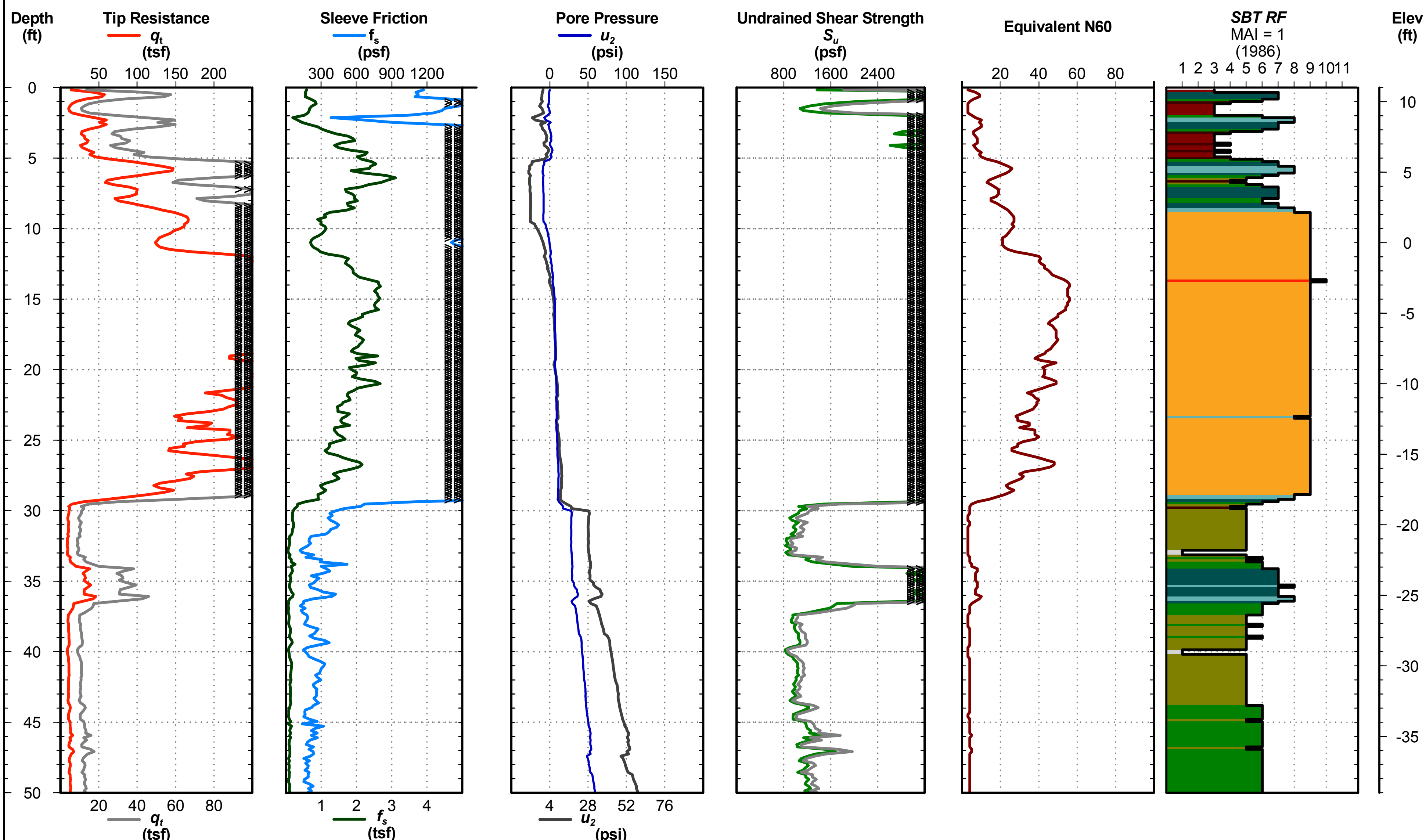
Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

CONE PENETRATION TEST

TFC-1

Project No: G0389
Date: 10/29/2018
Latitude: 30.23653°
Longitude: -89.56319°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 80.4 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

EUSTIS
ENGINEERING
SINCE 1946

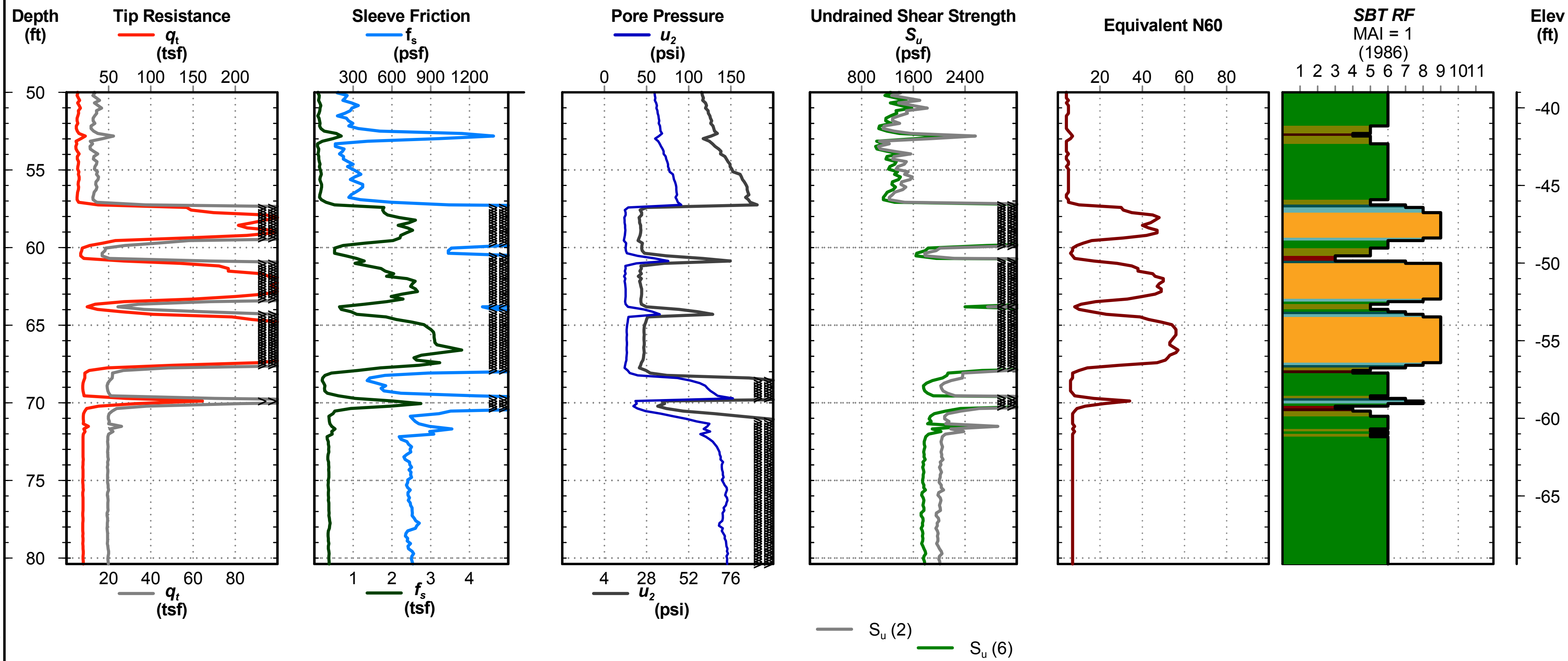
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-1

Project No: G0389
Date: 10/29/2018
Latitude: 30.23653°
Longitude: -89.56319°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 80.4 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

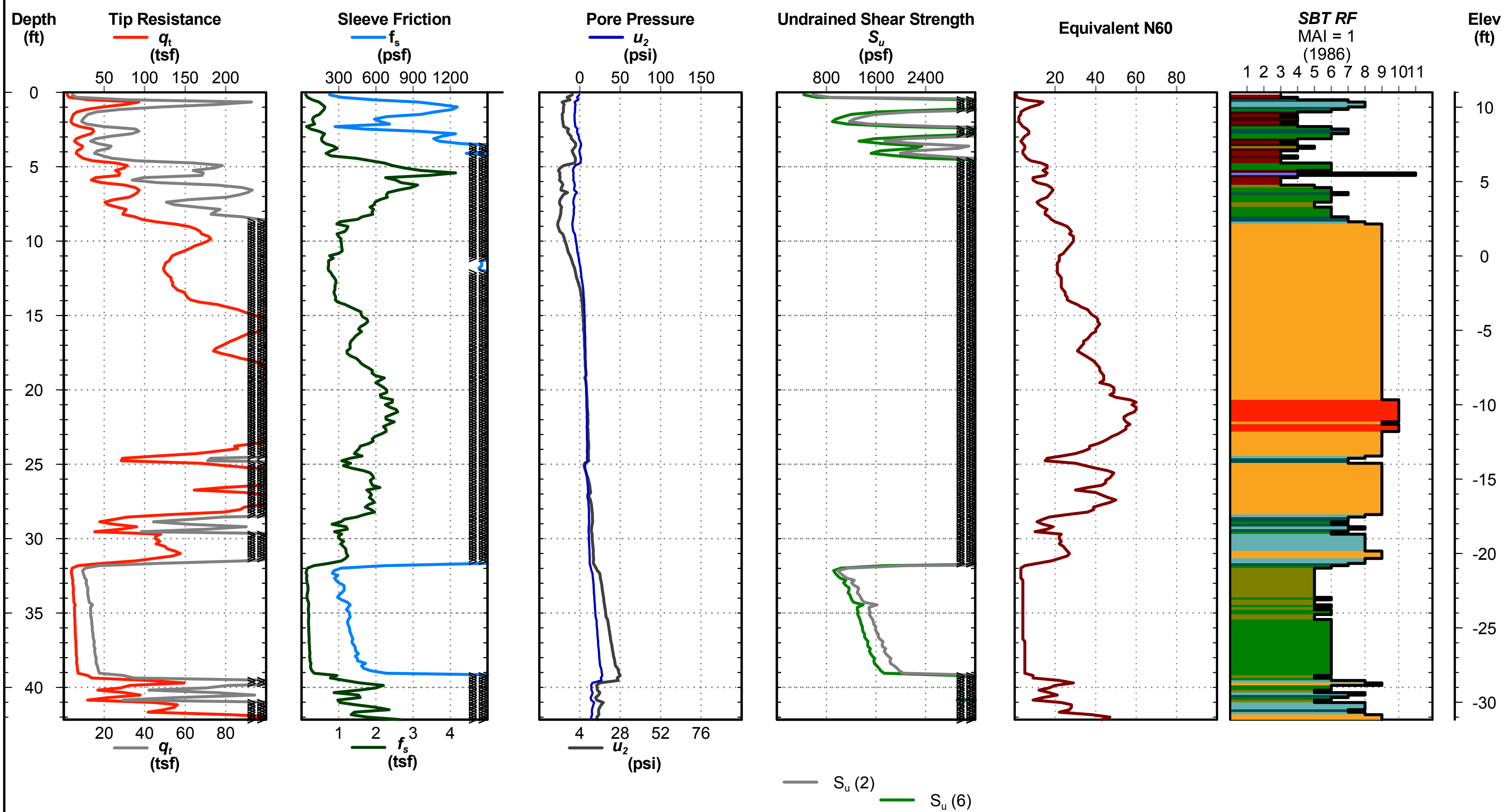
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-2

Project No: G0389
Date: 10/29/2018
Latitude: 30.23623°
Longitude: -89.56403°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 42.2 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-2.cpt

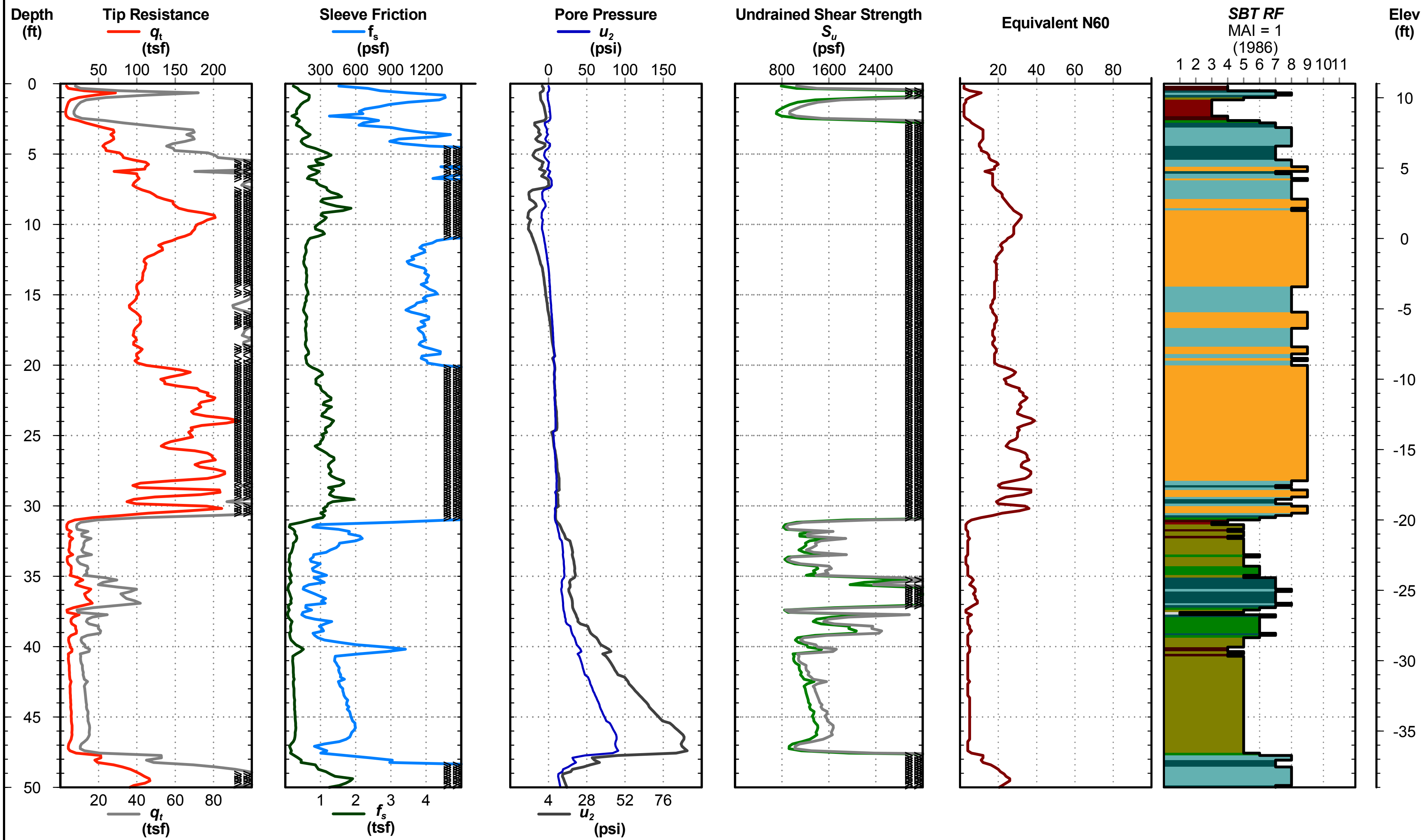
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-3

Project No: G0389
Date: 10/30/2018
Latitude: 30.23553°
Longitude: -89.56407°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 70.4 ft
Operator: G.Reitmeyer



EUSTIS GINT LIBRARY012017.GLB EE 5 GRAPH CPT LOG G0389.GPJ 2/15/19

Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-3.cpt

EUSTIS ENGINEERING SINCE 1946

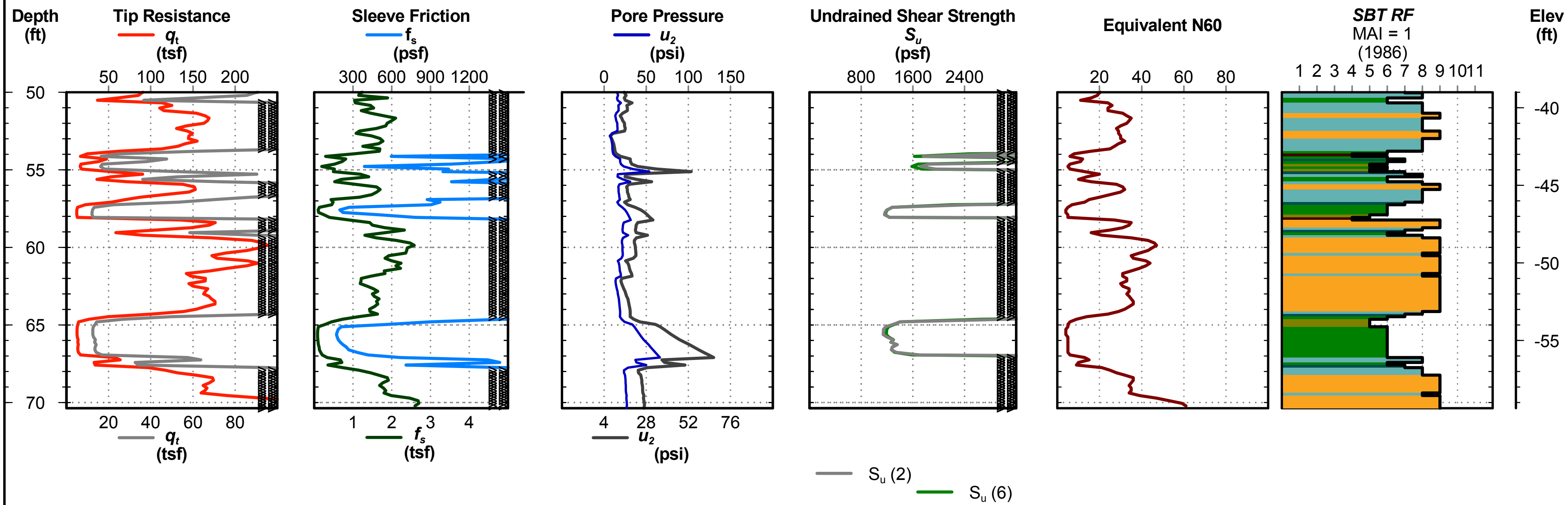
Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

CONE PENETRATION TEST

TFC-3

Project No: G0389
Date: 10/30/2018
Latitude: 30.23553°
Longitude: -89.56407°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 70.4 ft
Operator: G.Reitmeyer



EUSTIS GINT LIBRARY012017.GLB EE 5 GRAPH CPT LOG G0389.GPJ 2/15/19

Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-3.cpt

CONE PENETRATION TEST

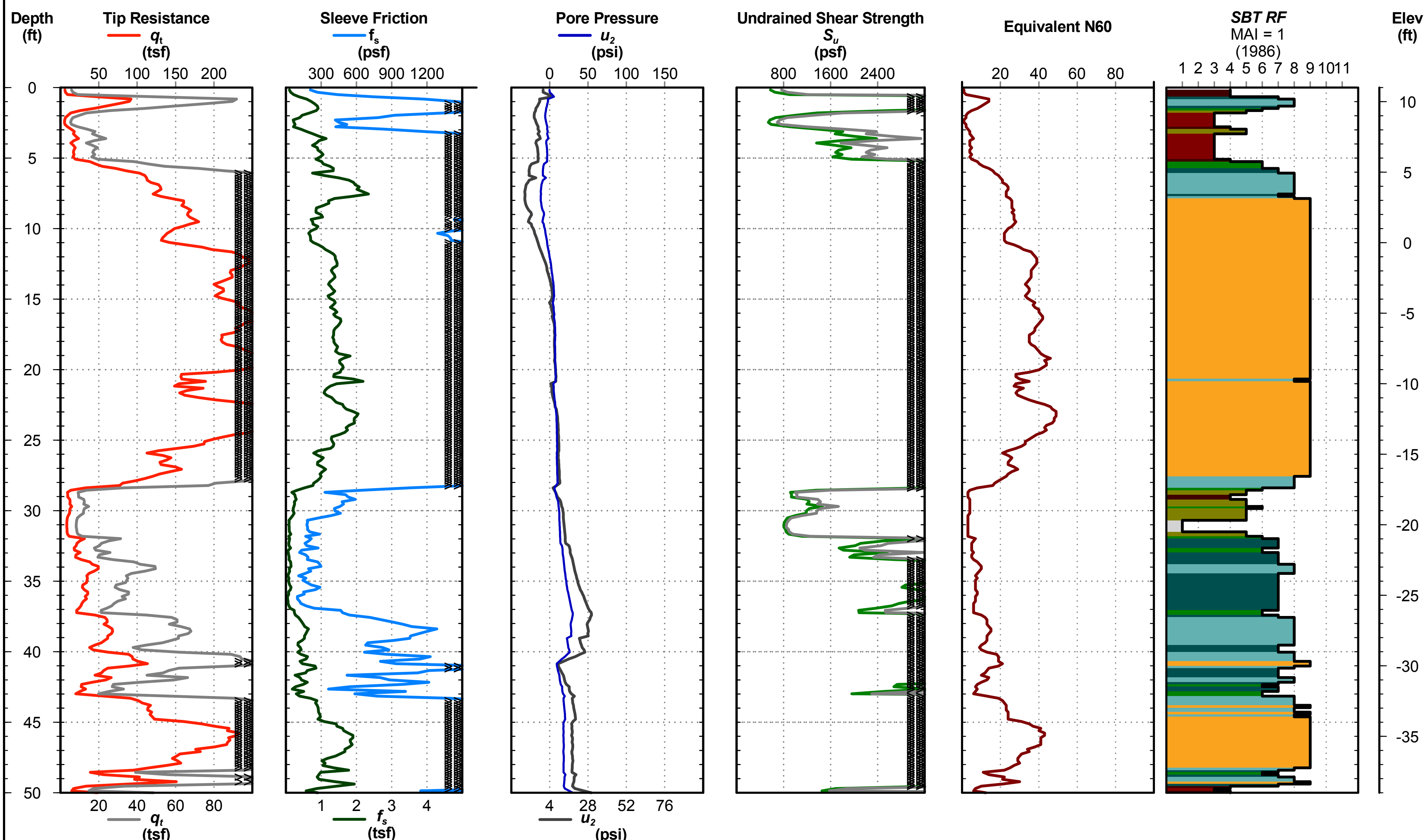
EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-4

Project No: G0389
Date: 10/30/2018
Latitude: 30.23648°
Longitude: -89.56237°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 80.5 ft
Operator: G.Reitmeyer



EUSTIS GINT LIBRARY012017.GLB EE 5 GRAPH CPT LOG G0389.GPJ 2/15/19

Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-4.cpt

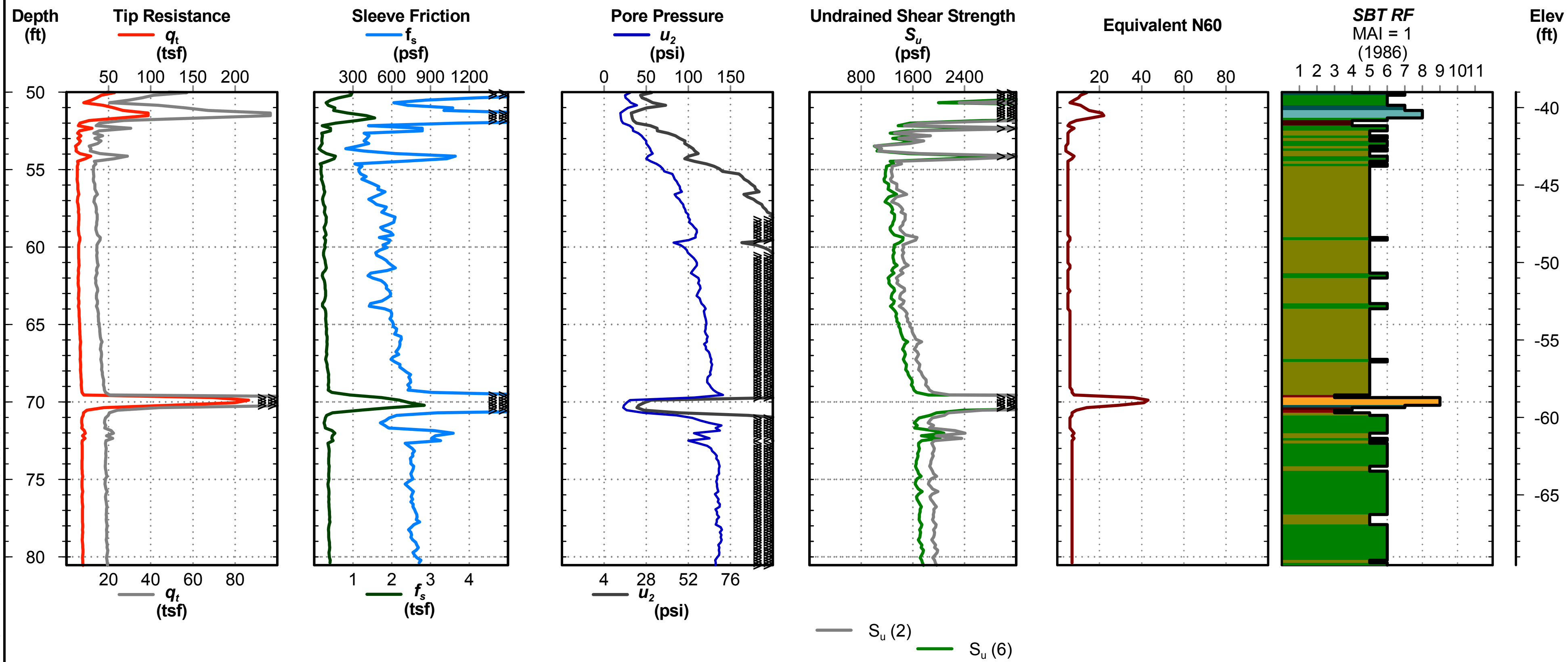
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-4

Project No: G0389
Date: 10/30/2018
Latitude: 30.23648°
Longitude: -89.56237°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 80.5 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-4.cpt

CONE PENETRATION TEST

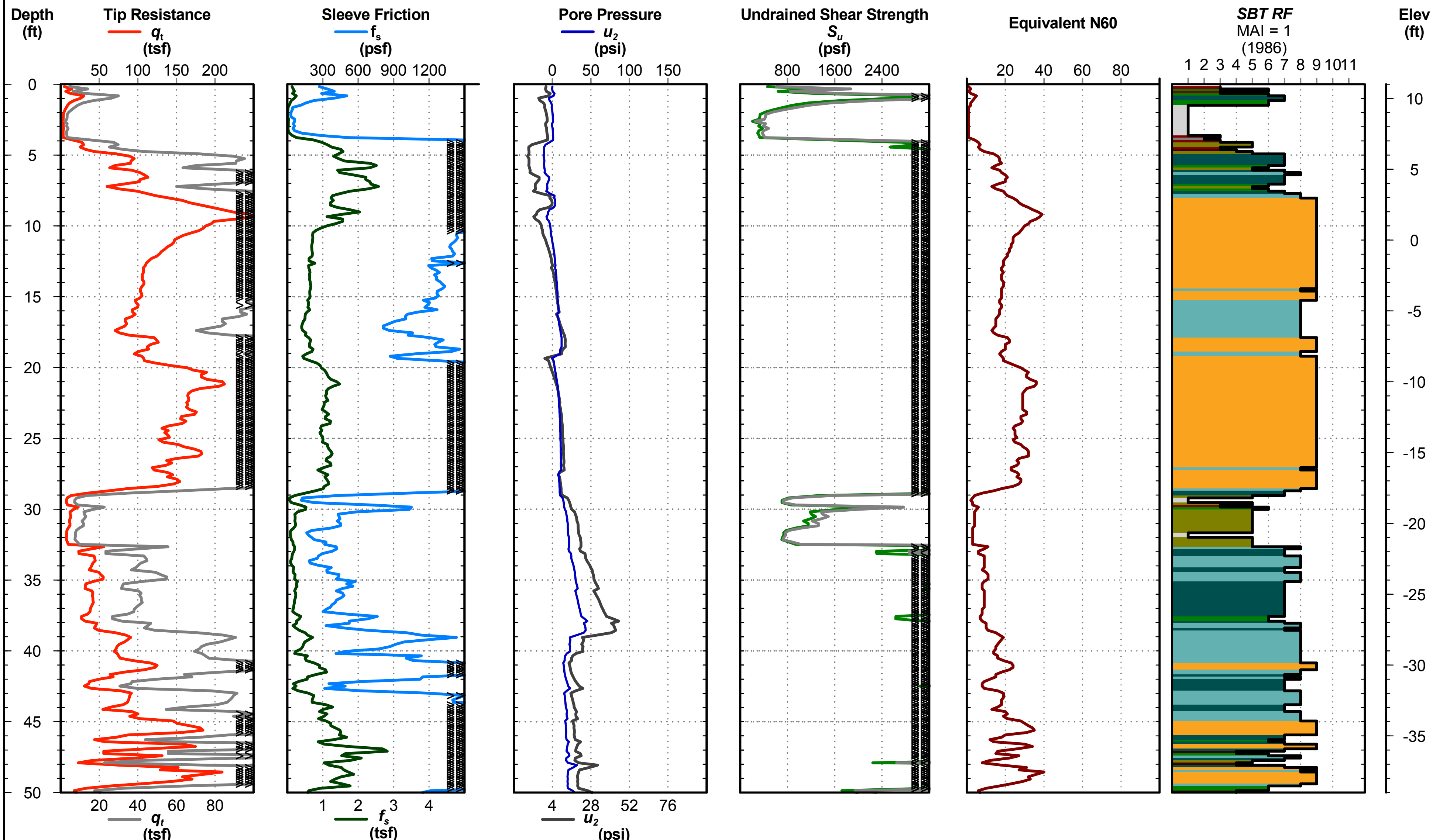
EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-5

Project No: G0389
Date: 10/30/2018
Latitude: 30.23580°
Longitude: -89.56287°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 80.5 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-5.cpt

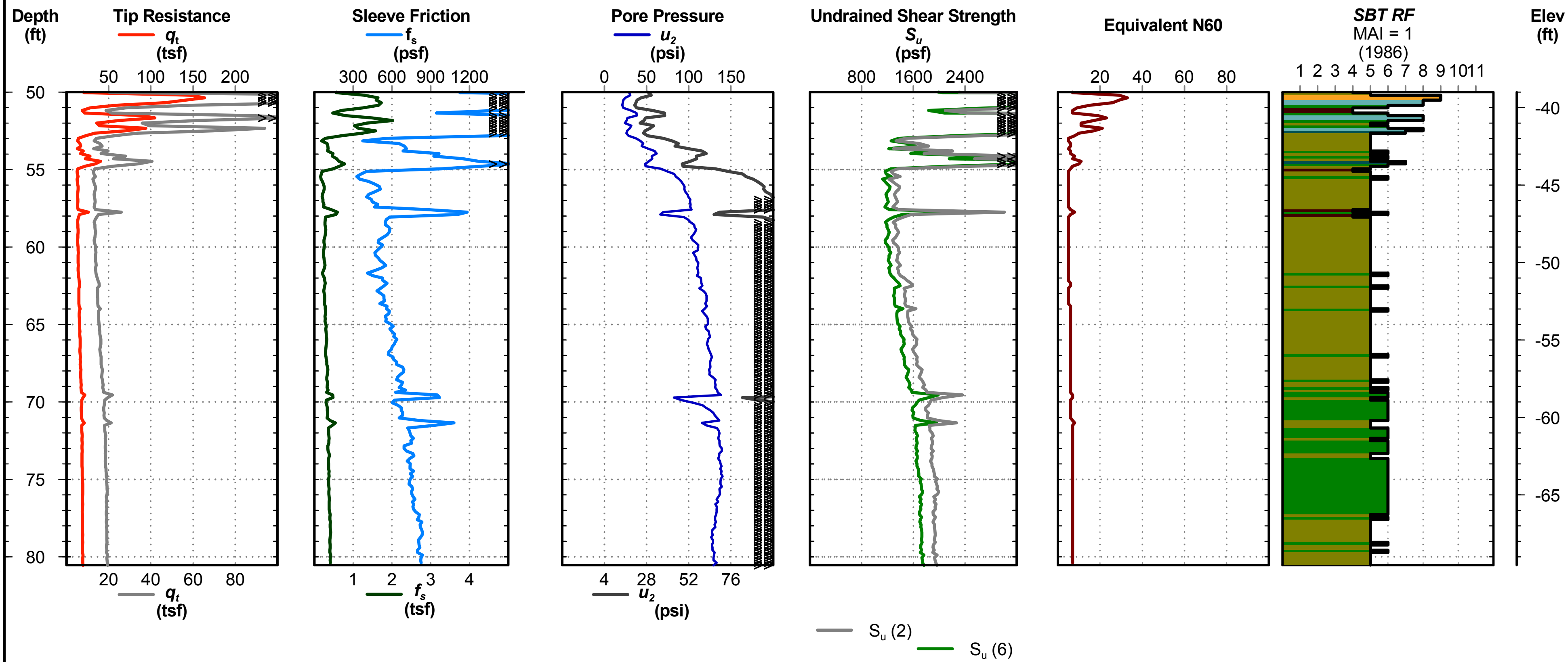
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-5

Project No: G0389
Date: 10/30/2018
Latitude: 30.23580°
Longitude: -89.56287°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 80.5 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-5.cpt

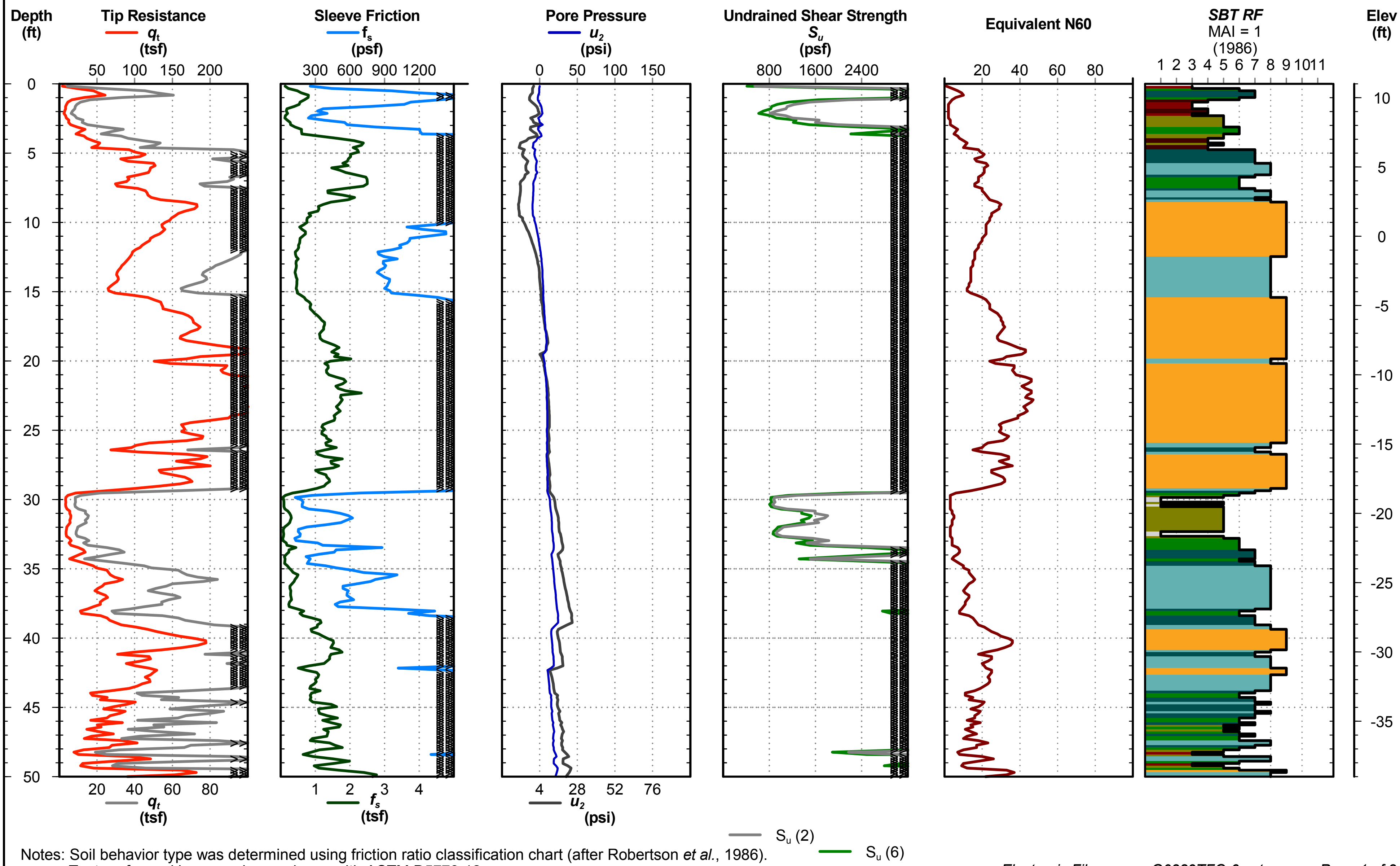
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-6

Project No: G0389
Date: 10/30/2018
Latitude: 30.23527°
Longitude: -89.56338°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 80.7 ft
Operator: G.Reitmeyer



EUSTIS GINT LIBRARY012017.GLB EE 5 GRAPH CPT LOG G0389.GPJ 2/15/19

Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-6.cpt

CONE PENETRATION TEST

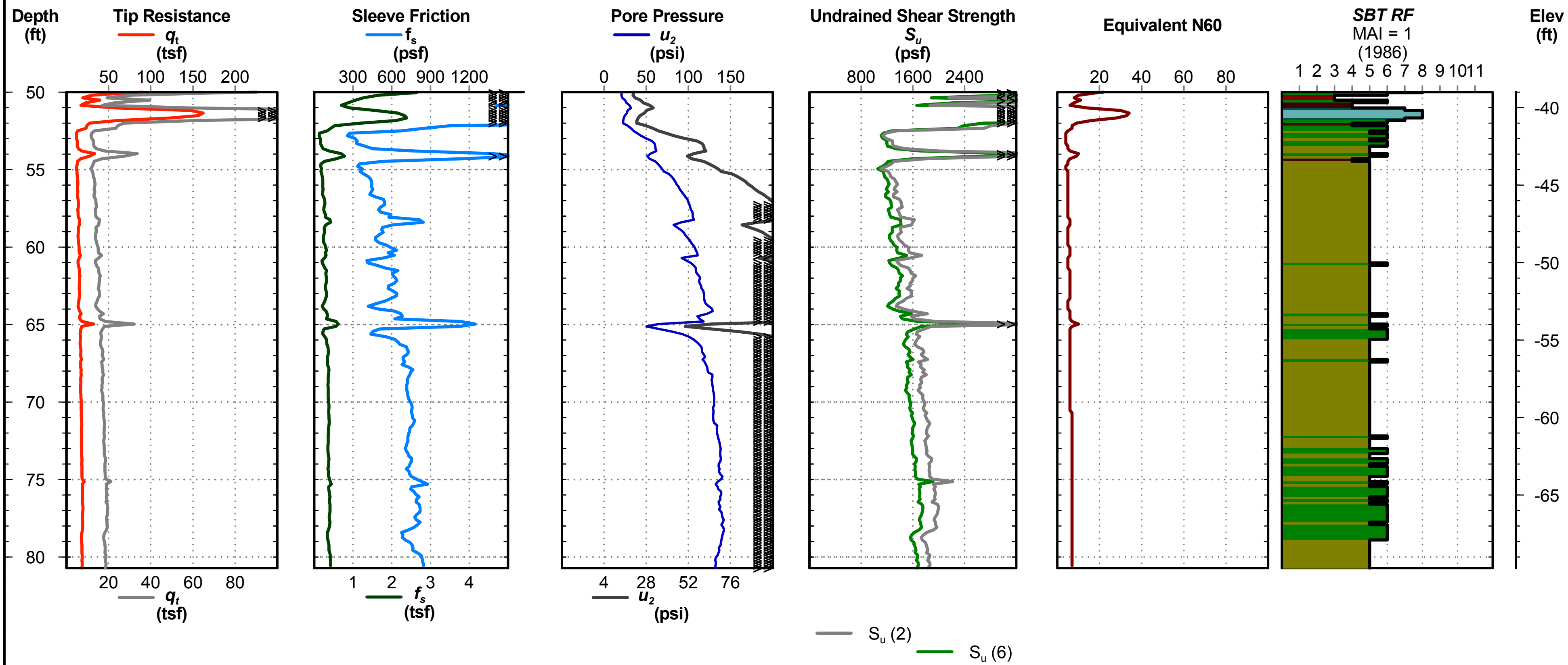
EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

TFC-6

Project No: G0389
Date: 10/30/2018
Latitude: 30.23527°
Longitude: -89.56338°
CPT ID: DSA1082

Elevation: 11.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 80.7 ft
Operator: G.Reitmeyer



EUSTIS GINT LIBRARY012017.GLB EE 5 GRAPH CPT LOG G0389.GPJ 2/15/19

Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389TFC-6.cpt

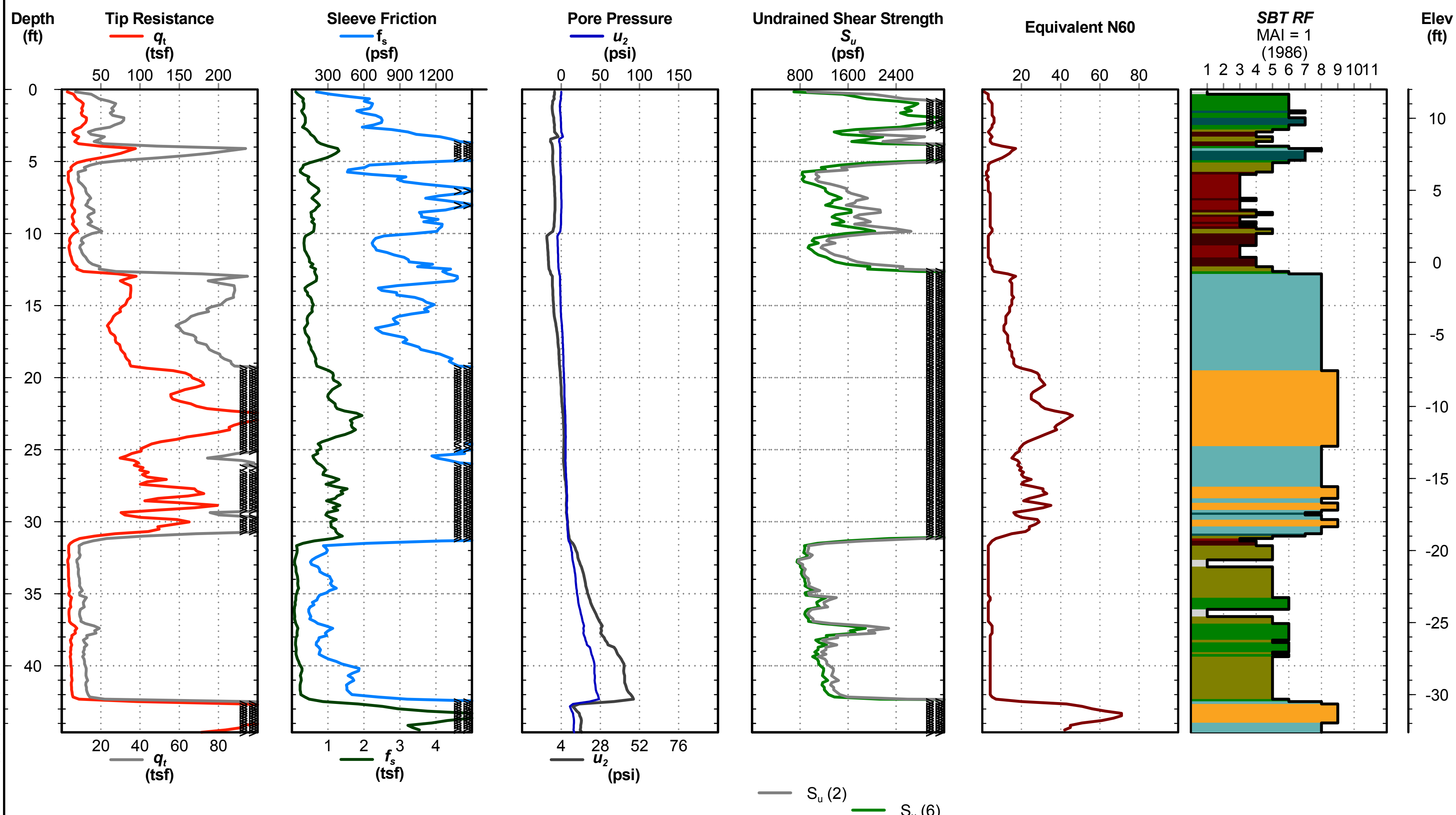
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

WMC-1

Project No: G0389
Date: 10/29/2018
Latitude: 30.23898°
Longitude: -89.56760°
CPT ID: DSA1082

Elevation: 12.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 44.6 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389WMC-1.cpt

CONE PENETRATION TEST

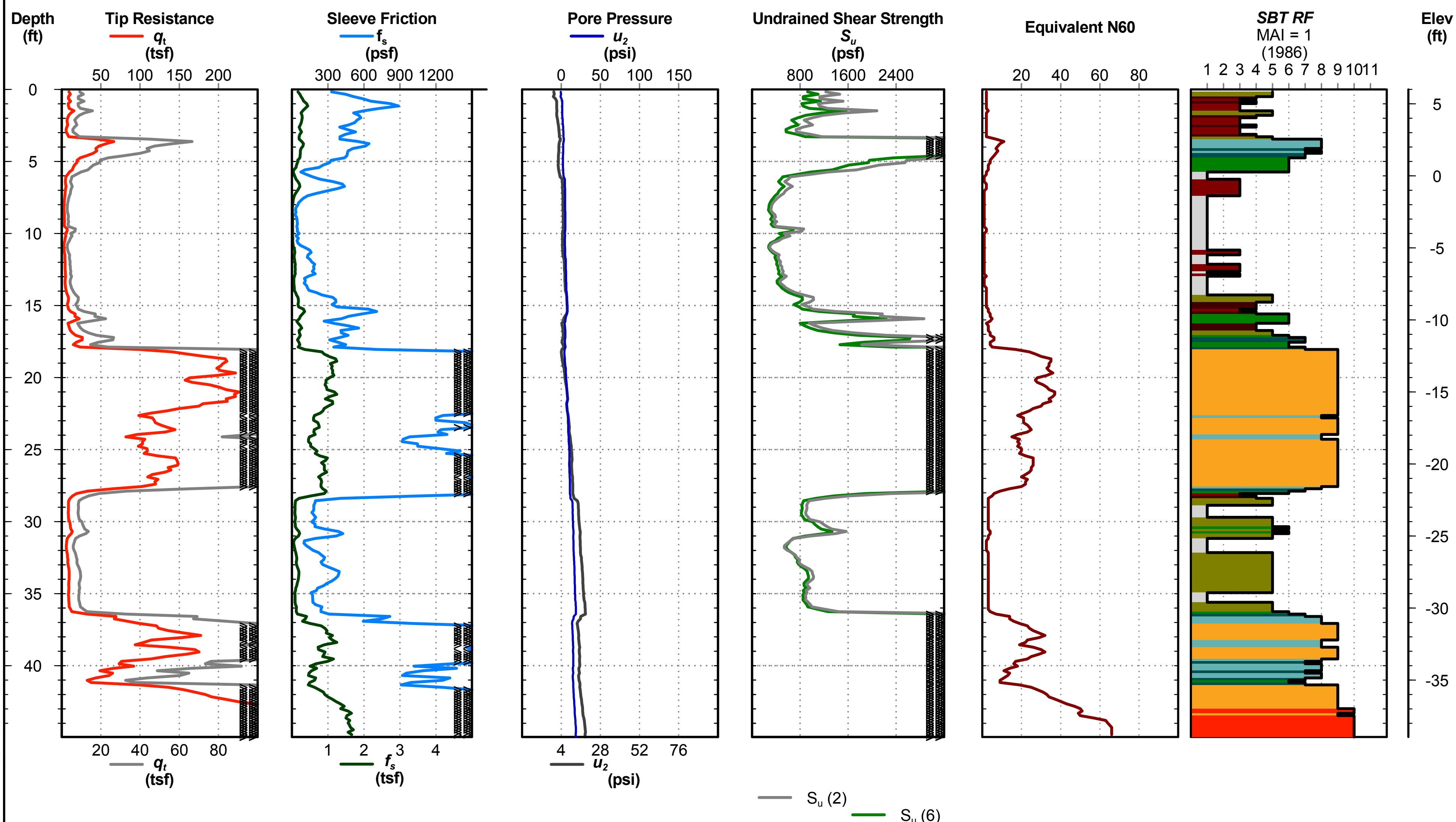
EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

WMC-2

Project No: G0389
Date: 10/26/2018
Latitude: 30.23837°
Longitude: -89.56760°
CPT ID: DSG0709

Elevation: 6.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 45.0 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

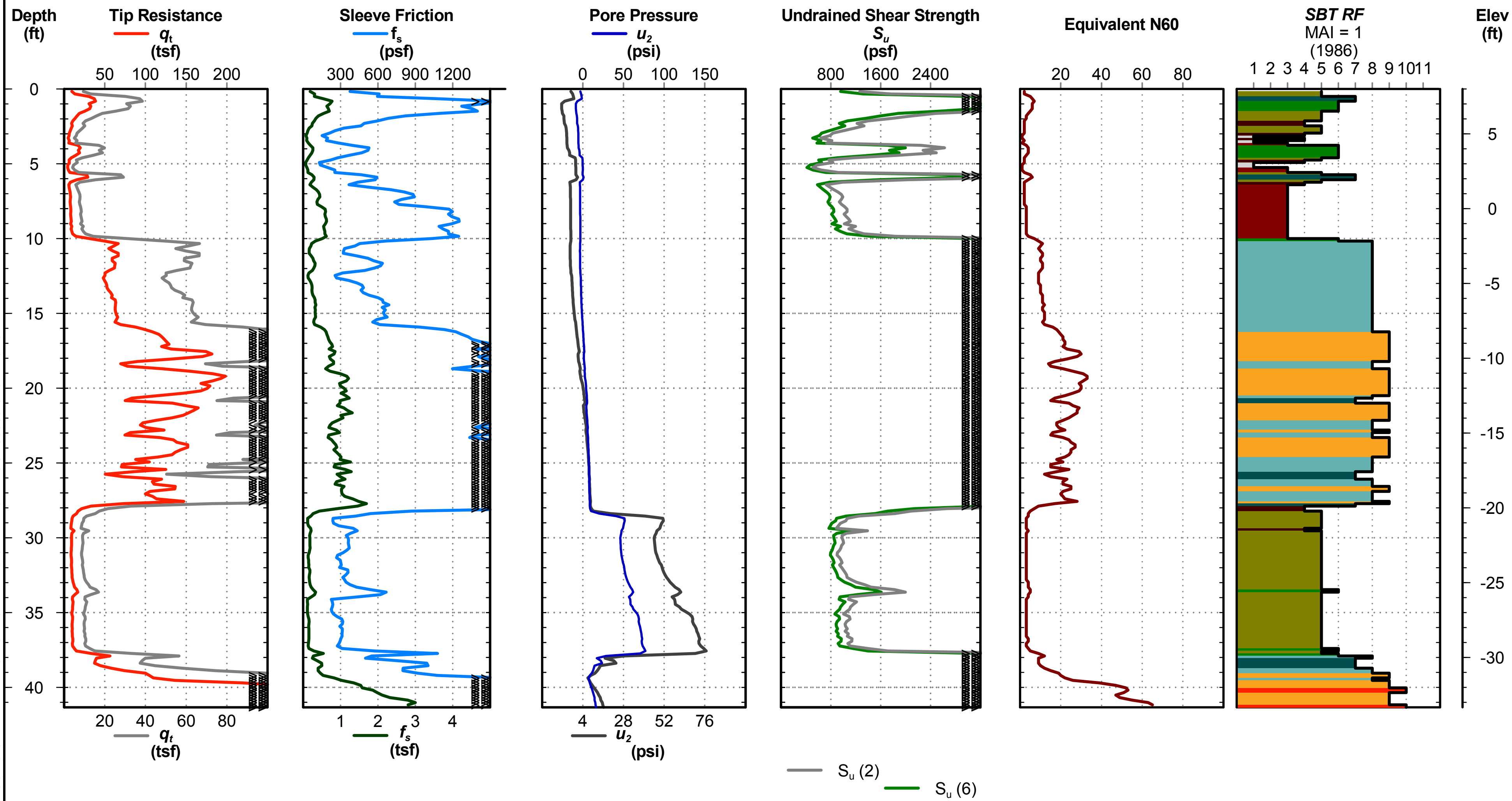
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

WMC-3

Project No: G0389
Date: 10/26/2018
Latitude: 30.23842°
Longitude: -89.56690°
CPT ID: DSG0709

Elevation: 8.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 41.3 ft
Operator: G.Reitmeyer



EUSTIS GINT LIBRARY012017.GLB EE 5 GRAPH CPT LOG G0389.GPJ 2/15/19

Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389WMC-3.cpt

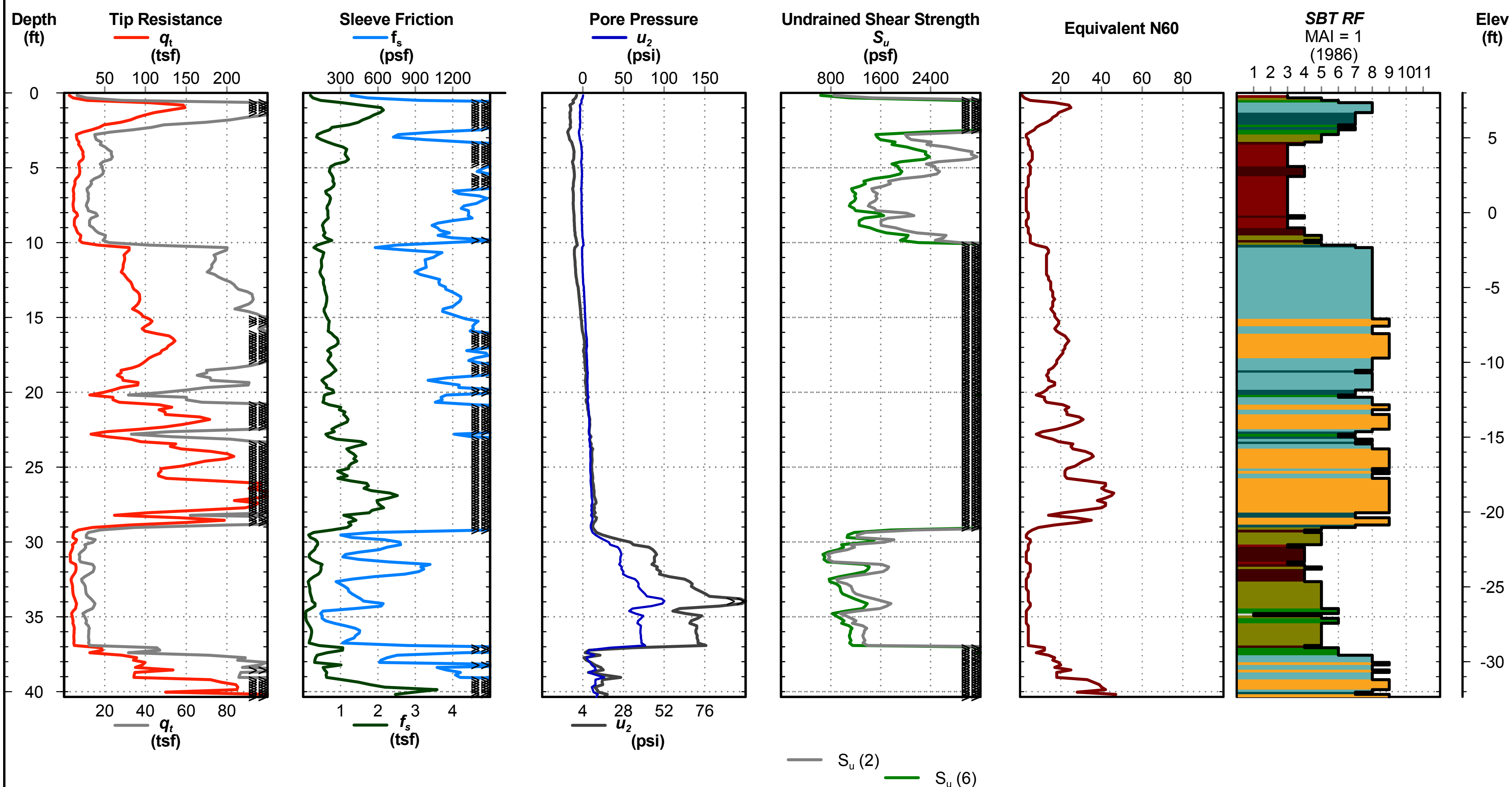
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

WMC-4

Project No: G0389
Date: 10/29/2018
Latitude: 30.23768°
Longitude: -89.56690°
CPT ID: DSA1082

Elevation: 8.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 40.4 ft
Operator: G.Reitmeyer



EUSTIS GINT LIBRARY012017.GLB EE 5 GRAPH CPT LOG G0389.GPJ 2/15/19

Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389WMC-4.cpt

CONE PENETRATION TEST

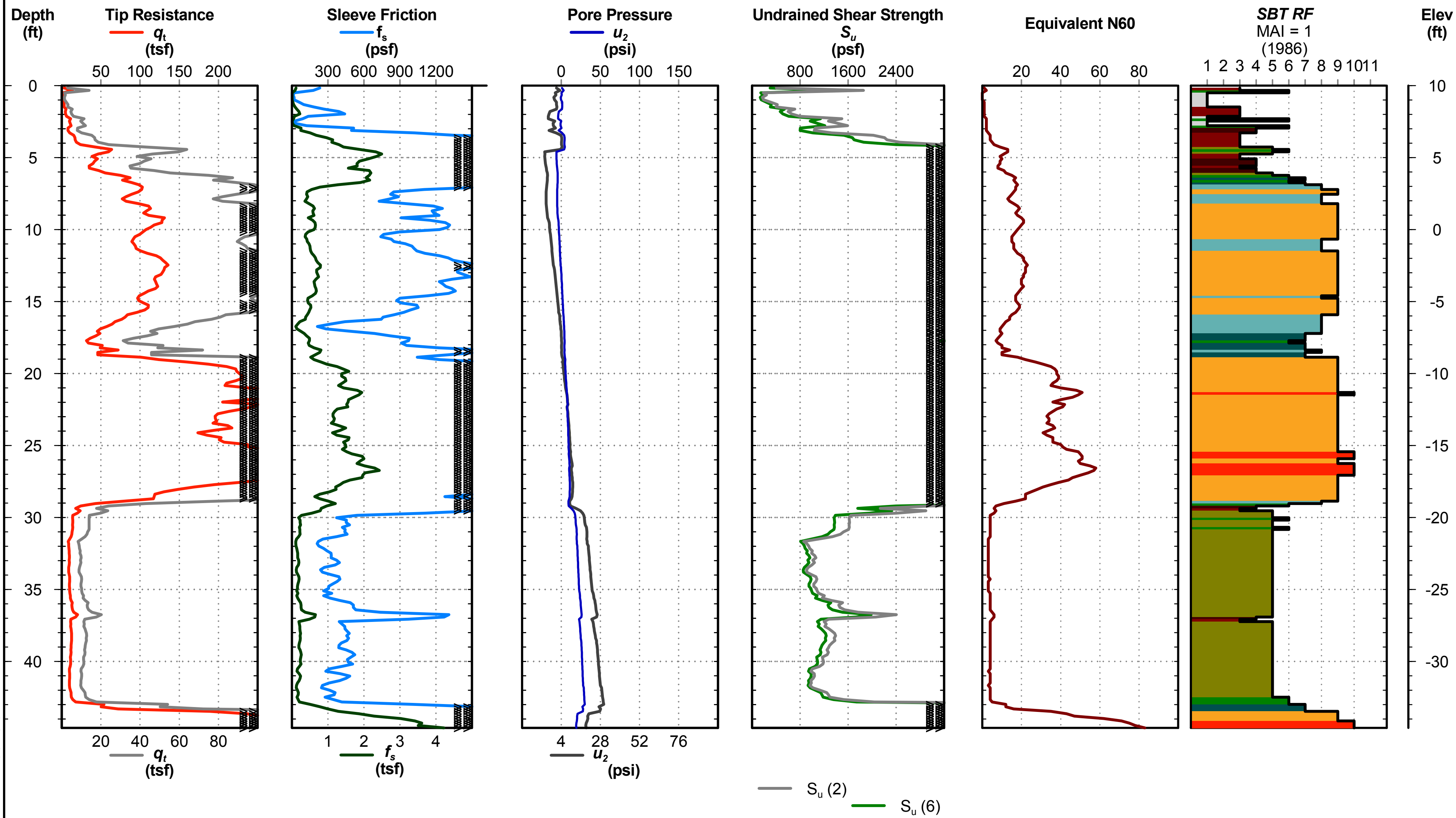
EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

WSC-1

Project No: G0389
Date: 10/26/2018
Latitude: 30.23775°
Longitude: -89.56442°
CPT ID: DSG0709

Elevation: 10.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 44.6 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

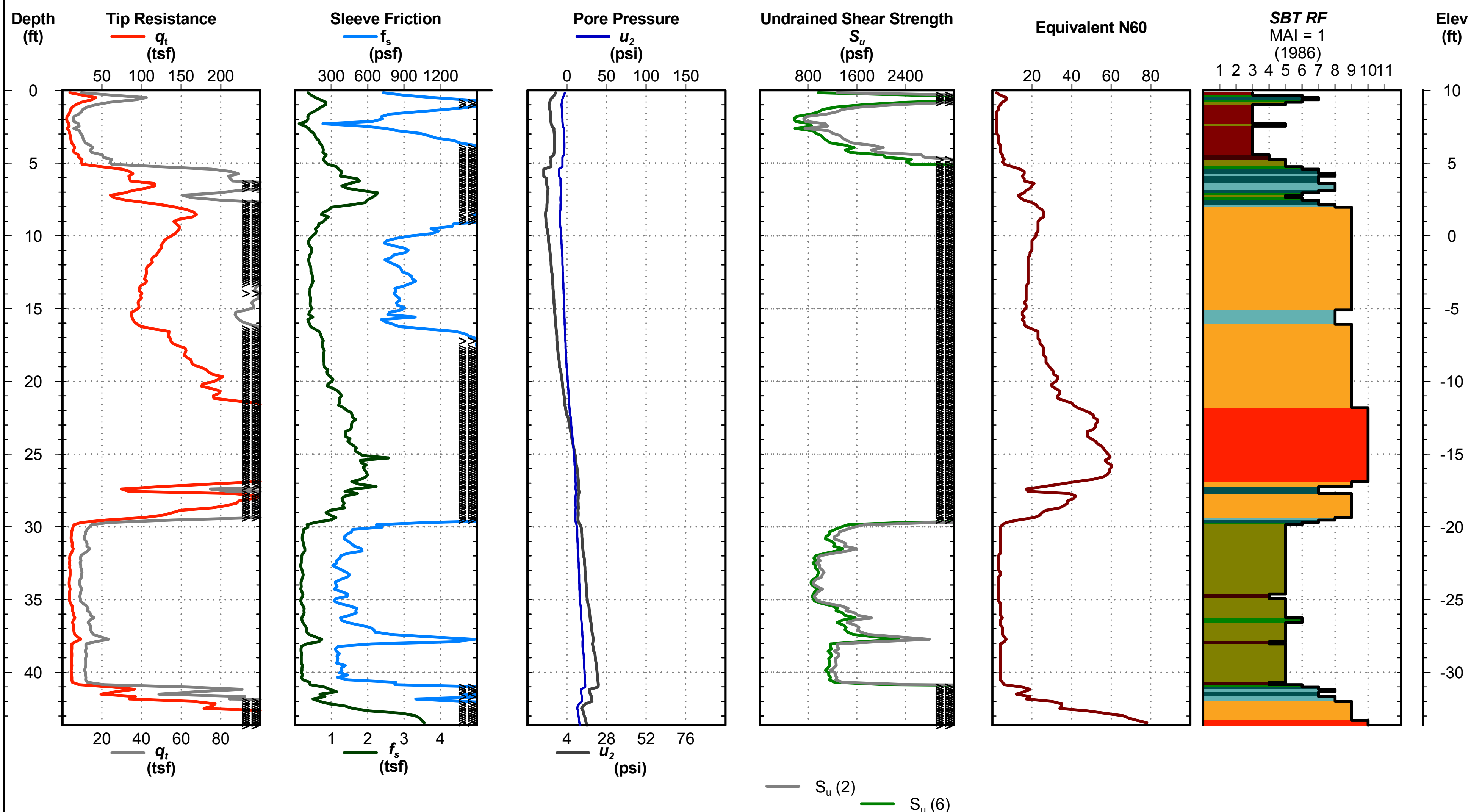
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

WSC-2

Project No: G0389
Date: 10/26/2018
Latitude: 30.23703°
Longitude: -89.56433°
CPT ID: DSG0709

Elevation: 10.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 43.6 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389WSC-2.cpt

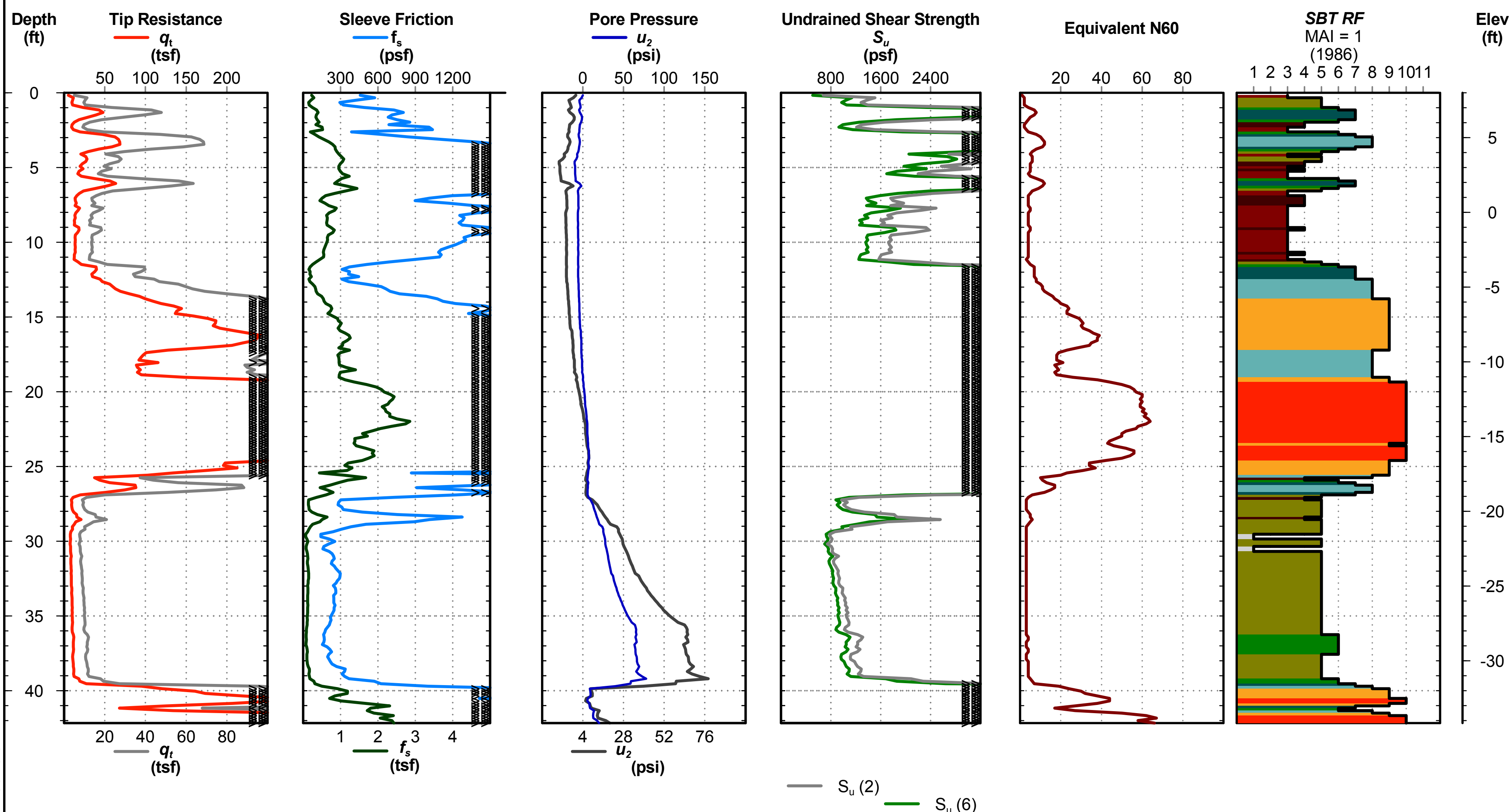
CONE PENETRATION TEST

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

WTC-1

Project No: G0389
Date: 10/30/2018
Latitude: 30.23915°
Longitude: -89.56308°
CPT ID: DSA1082

Elevation: 8.0
Datum: NAVD88
Est. Water Depth: 2.0 ft
Total Depth: 42.2 ft
Operator: G.Reitmeyer



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-12.

Electronic Filename: G0389WTC-1x.cpt

CPT Correlations

References are in parenthesis next to the appropriate equation.

General

p_a=atmospheric pressure (for unit normalization)
q_t=corrected cone tip resistance (tsf)
f_s=friction sleeve resistance (tsf)
$R_f = 100\% \cdot (f_s/q_t)$
u_2=pore pressure behind cone tip (tsf)
u_0=hydrostatic pressure

$B_q = (u_2-u_0)/(q_t-\sigma_{vo})$
$Q_t=(q_t-\sigma_{vo})/\sigma'_{vo}$
$F_r=100\% \cdot f_s/(q_t-\sigma_{vo})$
$I_c = ((3.47-\log Q_t)^2 + (\log F_r+1.22)^2)^{0.5}$ 2
$I_{SBT} = ((3.47-\log(q_c/p_a))^2+(\log F_r+1.22)^2)^{0.5}$ 23
$I_{c\ J\&D} = \sqrt{\{3-\log(Q_t\cdot(1-B_q))\}^2 + [1.5+1.3\cdot\log(F_r)]^2}$ 27
$I_{c\ J\&B} = \sqrt{\{3-\log(Q_t\cdot(1-B_q)+1\}^2 + [1.5+1.3\cdot\log(F_r)]^2}$ 28

K_o

K_0 (1)	$K_o = (1-\sin\varphi)OCR^{\sin\varphi}$	
K_0 (2)	$K_o = 0.1(Q_t)$	1

Stress History

$OCR = \sigma_p'/\sigma'_{vo}$

OCR (1)	$\sigma_p' = 0.33(q_t - \sigma_{vo})$ - clays	8
OCR (2)	$\sigma_p' = 0.53(u_2 - u_o)$ - clays	9
OCR (3)	$\sigma_p' = 0.60(q_t - u_2)$ - clays	9
OCR (4)	$OCR = 0.25\, Q_t^{1.25}$ – clays	37
OCR (5)	$OCR = \left[\frac{0.192*(q_t/p_a)^{0.22}}{(1-\sin(\emptyset'))\cdot(\sigma'_{v0}/p_a{}^{0.31})}\right]^{\frac{1}{\sin(\emptyset'-0.27)}}$ – sands	35
OCR (6)	$\sigma_p' = .101\cdot p_a{}^{0.102}\cdot G_{max}{}^{0.478}\cdot \sigma'_{v0}{}^{0.420}$ – all soils	36

N-Value

$N_{60} = (q_t/p_a)/[8.5(1-I_c/4.6)]$ 6

Undrained Shear Strength

S_u (1)	$S_u = (u_2 - u_o)/N_u$	where $7 \le N_u \le 9$	10
S_u (2)	$S_u = (q_t - \sigma_{vo})/N_{kT}$	where $15 \le N_{kT} \le 20$	11
S_u (3)	$S_u = 0.091 * ((\sigma'_{vo}{}^{0.2}) * (q_t - \sigma_{vo})^{0.8}$		21
S_u (4)	$S_u = (q_c - \sigma_{vo})/N_k$	where $15 \le N_k \le 20$	11
S_u (5)	$S_u = q_t/N_c$	where XXX ≤ N_c ≤ YYY	
S_u (6)	$S_u = q_c/N_c$	where XXX ≤ N_c ≤ YYY	

Effective Cohesion

$c' = 0.02 * \sigma_p'$ 38

Drained Friction Angle

φ' (1)	$\varphi' = 17.6 + 11.0\text{Log}[q_t/(\sigma_{vo}')^{0.5}]$	1
φ' (2)	$\varphi' = \arctan[0.1 + 0.38\text{Log}(q_t/\sigma_{vo}')]$	13
φ' (3)	$\varphi' = 30.8\text{Log}[(f_s/\sigma_{vo}')+1.26]$ (for clays or sands)	14
φ' (4)	$\varphi' = 29.5\ B_q^{0.121}\ (0.256 + 0.33\ B_q + \text{Log}(Q_t))$	24

Unit Weight

$\rho = \gamma/\gamma_w$

$\rho = 0.8\text{Log}(V_s)$ V_s in m/sec 17

Relative Density and Void Ratio

D_R (1)	$D_R = 100(q_{c1}/305)^{1/2}$ where, $q_{c1} = q_c/(\sigma_{vo}')^{1/2}$	1
D_R (2)	$D_R = -1.292 + 0.268\ln(q_c \cdot (\sigma_{vo}'^{-0.5}))$	18
D_R (3)	$D_R = (1/2.41)\cdot\ln(q_{c1}/15.7)$	3
D_R (4)	$D_R = 1/2.91 * \ln((q_c/(61 * \sigma'_{vo}{}^{0.71}))*100$	20
D_R (5)	$D_R = 100*(0.268*\ln((q_t/p_a)/(\sigma'_{vo}/p_a)\text{^}0.5) - 0.675)$	34

$e_o = 1.099 - 0.204\log(q_{c1})$ 1

$E_D = 5\ q_t$ $I_D = 2.0 - 0.14(R_f)$ $K_D = E_D/(34.7\cdot I_D\cdot \sigma_{vo}')$

Compressibility

M (1) = $R_m\ E_D$ where R_m= function(I_D, K_D) see the following table 22

$I_D <= 0.6$	$R_M = 0.14 + 2.36 \log K_D$
$I_D >= 3$	$R_M = 0.5 + 2 \log K_D$
$0.6 < I_D < 3$	$R_M = R_{M,D} + (2.5 - R_{M,D})\log K_D$
	$R_{M,D} = 0.14 + 0.15(I_D - 0.6)$
$K_D > 10$	$R_M = 0.32 + 2.18 \log K_D$
$R_M <$ 0.85	$R_M = 0.85$

M (2)	$M = q_c\cdot 10^{(1.09-0.0075D}R^{)}$ *sands*	1
M (3)	$M = 8.25\ (q_t - \sigma_{vo})$ *clays*	1
M (4)	$M = \alpha \cdot G_{max}$ *where* $0.02 < \alpha < 2$ and G_{max} *is from* Vs	33

Rigidity Index

$I_R = exp\left[\left(\frac{1.5}{M} + 2.925\right)\cdot\left(\frac{q_t-\sigma_{vo}}{q_t-u_2}\right) - 2.925\right]$ where $M = 6\sin\emptyset'/(3-\sin\emptyset')$ 39

Sensitivity

S_t (1)	$S_t = 7.5/R_f$	2
S_t (2)	$S_t = (q_t - \sigma_{vo})/(15\cdot f_s)$	2

Fines Content

$FC = [(3.58-\log(q_t))^2+(1.43+\log(R_f))^2]^{1.8}$ 4

$FC = [5.31(I_{cfs})^{2.31}]+9.61$, where $I_{cfs} = [(1.95-\text{Log}Q_t)^2+(\text{log}F_r+1.78)^2]^{0.5}$

Shear Wave Velocity

$V_s(1) = 277 \cdot q_t^{0.13} \cdot {\sigma'_{vo}}^{0.27}$ (sands) - m/s and MPa 29

$V_s(2) = 1.75 \cdot q_t^{0.627}$ (clays) - m/s and kPa 30

$V_s(3) = (10.1 \cdot \log q_t - 11.4)^{1.67} \cdot (\frac{f_s}{q_t} \cdot 100)^{0.3}$ (all soils) – m/s and kPa 31

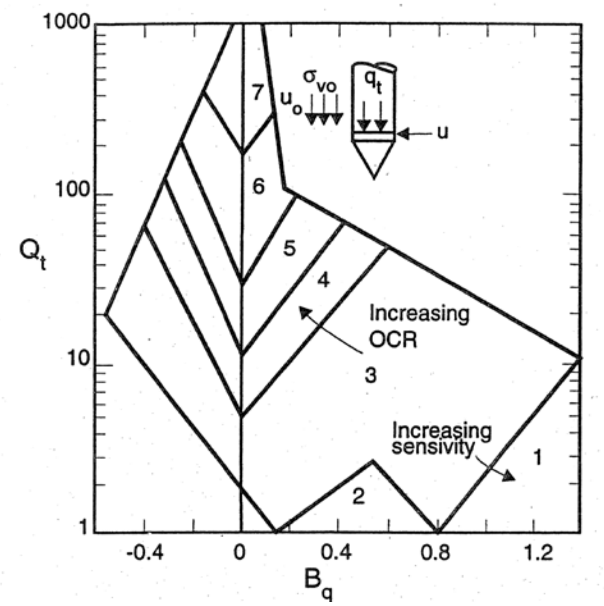
$V_s(4) = 118.8 \cdot \log f_s + 18.5$ (all soils) – m/s and kPa 32

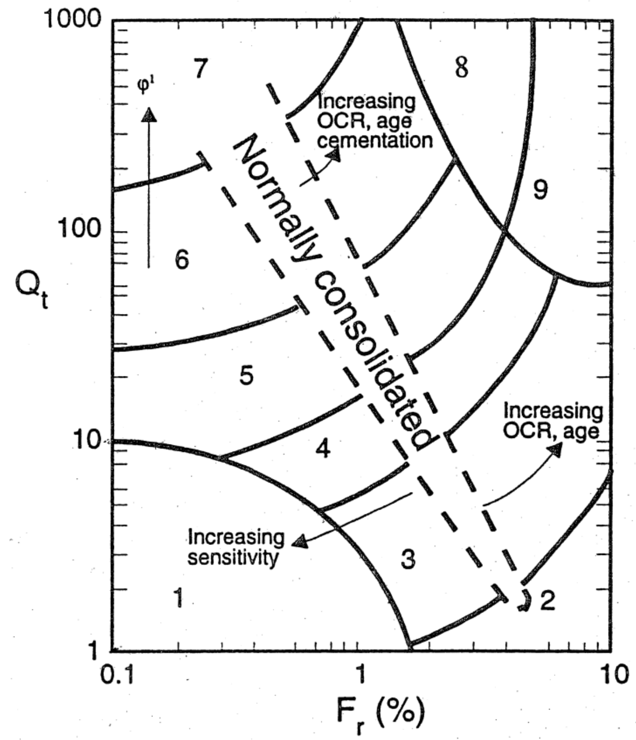
$G_{max} = \rho V_s^2$

Hydraulic Conductivity

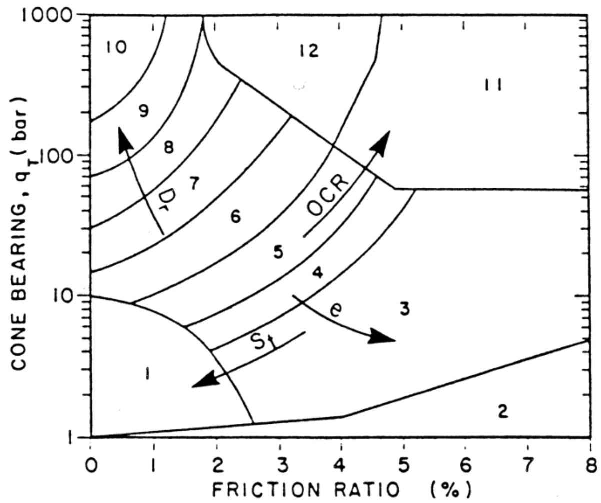
Lookup based on SBT and SBTn (1986 and 1990) 40

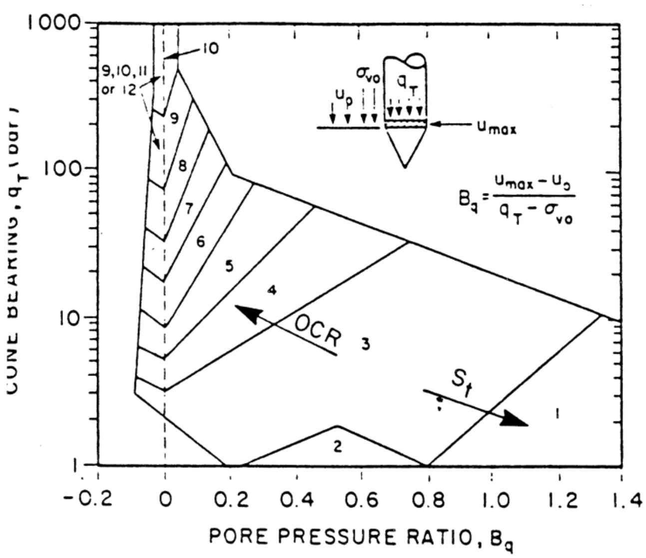
Normalized Soil Behavior Types - Robertson & Campanella (1990)





Non-Normalized Soil Behavior Types – Robertson & Campanella (1986)





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APPENDIX III

EUSTIS ENGINEERING SINCE 1946

LEGEND AND NOTES FOR LOG OF BORING AND TEST RESULTS

PP Pocket penetrometer: Resistance in tons per square foot

SPT Standard Penetration Test: Number of blows of a 140-lb hammer dropped 30 inches required to drive 2-in. O.D., 1.4-in. I.D. sampler a distance of 1 foot into the soil after first seating it 6 inches. Values shown have not been corrected.

SPLR Type of Sampling Shelby SPT Auger Vibracore Geoprobe No sample

SYMBOL Clay Silt Sand Peat/Humus Shells Stone/Gravel

Predominant type shown heavy; modifying type shown light

USC Unified Soil Classification

DENSITY Unit weight in pounds per cubic foot

SHEAR TESTS

TYPE

- UC Unconfined compression shear
- OB Unconsolidated undrained triaxial compression shear on one specimen confined at the approximate overburden pressure
- UU Unconsolidated undrained triaxial compression shear
- ϕ Angle of internal friction in degrees
- c Cohesion in pounds per square foot

ATTERBERG LIMITS

- LL Liquid Limit
- PL Plastic Limit
- PI Plasticity Index

OTHER TESTS

- CON Consolidation
- -#200 Percent passing a U.S. No. 200 sieve
- SV Particle size distribution (sieve only)
- PD Particle size distribution (sieve and hydrometer)
- k Coefficient of permeability in centimeters per second
- SP Swelling pressure in pounds per square foot

Other laboratory test results reported on separate figures

GENERAL NOTES

(1) If a ground water depth is shown on the boring log, these observations were made at the time of drilling and were measured below the existing ground surface. These observations are shown on the boring logs. However, ground water levels may vary due to seasonal fluctuations and other factors. If important to construction, the depth to ground water should be determined by those persons responsible for construction immediately prior to beginning work.

(2) While the individual logs of borings are considered to be representative of subsurface conditions at their respective locations on the dates shown, it is not warranted that they are representative of subsurface conditions at other locations and times.

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: A-1

Project No: G0389
Date: 01/31/2019
Latitude: 30.23799°
Longitude: -89.56737°
Water Depth: See Text
Total Depth: 5.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Loose tan fine sand w/roots, trace of fine gravel, shell fragments, clay pockets, & silt	SP	PB-1	0	16										
					Soft tan & gray weak red silty clay	CL	PB-2	1											
							PB-3	2	18										
					Medium stiff tan & light gray silty clay	CL	PB-4	3											
					w/trace of clay lenses & fine sand pockets		PB-5	4	28										
5																			
10																			
15																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: A-2

Project No: G0389
Date: 01/31/2019
Latitude: 30.23791°
Longitude: -89.56798°

Water Depth: See Text
Total Depth: 5.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Soft brown silty clay w/few crushed limestone	CL	PB-1	0											
					w/trace of fine sand & organic matter		PB-2	1	12										
							PB-3	2											
					Soft brown sandy clay	CL	PB-4	3	14										
					Soft brown & tan silty clay w/clay lenses	CL	PB-5	4											
5																			
10																			
15																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: A-3

Project No: G0389
Date: 01/31/2019
Latitude: 30.23757°
Longitude: -89.56370°

Water Depth: See Text
Total Depth: 5.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Medium dense tan & gray silty sand w/crushed limestone	SM	PB-1	0	3										
					Medium stiff tan sandy clay	CL	PB-2	1											
							PB-3	2	17										
					Medium stiff tan silty clay	CL	PB-4	3											
					w/fine sand pockets & trace of concretions		PB-5	4	17										
5																			
10																			
15																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

EUSTIS ENGINEERING SINCE 1946

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: A-4

Project No: G0389
Date: 01/31/2019
Latitude: 30.23564°
Longitude: -89.56470°

Water Depth: See Text
Total Depth: 5.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Loose brown silty sand w/clay, crushed limestone, & shells	SM	PB-1	0											
					Loose tan clayey silt w/fine sand & few clay pockets	ML	PB-2	1	15										
					Soft tan sandy clay	CL	PB-3	2											
							PB-4	3	16										
					Medium stiff tan silty clay	CL	PB-5	4											
5																			
10																			
15																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

LOG OF BORING AND TEST RESULTS

Hancock County
Port and Harbor Commission
Restore Project
Proposed Facility
Port Bienville, Mississippi

Boring: A-5

Project No: G0389
Date: 01/31/2019
Latitude: 30.23571°
Longitude: -89.56469°

Water Depth: See Text
Total Depth: 5.0 ft

Scale in Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth in Feet	Water Content %	Density Dry pcf	Density Wet pcf	Shear Tests Type	Shear Tests ϕ	Shear Tests C psf	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Loose gray medium to fine sand w/trace of clay pockets & roots	SP	PB-1	0	14										
					Soft brown & tan silty clay w/trace of sand	CL	PB-2	1											
					w/few fine sand		PB-3	2	16										
							PB-4	3											
					w/trace of fine sand & organic matter		PB-5	4	17										
5																			
10																			
15																			

NOTES:

EUSTIS GINT LIBRARY012017.GLB EE STANDARD BORING LOG G0389.GPJ 2/15/19

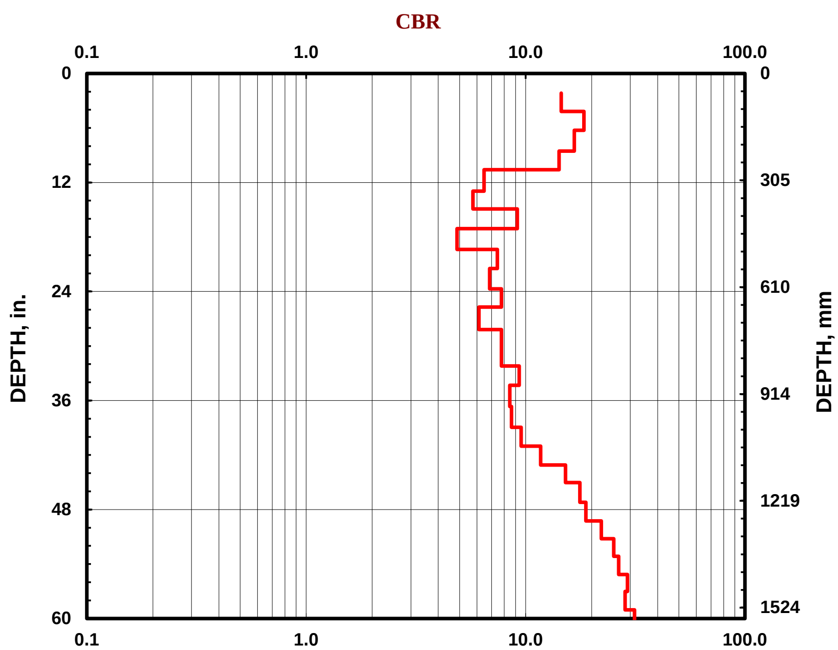
DCP TEST DATA

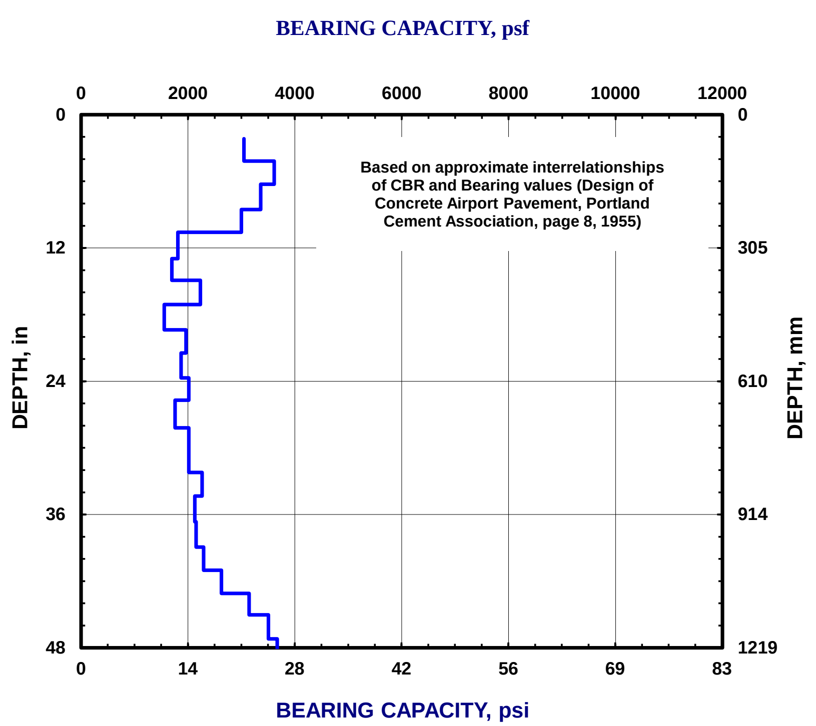
Project No.: *EE G0389 - Port Bienville* **Date:** 31-Jan-19
Location: DCPT-1 **Soil Type(s):** 0' to 1' (SP), 1' to 4' (CL)

Hammer
- (selected) 10.1 lbs.
- 17.6 lbs.
- Both hammers used

Soil Type
- CH
- CL
- (selected) All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
8	55	2
7	106	2
9	159	2
9	217	2
7	269	2
4	329	2
3	379	2
5	434	2
3	492	2
4	545	2
4	602	2
4	653	2
4	716	2
4	767	2
4	818	2
5	872	2
5	931	2
5	989	2
5	1042	2
6	1095	2
7	1144	2
9	1199	2
9	1251	2
10	1301	2
11	1350	2
12	1401	2
12	1448	2
13	1500	2
15	1555	2





DCP TEST DATA

Project No.: *EE G0389 - Port Bienville* Date: 31-Jan-19
Location: DCPT-2 Soil Type(s): Low plasticity Clay with CBR<10

Hammer
- ○ 10.1 lbs.
- ○ 17.6 lbs.
- ◉ Both hammers used

Soil Type
- ○ CH
- ◉ CL
- ○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
20	53	2
6	103	1
9	154	1
14	206	1
15	257	1
16	306	1
11	359	1
7	410	1
5	461	1
4	512	1
8	565	2
8	621	2
8	674	2
7	729	2
5	784	2
5	832	2
6	888	2
5	939	2
5	997	2
6	1051	2
8	1102	2
7	1145	2
7	1193	2
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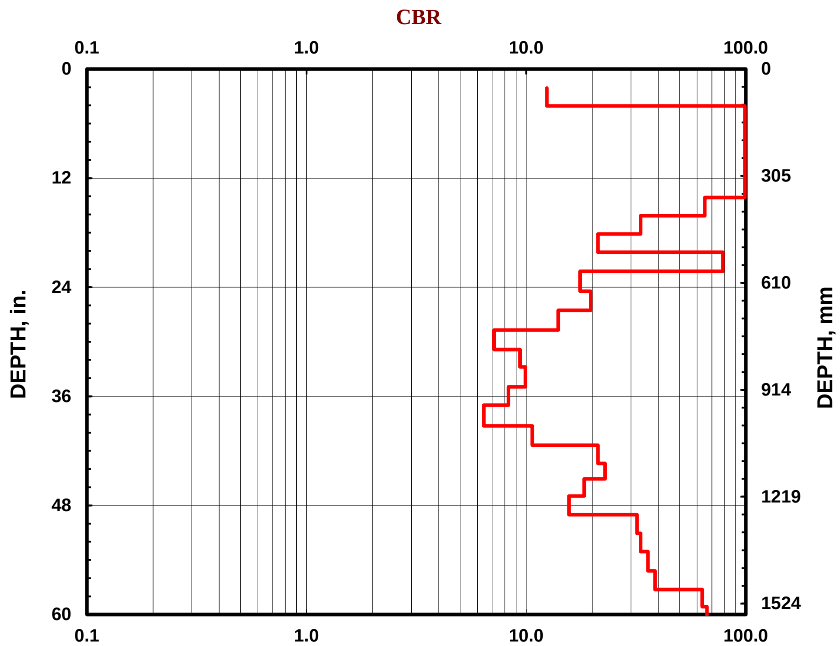
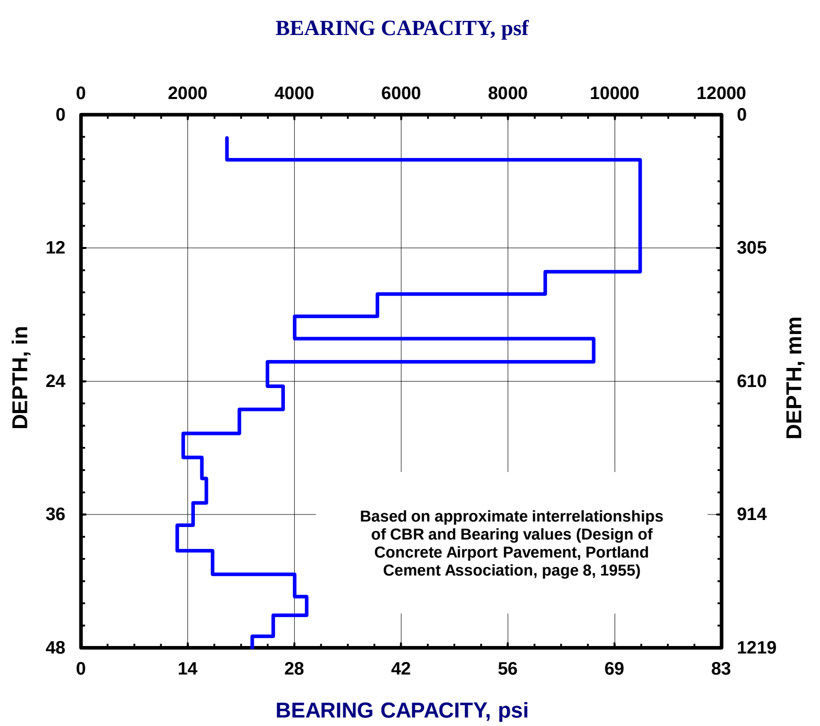
10	1297	2
10	1348	2
11	1402	2
11	1454	2
13	1502	2
15	1556	2





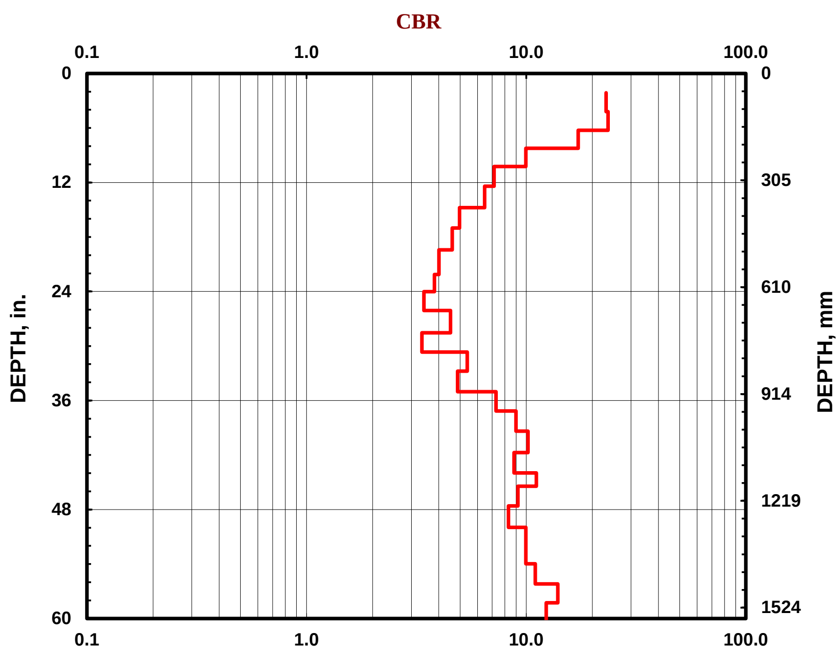
DCP TEST DATA

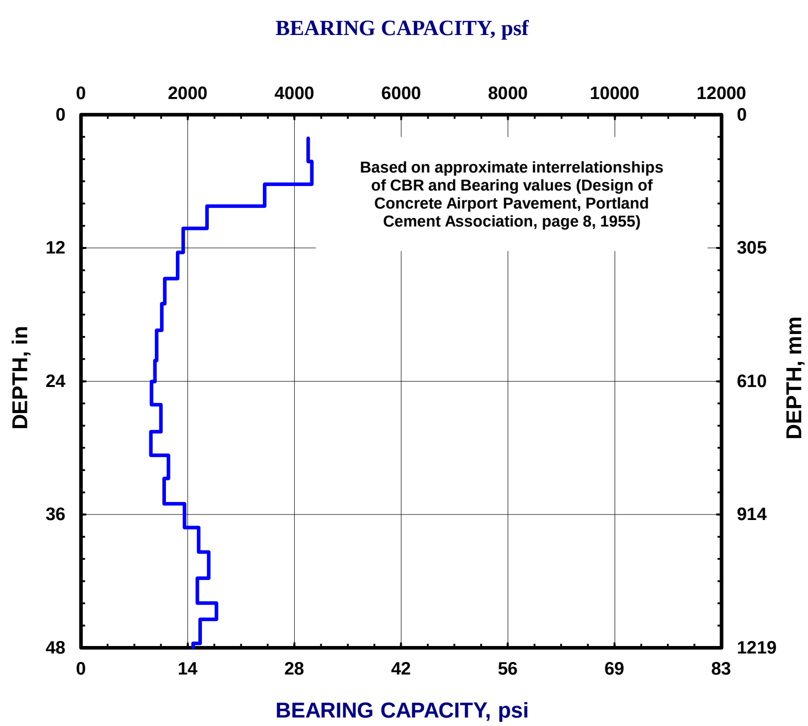
Project No.: *EE G0389 - Port Bienville*
Date: 31-Jan-19
Location: DCPT-3
Soil Type(s): 0' t0 1' (SM), 1' to 4' (CL)

Hammer
- (●) 10.1 lbs.
- ( ) 17.6 lbs.
- ( ) Both hammers used

Soil Type
- ( ) CH
- ( ) CL
- (●) All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
13	54	2
11	107	2
11	159	2
8	209	2
5	260	2
4	315	2
4	375	2
3	432	2
3	493	2
3	562	2
2	610	2
2	663	2
3	725	2
2	779	2
3	832	2
3	890	2
4	944	2
5	1000	2
6	1060	2
5	1117	2
4	1154	2
5	1209	2
5	1269	2
5	1320	2
5	1371	2
6	1427	2
7	1480	2
7	1539	2
5	1593	2





DCP TEST DATA

Project No.: *EE G0389 - Port Bienville* **Date:** 31-Jan-19

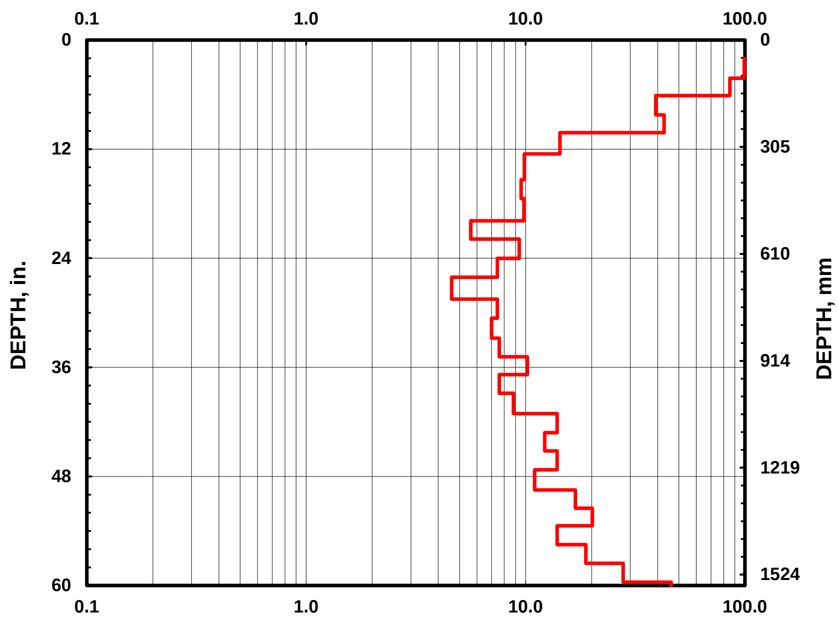
Location: DCPT-4 **Soil Type(s):** 0' t0 1' (SM), 1' to 2' (ML), 2' to 5' (CL)

Hammer
- ○ 10.1 lbs.
- ○ 17.6 lbs.
- ◉ Both hammers used

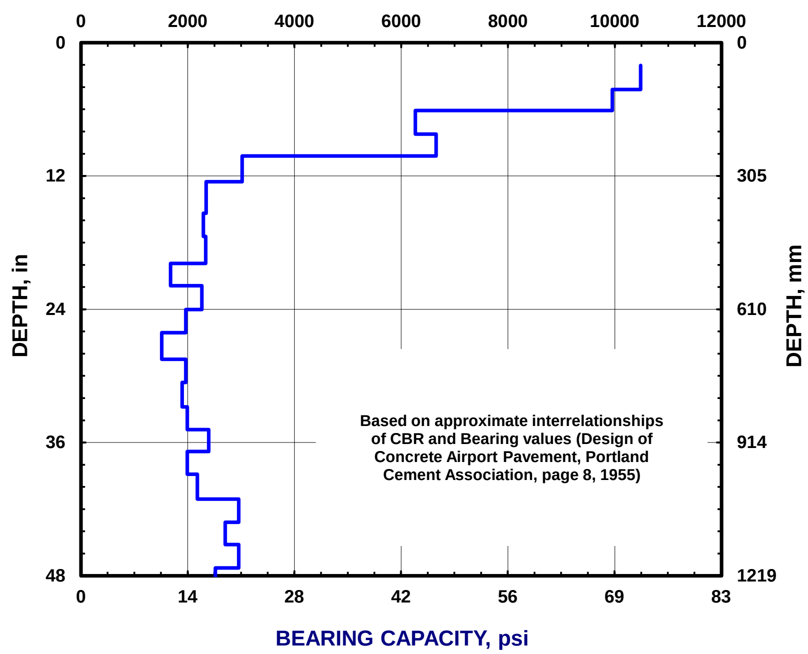
Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
16	52	1
21	107	1
16	155	1
9	209	1
9	259	2
8	318	2
7	390	2
5	443	2
6	505	2
3	556	2
5	610	2
4	663	2
3	724	2
4	777	2
4	833	2
4	885	2
5	935	2
4	987	2
5	1044	2
7	1097	2
6	1148	2
7	1201	2
6	1257	2
8	1308	2
9	1357	2
7	1410	2
9	1462	2
13	1515	2
20	1567	2

CBR



BEARING CAPACITY, psf



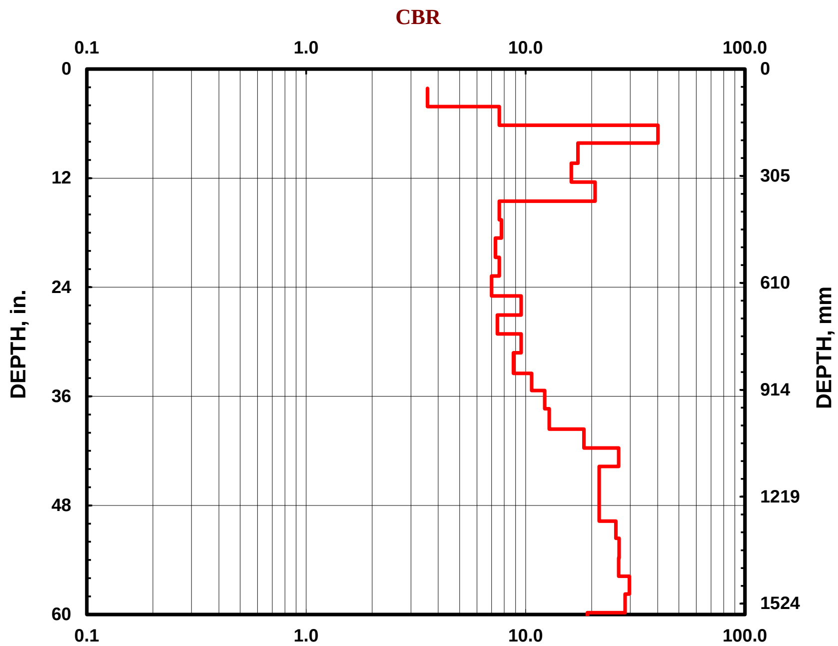
DCP TEST DATA

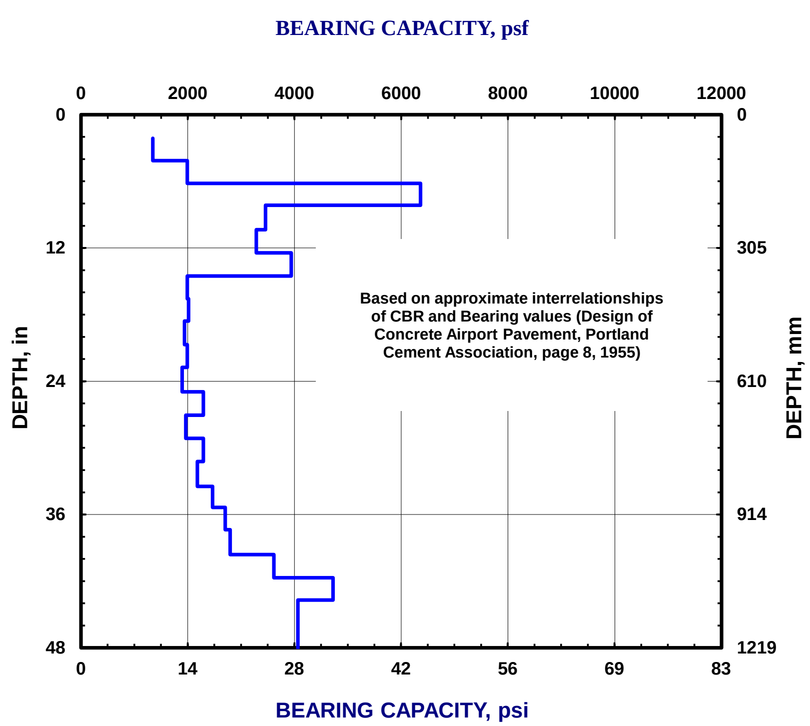
Project No.: *EE G0389 - Port Bienville* **Date:** 31-Jan-19
Location: DCPT-5 **Soil Type(s):** 0' to 1' (SP), 1' to 5' (CL)

Hammer
- ○ 10.1 lbs.
- ○ 17.6 lbs.
- ◉ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
1	54	2
2	105	2
4	157	2
17	207	2
9	263	1
4	316	1
5	369	2
4	421	2
4	472	2
4	526	2
4	578	2
4	634	2
5	687	2
4	740	2
5	793	2
5	850	2
5	898	2
6	949	2
7	1006	2
9	1059	2
12	1110	2
10	1161	2
10	1212	2
10	1263	2
11	1311	2
13	1366	2
12	1417	2
13	1467	2
13	1519	2
10	1576	2
7	1626	2





APPENDIX IV

Pace Analytical®
www.pacelabs.com

Pace Analytical Services, LLC
1000 Riverbend Blvd - Suite F
St. Rose, LA 70087
(504)469-0333

January 23, 2019

Ryan Rodrigue
Eustis Engineering
3011 28th Street
Metairie, LA 70002

RE: Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Dear Ryan Rodrigue:

Enclosed are the analytical results for sample(s) received by the laboratory on January 21, 2019. The results relate only to the samples included in this report. Results reported herein conform to the most current, applicable TNI/NELAC standards and the laboratory's Quality Assurance Manual, where applicable, unless otherwise noted in the body of the report.

If you have any questions concerning this report, please feel free to contact me.

Sincerely,

Karen H Brown

Karen Brown
karen.brown@pacelabs.com
(504)469-0333
Project Manager

Enclosures

cc: Ben Cody, Eustis Engineering

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(504)469-0333

CERTIFICATIONS

Project:	Hancock County Port/Harbor
Pace Project No.:	2094075

New Orleans Certification IDs

California Env. Lab Accreditation Program Branch: 11277CA
Florida Department of Health (NELAC): E87595
Illinois Environmental Protection Agency: 0025721
Kansas Department of Health and Environment (NELAC): E-10266
Louisiana Dept. of Environmental Quality (NELAC/LELAP): 02006
Pennsylviania Dept. of Env Protection (NELAC): 68-04202
Texas Commission on Env. Quality (NELAC): T104704405-09-TX
U.S. Dept. of Agriculture Foreign Soil Import: P330-10-00119
Commonwealth of Virginia (TNI): 480246

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SAMPLE SUMMARY

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Lab ID	Sample ID	Matrix	Date Collected	Date Received
2094075001	**Composite # 1**	Solid	01/21/19 12:00	01/21/19 14:00
2094075002	**Composite # 2**	Solid	01/21/19 12:00	01/21/19 14:00

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SAMPLE ANALYTE COUNT

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Lab ID	Sample ID	Method	Analysts	Analytes Reported	Laboratory
2094075001	**Composite # 1**	EPA 9045	DPF	1	PASI-N
		EPA 9050	DPF	1	PASI-N
		EPA 9038	MHM	1	PASI-N
		EPA 9251	MHM	1	PASI-N
2094075002	**Composite # 2**	EPA 9045	DPF	1	PASI-N
		EPA 9050	DPF	1	PASI-N
		EPA 9038	MHM	1	PASI-N
		EPA 9251	MHM	1	PASI-N

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PROJECT NARRATIVE

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Method: **EPA 9045**
Description: 9045 pH Soil
Client: Eustis Engineering
Date: January 23, 2019

General Information:
2 samples were analyzed for EPA 9045. All samples were received in acceptable condition with any exceptions noted below or on the chain-of custody and/or the sample condition upon receipt form (SCUR) attached at the end of this report.

Hold Time:
The samples were analyzed within the method required hold times with any exceptions noted below.

Method Blank:
All analytes were below the report limit in the method blank, where applicable, with any exceptions noted below.

Laboratory Control Spike:
All laboratory control spike compounds were within QC limits with any exceptions noted below.

Matrix Spikes:
All percent recoveries and relative percent differences (RPDs) were within acceptance criteria with any exceptions noted below.

Duplicate Sample:
All duplicate sample results were within method acceptance criteria with any exceptions noted below.

Additional Comments:

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PROJECT NARRATIVE

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Method: **EPA 9050**
Description: 9050 Specific Conductance
Client: Eustis Engineering
Date: January 23, 2019

General Information:
2 samples were analyzed for EPA 9050. All samples were received in acceptable condition with any exceptions noted below or on the chain-of custody and/or the sample condition upon receipt form (SCUR) attached at the end of this report.

Hold Time:
The samples were analyzed within the method required hold times with any exceptions noted below.

Method Blank:
All analytes were below the report limit in the method blank, where applicable, with any exceptions noted below.

Laboratory Control Spike:
All laboratory control spike compounds were within QC limits with any exceptions noted below.

Matrix Spikes:
All percent recoveries and relative percent differences (RPDs) were within acceptance criteria with any exceptions noted below.

Duplicate Sample:
All duplicate sample results were within method acceptance criteria with any exceptions noted below.

Additional Comments:

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PROJECT NARRATIVE

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Method: **EPA 9038**
Description: 9038 Sulfate, Turbidimetric
Client: Eustis Engineering
Date: January 23, 2019

General Information:
2 samples were analyzed for EPA 9038. All samples were received in acceptable condition with any exceptions noted below or on the chain-of custody and/or the sample condition upon receipt form (SCUR) attached at the end of this report.

Hold Time:
The samples were analyzed within the method required hold times with any exceptions noted below.

Sample Preparation:
The samples were prepared in accordance with EPA 9038 with any exceptions noted below.

Method Blank:
All analytes were below the report limit in the method blank, where applicable, with any exceptions noted below.

Laboratory Control Spike:
All laboratory control spike compounds were within QC limits with any exceptions noted below.

Matrix Spikes:
All percent recoveries and relative percent differences (RPDs) were within acceptance criteria with any exceptions noted below.

Duplicate Sample:
All duplicate sample results were within method acceptance criteria with any exceptions noted below.

Additional Comments:

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PROJECT NARRATIVE

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Method: **EPA 9251**
Description: 9251 Chloride
Client: Eustis Engineering
Date: January 23, 2019

General Information:
2 samples were analyzed for EPA 9251. All samples were received in acceptable condition with any exceptions noted below or on the chain-of custody and/or the sample condition upon receipt form (SCUR) attached at the end of this report.

Hold Time:
The samples were analyzed within the method required hold times with any exceptions noted below.

Sample Preparation:
The samples were prepared in accordance with EPA 9251 with any exceptions noted below.

Method Blank:
All analytes were below the report limit in the method blank, where applicable, with any exceptions noted below.

Laboratory Control Spike:
All laboratory control spike compounds were within QC limits with any exceptions noted below.

Matrix Spikes:
All percent recoveries and relative percent differences (RPDs) were within acceptance criteria with any exceptions noted below.

Duplicate Sample:
All duplicate sample results were within method acceptance criteria with any exceptions noted below.

Additional Comments:

This data package has been reviewed for quality and completeness and is approved for release.

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ANALYTICAL RESULTS

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Sample: Composite # 1 **Lab ID: 2094075001** Collected: 01/21/19 12:00 Received: 01/21/19 14:00 Matrix: Solid

Results reported on a "wet-weight" basis

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
9045 pH Soil	Analytical Method: EPA 9045							
pH at 25 Degrees C	**5.6**	Std. Units	0.010	1		01/22/19 15:44		
9050 Specific Conductance	Analytical Method: EPA 9050							
Specific Conductance	**307**	umhos/cm	1.0	1		01/22/19 18:38		
9038 Sulfate, Turbidimetric	Analytical Method: EPA 9038 Preparation Method: EPA 9038							
Sulfate	**9100**	mg/kg	2420	50	01/22/19 09:10	01/22/19 12:49	14808-79-8	M6
9251 Chloride	Analytical Method: EPA 9251 Preparation Method: EPA 9251							
Chloride	**1100**	mg/kg	96.8	10	01/22/19 09:16	01/22/19 11:46	16887-00-6	

Sample: Composite # 2 **Lab ID: 2094075002** Collected: 01/21/19 12:00 Received: 01/21/19 14:00 Matrix: Solid

Results reported on a "wet-weight" basis

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
9045 pH Soil	Analytical Method: EPA 9045							
pH at 25 Degrees C	**6.0**	Std. Units	0.010	1		01/22/19 15:54		
9050 Specific Conductance	Analytical Method: EPA 9050							
Specific Conductance	**276**	umhos/cm	1.0	1		01/22/19 18:38		
9038 Sulfate, Turbidimetric	Analytical Method: EPA 9038 Preparation Method: EPA 9038							
Sulfate	**10400**	mg/kg	2370	50	01/22/19 09:10	01/22/19 12:49	14808-79-8	
9251 Chloride	Analytical Method: EPA 9251 Preparation Method: EPA 9251							
Chloride	**2230**	mg/kg	95.0	10	01/22/19 09:16	01/22/19 11:46	16887-00-6	

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QUALITY CONTROL DATA

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

QC Batch: 131767 | Analysis Method: EPA 9045
QC Batch Method: EPA 9045 | Analysis Description: 9045 pH
Associated Lab Samples: 2094075001, 2094075002

LABORATORY CONTROL SAMPLE: 572124

Parameter	Units	Spike Conc.	LCS Result	LCS % Rec	% Rec Limits	Qualifiers
pH at 25 Degrees C	Std. Units	6	6.0	100	97-103	

SAMPLE DUPLICATE: 572125

Parameter	Units	2094075001 Result	Dup Result	RPD	Max RPD	Qualifiers
pH at 25 Degrees C	Std. Units	5.6	5.5	1	20	

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

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QUALITY CONTROL DATA

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

QC Batch: 131768 — Analysis Method: EPA 9050
QC Batch Method: EPA 9050 — Analysis Description: 9050 Specific Conductance
Associated Lab Samples: 2094075001, 2094075002

METHOD BLANK: 572126 — Matrix: Solid
Associated Lab Samples: 2094075001, 2094075002

Parameter	Units	Blank Result	Reporting Limit	Analyzed	Qualifiers
Specific Conductance	umhos/cm	ND	1.0	01/22/19 18:38	

LABORATORY CONTROL SAMPLE: 572127

Parameter	Units	Spike Conc.	LCS Result	LCS % Rec	% Rec Limits	Qualifiers
Specific Conductance	umhos/cm	1410	1370	97	95-105	

SAMPLE DUPLICATE: 572128

Parameter	Units	2094075001 Result	Dup Result	RPD	Max RPD	Qualifiers
Specific Conductance	umhos/cm	307	305	1	20	

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

REPORT OF LABORATORY ANALYSIS

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QUALITY CONTROL DATA

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

QC Batch: 131689 | Analysis Method: EPA 9038
QC Batch Method: EPA 9038 | Analysis Description: 9038 Sulfate, Turbidimetric
Associated Lab Samples: 2094075001, 2094075002

METHOD BLANK: 571830 | Matrix: Solid
Associated Lab Samples: 2094075001, 2094075002

Parameter	Units	Blank Result	Reporting Limit	Analyzed	Qualifiers
Sulfate	mg/kg	ND	50.0	01/22/19 12:44	

LABORATORY CONTROL SAMPLE: 571831

Parameter	Units	Spike Conc.	LCS Result	LCS % Rec	% Rec Limits	Qualifiers
Sulfate	mg/kg	200	211	106	90-110	

MATRIX SPIKE SAMPLE: 571833

Parameter	Units	2094075001 Result	Spike Conc.	MS Result	MS % Rec	% Rec Limits	Qualifiers
Sulfate	mg/kg	9100	4940	9560	9	75-125	M6

SAMPLE DUPLICATE: 571832

Parameter	Units	2094075001 Result	Dup Result	RPD	Max RPD	Qualifiers
Sulfate	mg/kg	9100	8100	12	20	

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

Pace Analytical Services, LLC
1000 Riverbend Blvd - Suite F
St. Rose, LA 70087
(504)469-0333

QUALITY CONTROL DATA

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

QC Batch: 131688 — Analysis Method: EPA 9251
QC Batch Method: EPA 9251 — Analysis Description: 9251 Chloride
Associated Lab Samples: 2094075001, 2094075002

METHOD BLANK: 571826 — Matrix: Solid
Associated Lab Samples: 2094075001, 2094075002

Parameter	Units	Blank Result	Reporting Limit	Analyzed	Qualifiers
Chloride	mg/kg	ND	10.0	01/22/19 11:34	

LABORATORY CONTROL SAMPLE: 571827

Parameter	Units	Spike Conc.	LCS Result	LCS % Rec	% Rec Limits	Qualifiers
Chloride	mg/kg	829	772	93	90-110	

MATRIX SPIKE SAMPLE: 571829

Parameter	Units	2094075001 Result	Spike Conc.	MS Result	MS % Rec	% Rec Limits	Qualifiers
Chloride	mg/kg	1100	9870	13100	122	75-125	

SAMPLE DUPLICATE: 571828

Parameter	Units	2094075001 Result	Dup Result	RPD	Max RPD	Qualifiers
Chloride	mg/kg	1100	1260	13	20	

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

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QUALIFIERS

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

DEFINITIONS

DF - Dilution Factor, if reported, represents the factor applied to the reported data due to dilution of the sample aliquot.

ND - Not Detected at or above adjusted reporting limit.

TNTC - Too Numerous To Count

J - Estimated concentration above the adjusted method detection limit and below the adjusted reporting limit.

MDL - Adjusted Method Detection Limit.

PQL - Practical Quantitation Limit.

RL - Reporting Limit - The lowest concentration value that meets project requirements for quantitative data with known precision and bias for a specific analyte in a specific matrix.

S - Surrogate

1,2-Diphenylhydrazine decomposes to and cannot be separated from Azobenzene using Method 8270. The result for each analyte is a combined concentration.

Consistent with EPA guidelines, unrounded data are displayed and have been used to calculate % recovery and RPD values.

LCS(D) - Laboratory Control Sample (Duplicate)

MS(D) - Matrix Spike (Duplicate)

DUP - Sample Duplicate

RPD - Relative Percent Difference

NC - Not Calculable.

SG - Silica Gel - Clean-Up

U - Indicates the compound was analyzed for, but not detected.

N-Nitrosodiphenylamine decomposes and cannot be separated from Diphenylamine using Method 8270. The result reported for each analyte is a combined concentration.

Pace Analytical is TNI accredited. Contact your Pace PM for the current list of accredited analytes.

TNI - The Nelac Institute

LABORATORIES

PASI-N Pace Analytical Services - New Orleans

ANALYTE QUALIFIERS

M6 Matrix spike and Matrix spike duplicate recovery not evaluated against control limits due to sample dilution.

REPORT OF LABORATORY ANALYSIS

Pace Analytical Services, LLC
1000 Riverbend Blvd - Suite F
St. Rose, LA 70087
(504)469-0333

QUALITY CONTROL DATA CROSS REFERENCE TABLE

Project: Hancock County Port/Harbor
Pace Project No.: 2094075

Lab ID	Sample ID	QC Batch Method	QC Batch	Analytical Method	Analytical Batch
2094075001	**Composite # 1**	EPA 9045	131767		
2094075002	**Composite # 2**	EPA 9045	131767		
2094075001	**Composite # 1**	EPA 9050	131768		
2094075002	**Composite # 2**	EPA 9050	131768		
2094075001	**Composite # 1**	EPA 9038	131689	EPA 9038	131721
2094075002	**Composite # 2**	EPA 9038	131689	EPA 9038	131721
2094075001	**Composite # 1**	EPA 9251	131688	EPA 9251	131718
2094075002	**Composite # 2**	EPA 9251	131688	EPA 9251	131718

WO#: 2094075

2094075

EUSTIS ENGINEERING LLC. SINCE 1946

Analysis Request a

327 LZ

Project No.	Project Name	Project Location
G0389	Hancock County Port + Harbor	Port Bienville, MS

Sample No./ Identification	Date and Time	Grab	Comp	Type Sample/ Container (Size/No.)	PRESERVATION Chemicals Added	PRESERVATION Iced	ANALYSIS REQUESTED TEST	ANALYSIS REQUESTED METHOD	LABORATORY REMARKS
Composite #1	1/21/19 12:00			Plastic Bag			PH, Specific Conductance, Sulfate, Chloride	EPA9045, EPA9050, EPA9038, EPA9051	
Composite #2	1/21/19 12:00			"			"	"	

Samplers: (Signature)	Relinquished by: (Signature)	Date / Time	Received by: (Signature)	Date / Time	Intact
	Sedric Stewart	Date: 01-21-19 Time: 14:00	[signature]	Date: 1-21-19 Time: 1400	yes ambient
	Relinquished by: (Signature)	Date: Time:	Received by: (Signature)	Date: Time:	Intact
Affiliation:	Relinquished by: (Signature)	Date: Time:	Received by: (Signature)	Date: Time:	Intact

REMARKS:

All results to:
BDeschamp@Eustiseng.com
RRodrigue @Eustiseng.com

Composite #1 - CSB-1/1+2, CSB-2/2+3, CSB 3/1+2

Composite #2 - CSB-1/3+4, CSB-2/4+5, CSB-3/3

Pace Analytical

Sample Condition Upon Re

1000 Riverbend Blvd., Suite F
St. Rose, LA 70087

Proje

WO#: 2094075

PM: KHB Due Date: 01/31/19
CLIENT: 20-EUSTIS

Courier: ☐ Pace Courier ☐ Hired Courier ☐ Fed X ☐ UPS ☐ DHL ☐ USPS ☑ Customer ☐ Other

Custody Seal on Cooler/Box Present: [see COC]

Custody Seals intact: ☐Yes ☑No

Therometer Used:	☐ Therm Fisher IR 5 ☐ Therm Fisher IR 6 ☐ Therm Fisher IR 7

Type of Ice: Wet Blue None

Samples on ice: [see COC]

Date and Initials of person examining contents: 1-21-19

Cooler Temperature: [see COC]

Temp should be above freezing to 6°C

Temp must be measured from Temperature blank when present

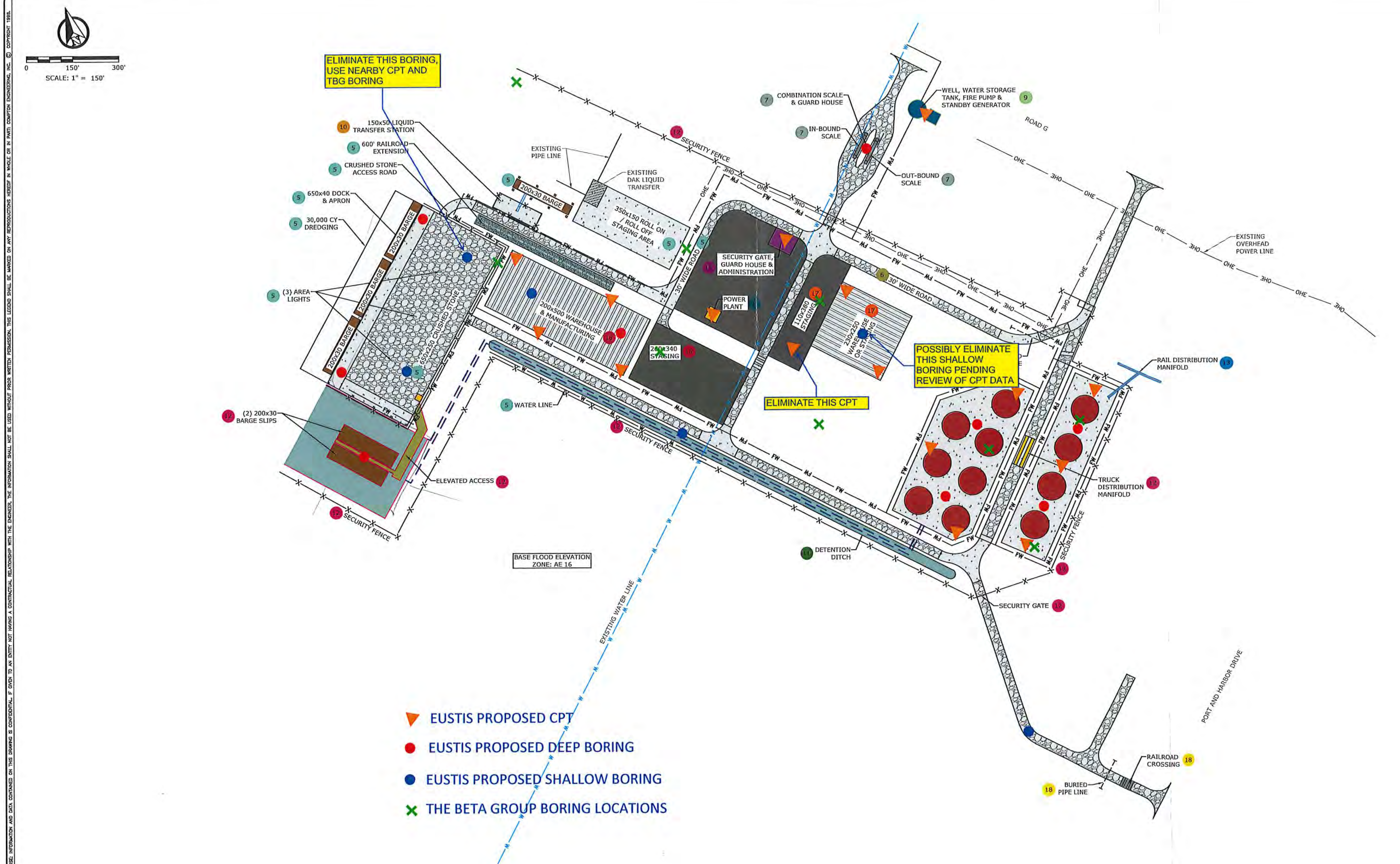
		Comments:
Temperature Blank Present"?	☐Yes ☑No ☐N/A	1
Chain of Custody Present:	☑Yes ☐No ☐N/A	2
Chain of Custody Complete:	☑Yes ☐No ☐N/A	3
Chain of Custody Relinquished:	☑Yes ☐No ☐N/A	4
Sampler Name & Signature on COC:	☐Yes ☑No ☐N/A	5
Samples Arrived within Hold Time:	☑Yes ☐No ☐N/A	6
Sufficient Volume:	☑Yes ☐No ☐N/A	7
Correct Containers Used:	☑Yes ☐No ☐N/A	8
Filtered vol. Rec. for Diss. tests	☐Yes ☑No ☐N/A	9
Sample Labels match COC:	☑Yes ☐No ☐N/A	10
All containers received within manafacture's precautionary and/or expiration dates.	☑Yes ☐No ☐N/A	11
All containers needing chemical preservation have been checked (except VOA, coliform, & O&G).	☐Yes ☐No ☑N/A	12
All containers preservation checked found to be in compliance with EPA recommendation.	☐Yes ☐No ☑N/A	13 If No, was preservative added? ☐Yes ☐No If added record lot no.: HNO3 ______ H2SO4 ______
Headspace in VOA Vials (>6mm):	☐Yes ☐No ☑N/A	14
Trip Blank Present:	☐Yes ☑No	15

Client Notification/ Resolution:

Person Contacted: ______________________ Date/Time: ____________

Comments/ Resolution: ______________________

APPENDIX V



1 / X1.0 PORT BIENVILLE DOCK PLAN
SCALE: 1" = 150'

COMPTON ENGINEERING, INC.
Engineering, Surveying, and Environmental Services
3036 Longfellow Drive
Bay St. Louis, Mississippi 39520
Phone: (228) 467-2770 Fax: (228) 467-2720
E-mail: bsl@comptonengineering.com

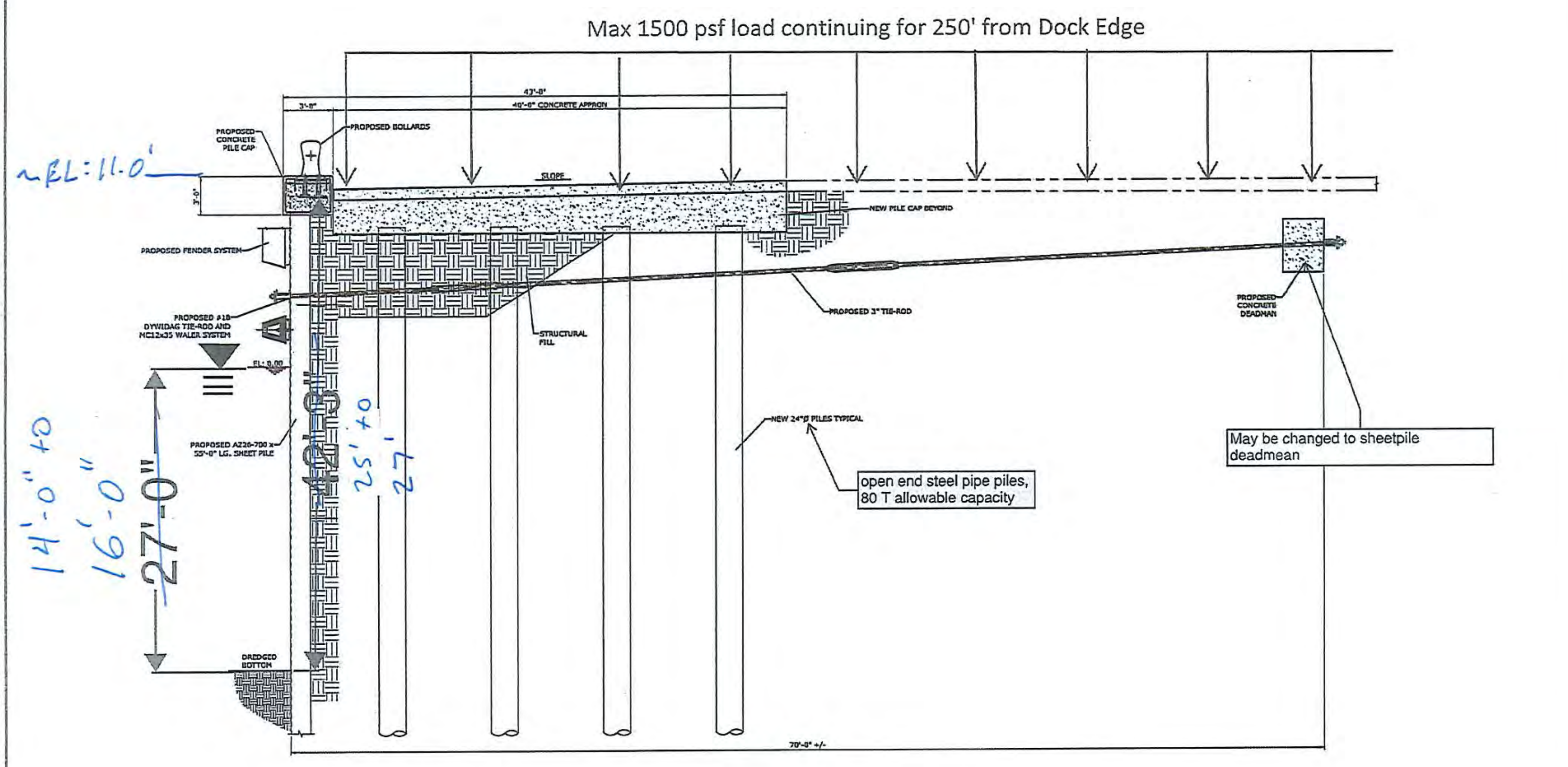
HANCOCK COUNTY PORT & HARBOR COMMISSION
PORT BIENVILLE, MISSISSIPPI

PORT BIENVILLE DOCK

SCALE:	AS NOTED
JOB NO.:	
DATE:	
DSGN.:	G. CLEMENS
DWG. BY:	B. LADNER
CHK.:	
APVD.:	

S-1

S:\Bay St Louis\0-PROJECTS\2018\218-032 - HCPHC - Support Services Docks and Waterways\218-032.004 - Site 4 Multi-Modal Facility\Dwgs\Current Design\Concept-Design\Xref\SLIDE 2 09-11-18.dwg, S-19, 10/16/2018 3:17:29 PM, tommyp



SCALE:	NTS
JOB NO.:	218-032
DATE:	
DSGN.:	JB
DWG. BY:	TP
CHK.:	
APVD.:	

SHEET OF

VERIFY SCALES

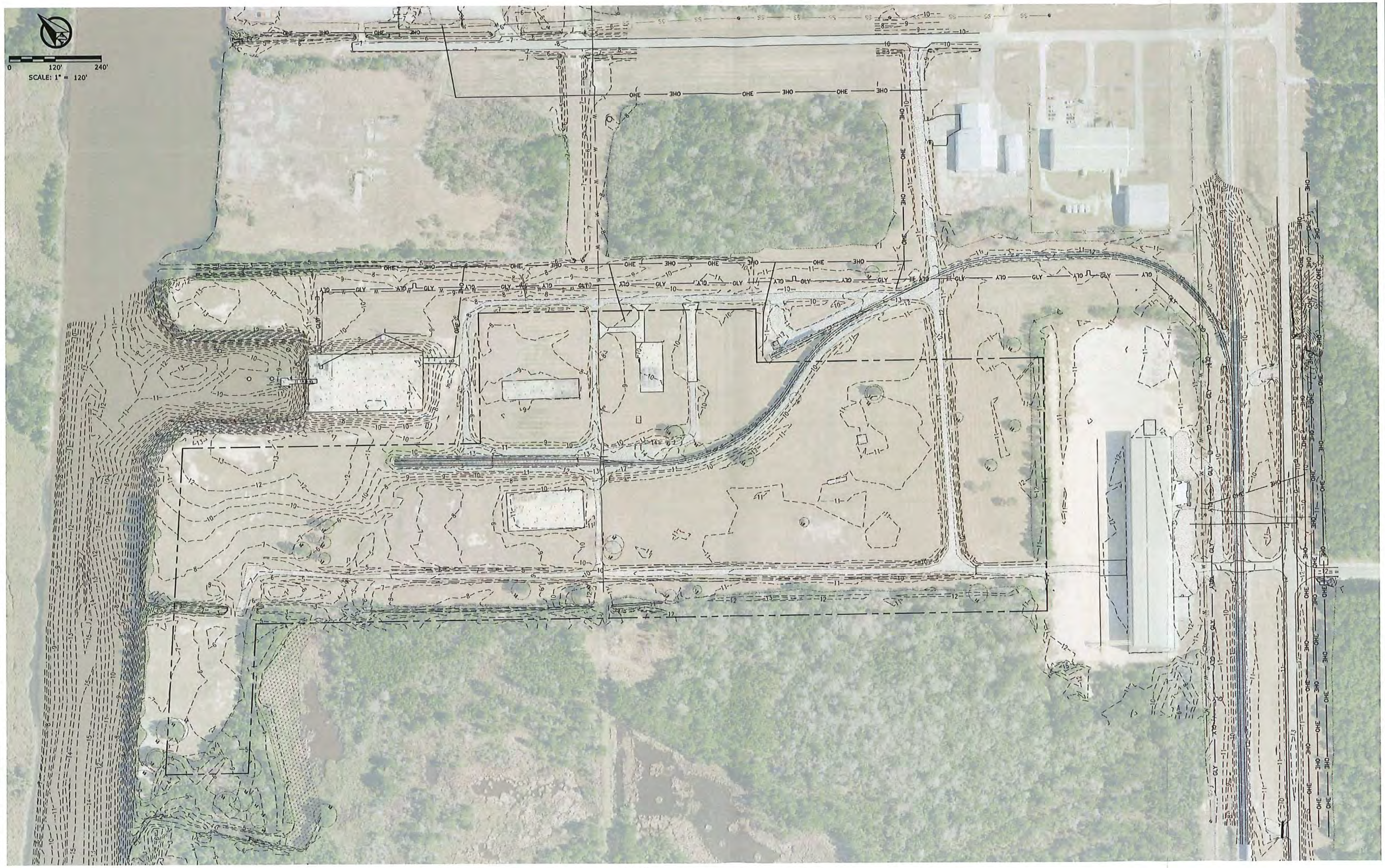
BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

0 1"

HANCOCK COUNTY PORT & HARBOR COMMISSION
PORT BIENVILLE, MISSISSIPPI

STRUCTURAL SCOPE

COMPTON ENGINEERING, INC.
Engineering, Surveying, and Environmental Services
3036 Longfellow Drive
Bay St. Louis, Mississippi 39520
Phone: (228) 467-2770 Fax: (228) 467-2720
E-mail: bsl@comptonengineering.com



1 C0.0 WETLAND CONTOUR MAP
SCALE: 1" - 120'-0"

COMPTON ENGINEERING, INC.
Engineering, Surveying, and Environmental Services
3036 Longfellow Drive
Bay St. Louis, Mississippi 39520
Phone: (228) 467-2770 Fax: (228) 467-2720
E-mail: bsl@comptonengineering.com

HANCOCK COUNTY PORT & HARBOR COMMISSION
PORT BIENVILLE, MISSISSIPPI

SITE 4 MULTI-MODAL FACILITY
WETLAND CONTOUR MAP

SCALE:	AS NOTED
JOB NO.:	218-032.004
DATE:	
DSGN.:	J. BURAS
DWG. BY:	B. LADNER
CHK.:	
APVD.:	

VERIFY SCALES

C0.0

Rev. No. A

DATE	DESCRIPTION	BY	APVD
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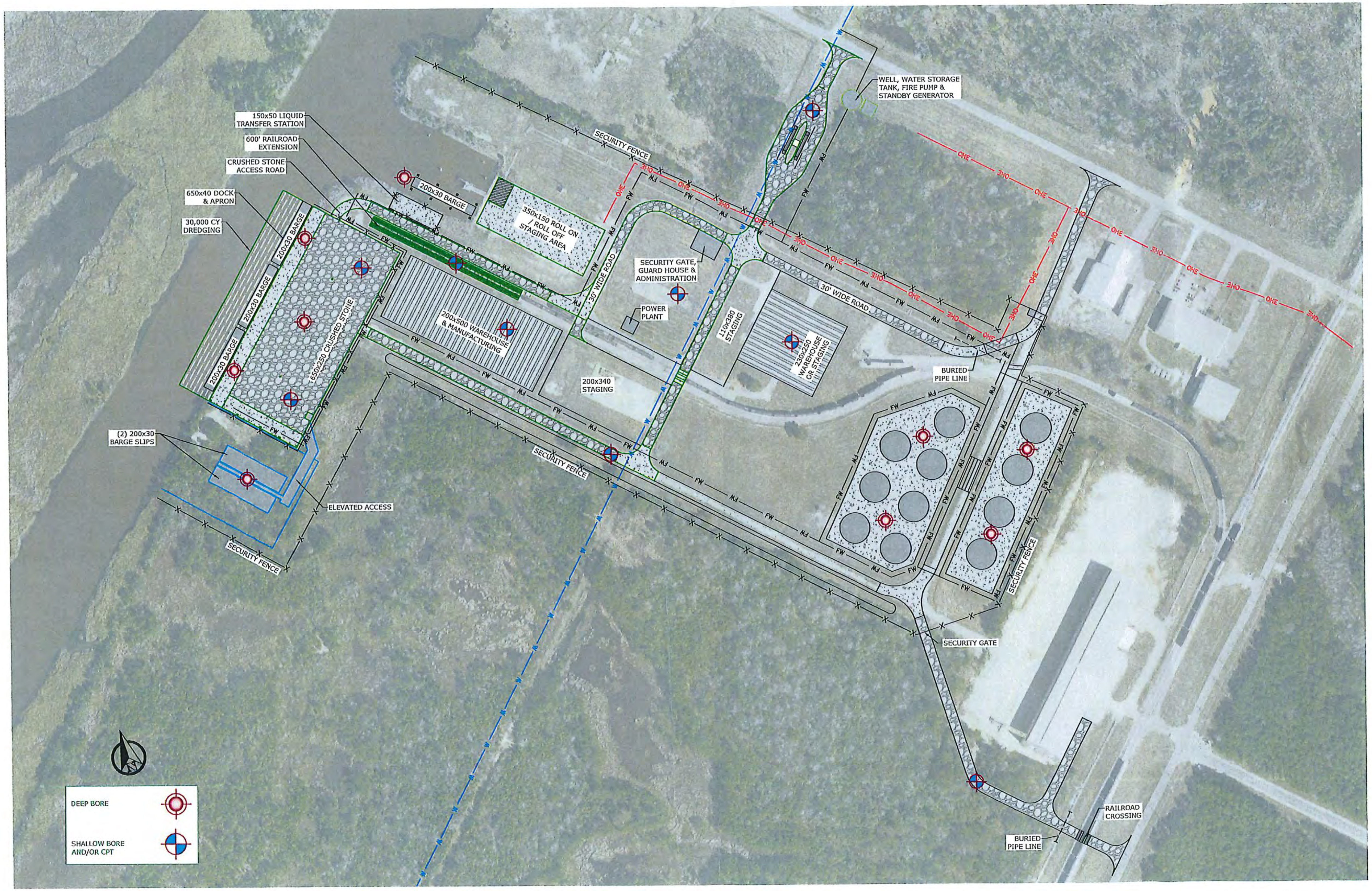


COMPTON ENGINEERING, INC.
Engineering, Surveying, and Environmental Services
3036 Longfellow Drive
Bay St. Louis, Mississippi 39520
Phone: (228) 467-2770 Fax: (228) 467-2720
E-mail: bsl@comptonengineering.com

HANCOCK COUNTY PORT & HARBOR COMMISSION
PORT BIENVILLE, MISSISSIPPI

BORING LOCATION MAP

SCALE:	AS NOTED
JOB NO.:	218-032.004
DATE:	
DSGN.:	JB
DWG. BY:	TP
CHK.:	
APVD.:	

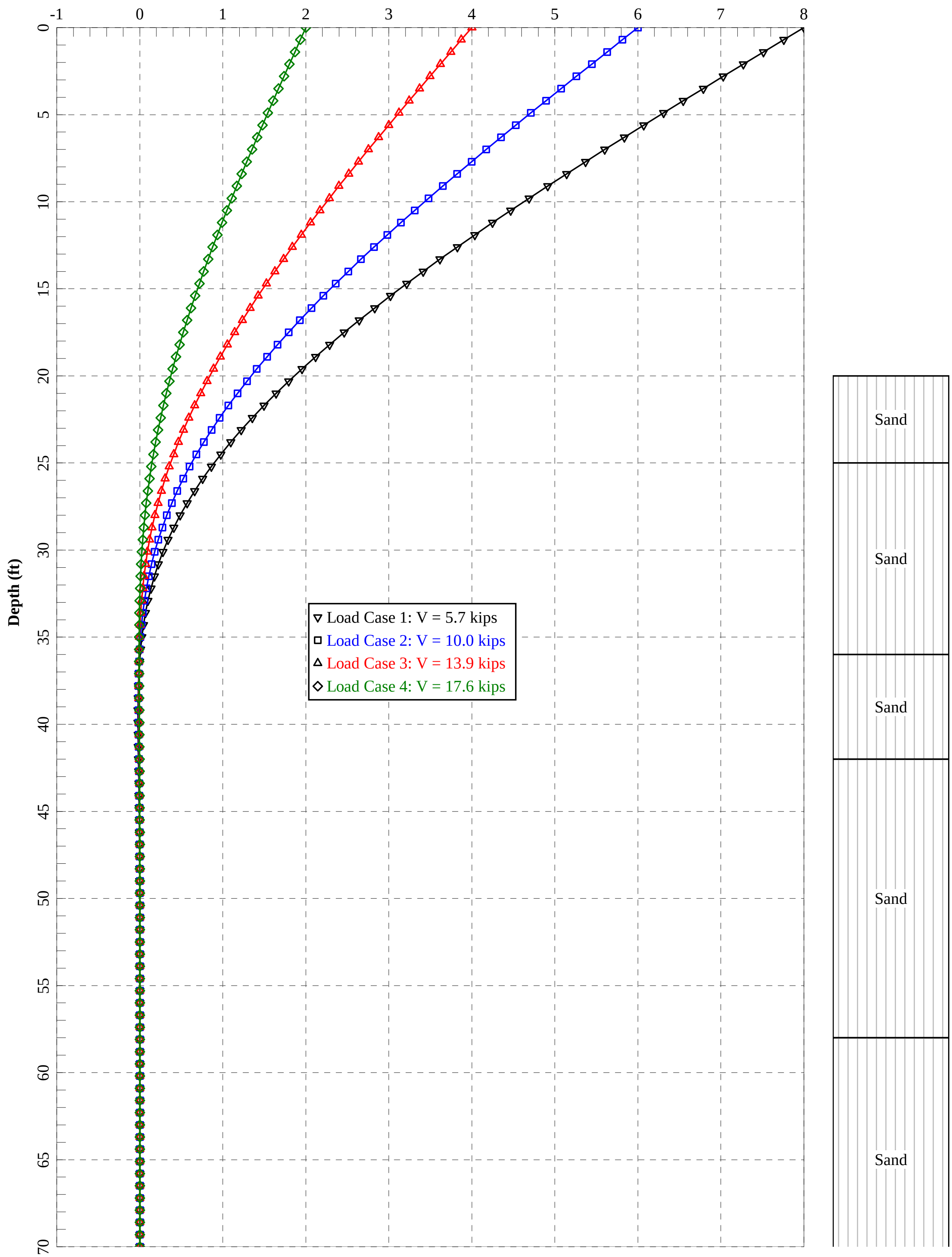
VERIFY SCALES
BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.
0 1"

Rev. No.
A

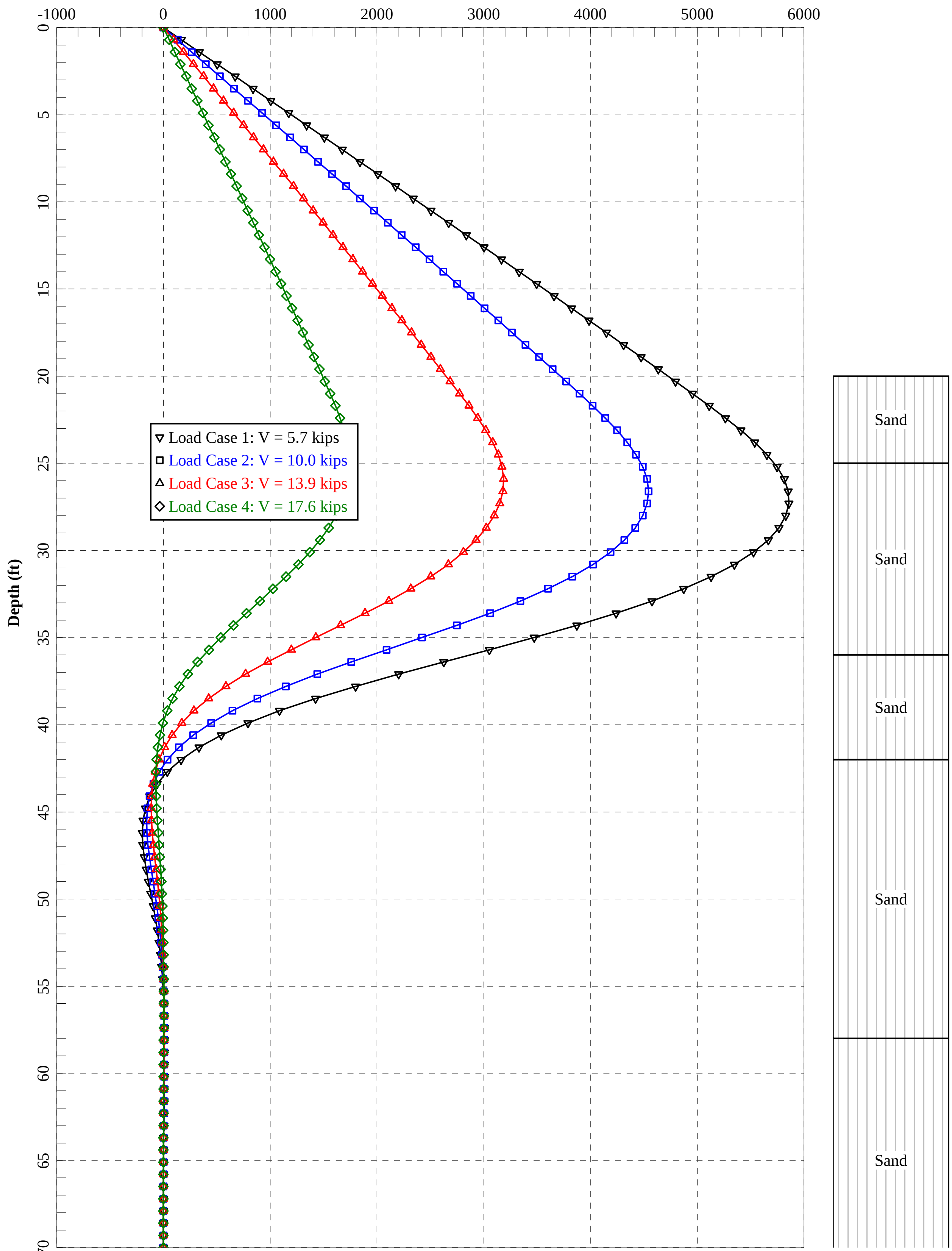
DATE	DESCRIPTION	BY	APVD

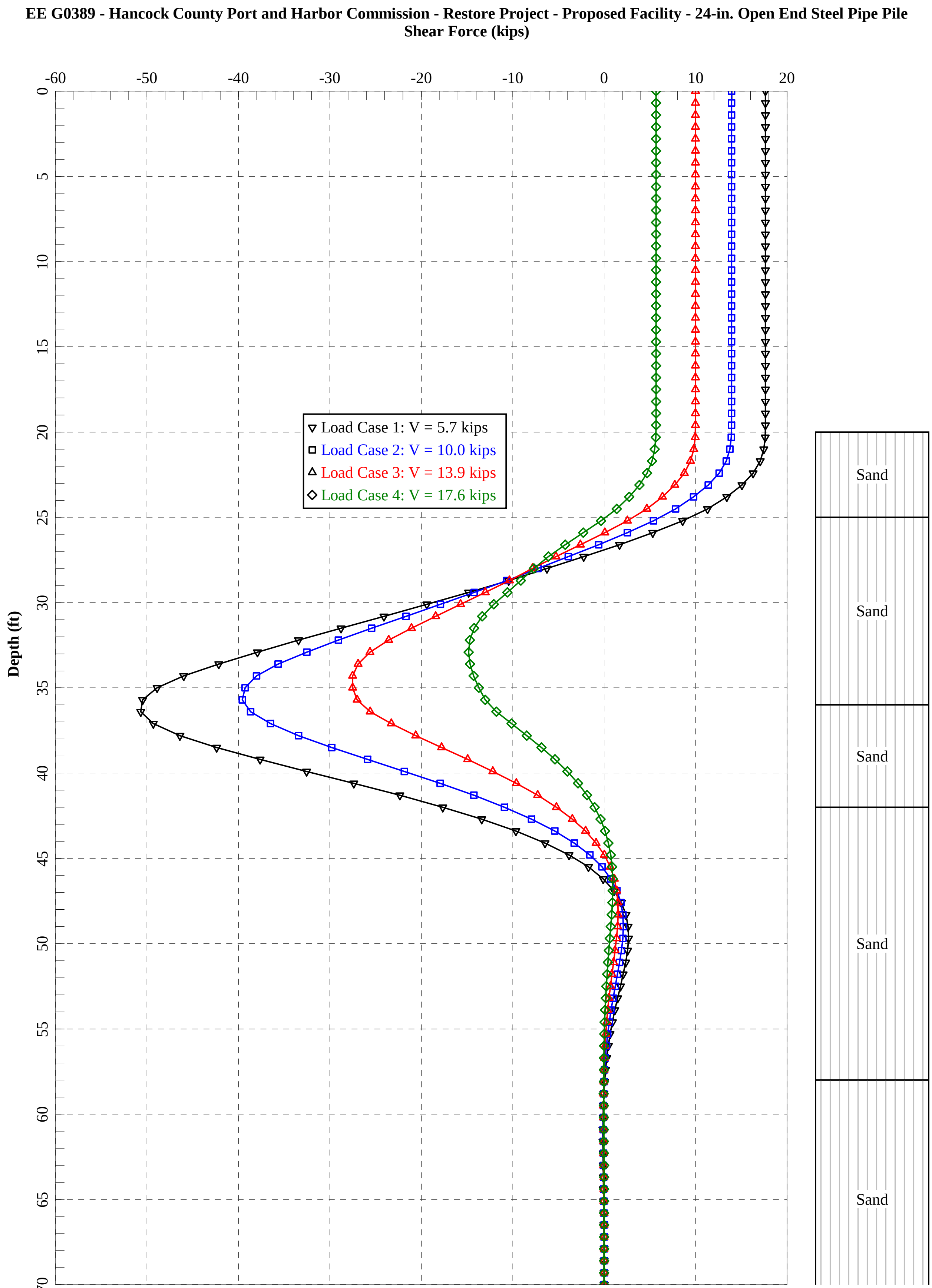
APPENDIX VI

EE G0389 - Hancock County Port and Harbor Commission - Restore Project - Proposed Facility - 24-in. Open End Steel Pipe Pile
Lateral Pile Deflection (inches)



EE G0389 - Hancock County Port and Harbor Commission - Restore Project - Proposed Facility - 24-in. Open End Steel Pipe Pile
Bending Moment (in-kips)




EE G0389 - Hancock County Port and Harbor Commission - Restore Project - Proposed Facility - 24-in. Open End Steel Pipe Pile
Shear Force (kips)
-60
-50
-40
-30
-20
-10
0
10
20
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Load Case 1: V = 5.7 kips
Load Case 2: V = 10.0 kips
Load Case 3: V = 13.9 kips
Load Case 4: V = 17.6 kips
Sand
Sand
Sand
Sand
Sand