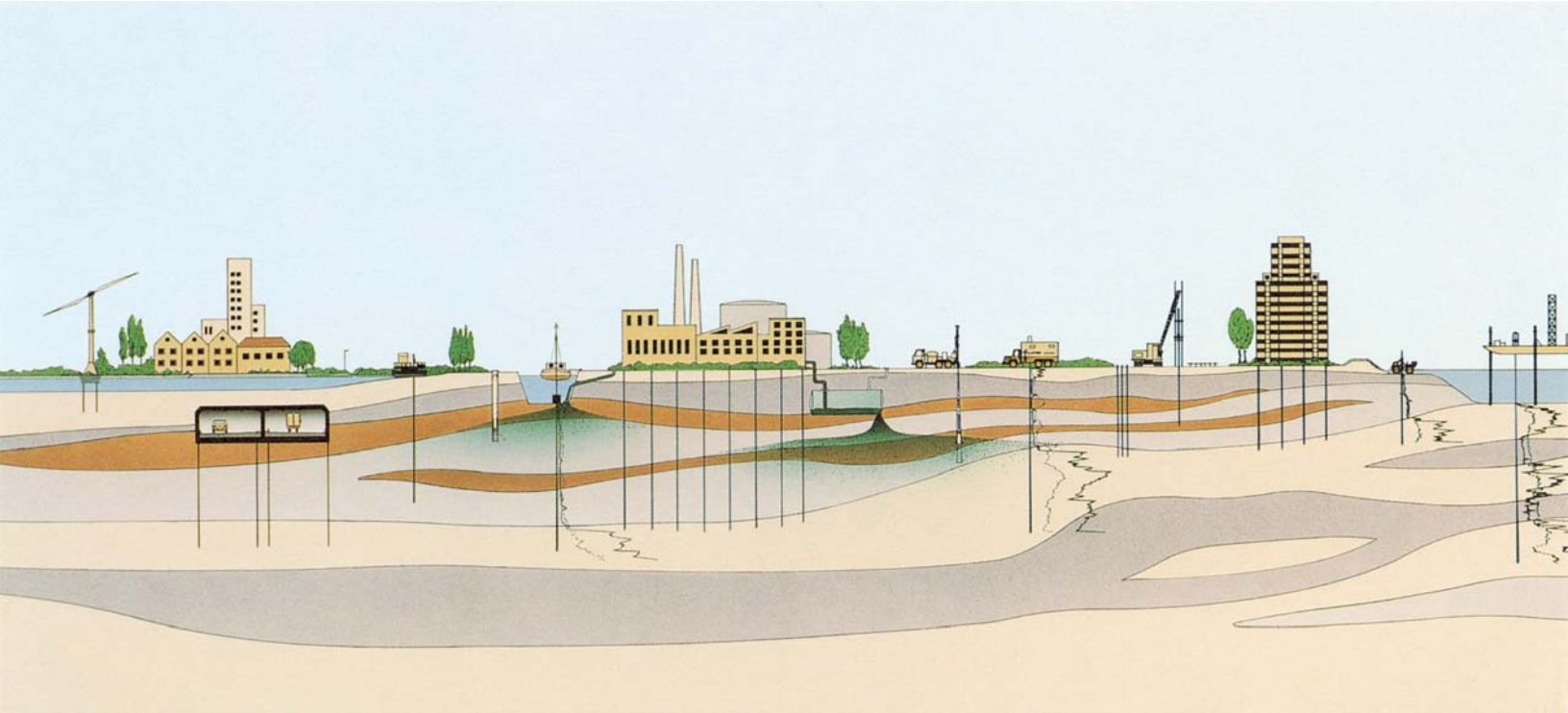
FUGRO CONSULTANTS, INC.

GEOTECHNICAL STUDY
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCKWOOD, ANDREWS & NEWNAM, INC.
HOUSTON, TEXAS



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Report No. 04.10120193
April 11, 2013

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Attention: Mr. E. Tyson Thomas, P.E.
Vice President

Geotechnical Study
Berth 6 and Bulkhead Wall Expansion
Port of Port Arthur
Port Arthur, Texas

Fugro Consultants, Inc. (Fugro) is pleased to present this report of our geotechnical study for the proposed Berth 6 and Bulkhead Wall Expansion at the Port of Port Arthur Facility along the Sabine-Neches Ship Channel in Port Arthur, Texas. Mr. Jon Jelinek, P.E. with Lockwood, Andrews & Newnam, Inc. (LAN) requested our services during a meeting with Mr. John Juenger, P.E. with Fugro on June 18, 2012. This study was performed in general accordance with our Proposal No. 04.10120193p-Rev. No. 1 dated August 9, 2012. Our services were authorized with LAN Work Authorization No. 120-10849-016-000 dated September 28, 2012.

This final report contains the results of our field and laboratory testing and our geotechnical recommendations for the proposed Berth 6 and Bulkhead Wall Expansion. This report also incorporates comments received from the Project Team on our preliminary report dated February 26, 2013. We appreciate the opportunity to be of continued service to LAN. Please call us if you have any questions with this report or when we may be of further service.

Sincerely,
FUGRO CONSULTANTS, INC.
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1.0 INTRODUCTION

1.1 Project Description

We understand that the Port of Port Arthur is planning to expand their existing wharf to include a new berth, designated Berth 6, at their facility along the Sabine-Neches Ship Channel in Port Arthur, Texas. A *Vicinity Map* of the project site is presented on Plate 1. The proposed Berth 6 will include construction of a 61.5-foot wide, 600-foot long extension of the existing pile supported wharf and extension of the existing bulkhead wall and accompanying anchor wall. The expansion will also include a 20-foot square breasting and mooring dolphin to be constructed about 300 feet from the straight-line projection of the end of the dock, and a 9-foot square deadman mooring dolphin constructed on the shoreline about 300 feet beyond the breasting dolphin. The proposed Berth 6 and supporting expansion is shown on our *Plan of Exploration* presented on Plate 2.

Based on the information provided to us by LAN, the Berth 6 deck will be supported on a combination of 24-inch driven pre-cast, pre-stressed (PCPS) concrete piles and 54-inch diameter hollow spun PCPS piles. Current plans are for the bulkhead wall to be a combi-wall system with 48-inch diameter “king-pile” pipe sections spaced on 102-inch centers infilled with steel sheet piling and tied-back to an A-frame anchor wall. We understand that the design surcharge load on the area behind the bulkhead wall will be approximately 1,000 psf. Consideration is being given to the use of a pile supported relieving platform behind the bulkhead wall.

The proposed design mudline elevation at the face of the proposed Berth 6 will be El. -48 feet plus an allowance for 2 feet of over-dredge. The mudline in front of the bulkhead wall underneath the deck is planned to be sloped on a combination of 2-horizontal to 1-vertical and 3-horizontal to 1-vertical and will include slope protection to protect against possible scour and erosion. The top of the Berth 6 deck will match the configuration of the existing wharf deck with the top of the deck at El. +13.67 feet at the berthing edge and at El. +14.37 feet at the landside edge. The conceptual design calls for the bulkhead wall to be tied back to an A-frame anchor wall.

In addition to the wharf expansion, proposed landside elements for the project also include storm water collection and outfall structures. Also, we understand that the Port of Port Arthur plans to construct approximately 1,200 lineal-feet of new shoreline stabilization between the end of Berth 6 and State Highway 82 Bridge. Current plans for the shoreline stabilization include articulated block mats anchored to the existing shoreline. Backfilling activities are also planned for 2 low-lying areas along the Port facility.

LAN has been retained by the Port of Port Arthur as the Design Engineer for this project. LAN contracted Fugro to perform a geotechnical study at the project site to assist them in their design of the proposed wharf expansion. LAN has also retained Lloyd Engineering, Inc. (Lloyd) to assist the Project Team with the design of the proposed Berth 6.

1.2 Purposes and Scope

The purposes of our geotechnical study were to: 1) explore and evaluate subsurface soil conditions at the location of the proposed Berth 6, and 2) provide geotechnical recommendations to guide others in the design and construction of foundations for the proposed structures and shoreline erosion protection. We accomplished these purposes by performing the following tasks:

- Reviewing existing geotechnical reports provided to us by LAN and previous studies performed by Fugro in the area. LAN provided logs of soil borings performed by Gore Engineering, Inc. in 1993 and 1995 and STS Consultants, Ltd. In 1996. Geotechnical reports previous prepared by Fugro near the project site and reviewed for this study are provided below:
    - Fugro Report No. 0401-3539 dated November 13, 1996
    - Fugro Report No. 0401-3723 dated June 13, 1997
    - Fugro Report No. 0401-4351 dated June 30, 1999
- Drilling 5 geotechnical soil borings, 2 on land to a depth of 40 feet below existing grade, and 3 in the water from a barge to a depth of 120 feet below existing mudline to explore subsurface conditions and obtain samples for geotechnical laboratory testing.
- Hydraulically advancing 9 Cone Penetrometer Test (CPT) soundings on land, 3 to a depth of 40 feet below existing grade, and 6 to a depth of 120 feet below existing grade, to explore subsurface conditions.
- Performing field and laboratory tests on selected soil samples to evaluate the engineering properties of the subsurface soils.
- Analyzing the field and laboratory data to develop geotechnical engineering recommendations for foundations of the proposed structures.
- Preparing this engineering report summarizing our findings and recommendations.

Environmental assessments, compliance with state and federal regulatory requirements, and environmental analyses, were beyond the scope of our services. A geologic fault study was also beyond the scope of our services.

1.3 Applicability of Report

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

We have prepared this report exclusively for LAN to assist them in their design of the proposed piles, bulkhead wall, and shoreline erosion protection as described in this report. We have

conducted our services using the standard level of care and diligence normally practiced by recognized engineering firms now performing similar services under similar circumstances. We intend for this report, including all illustrations, to be used in its entirety. This report should be made available to prospective contractors for information only and not as a warranty of subsurface conditions. It should not be used, whether in whole or part, as a stand-alone construction specifications document for design or construction of the proposed wharf expansion.

2.0 FIELD INVESTIGATION

Our field activities related to geotechnical soil borings and CPT soundings are discussed in this section. We have included discussion relating to drilling and sampling methods, CPT soundings, depth-to-water observations, and borehole and sounding completion.

2.1 General

Our field exploration plan was based on information provided to us by LAN and our understanding of the proposed wharf expansion. Our field services consisted of land-based and barge-based exploration. We drilled a total of 5 geotechnical soil borings, 2 on land and 3 in the water from a barge, and advanced 9 CPT soundings on land in an attempt to characterize the subsurface conditions at the project site. The borings were designated Boring B-1 through B-5 and the CPT soundings were designated CPT-1 through CPT-9. The completion depths of our land-based geotechnical soil borings and CPT soundings ranged from 40 feet to 120 feet below the existing ground surface and the completion depths for our barge-based borings were 120 feet below the existing mudline. The approximate location of the borings and soundings are shown on the *Plan of Exploration* presented on Plate 2. The land borings and CPT locations were selected and located onsite by LAN. Survey coordinates of the actual boring locations were not provided by LAN at the time of this report.

2.2 Bathymetric Survey

John Chance Land Surveys, Inc. performed a Hazard Survey within the area of Borings B-1, B-2, and B-3 prior to our field exploration activities in an attempt to detect possible debris at the mudline surface. Areas of scattered debris along with 2 sonar contacts were observed but did not impact the boring locations. The water depths were determined by using an Odom Hydrotrac. Based on the survey, the water depths above the mudline ranged from 13 to 49 feet.

2.3 Drilling Methods

2.3.1 Land. The borings drilled on land for this geotechnical study were drilled using an all-terrain vehicle (ATV) rig using a combination of both dry-auger and wet rotary drilling techniques. Both of the land-based borings were initially drilled with dry-auger techniques to depths as indicated on the boring logs. Both borings were then completed using wet-rotary techniques. Detailed descriptions of the soils encountered in the borings drilled for this project are presented on the boring logs in Appendix A on Plates A-1 through A-5. A key identifying the terms and symbols used on the boring logs is presented on Plates A-6a and A-6b.

2.3.2 Water. The borings drilled over water were completed using truck-mounted equipment placed on a jack up barge using wet rotary drilling techniques. Our truck mounted rig was mobilized to the site and ramps were used to provide drive-on access to the barge. The barge was then mobilized to the boring locations. We set temporary casing from the barge deck into the existing mudline and drilled with drilling fluids to maintain borehole integrity. Casing generally extended 10 to 15 feet below the mudline

2.4 Sampling Methods

Soil samples for the land-based borings were generally taken at about 2-foot intervals to the completion depth of the borings or to a depth of 16 feet, and at about 5-foot intervals thereafter as indicated on the boring logs. Soil samples for the barge-based borings were generally taken at about 5-foot intervals to a depth of 100-feet below the existing mudline, and at about 10-foot intervals thereafter to the completion depth of the borings as indicated on the boring logs. Undisturbed samples of cohesive soils were generally obtained by hydraulically pushing a 3-inch diameter, thin-walled tube a distance of about 24 inches. Our field procedure for cohesive soil sampling was conducted in general accordance with ASTM D1587 (Standard Practice for Thin-Walled Tube Sampling of Soils). The samples were extruded in the field and visually classified by our field technician. We obtained field estimates of the undrained shear strength of the recovered samples using a hand penetrometer or Torvane. The field estimates were modified for stiff to hard, over-consolidated natural cohesive soils, as described on Plate A-6b. Portions of each recovered soil sample were placed into appropriate containers for transportation to our laboratory.

Granular soil samples were generally obtained using the Standard Penetration Test (SPT) as described on Plate A-6b. Our field procedure for granular soil sampling was conducted in general accordance with the ASTM D1586 (Standard Method for Penetration Test and Split-Barrel Sampling of Soils). Our field technician recorded the hammer blows for each sampling interval. The SPT N-values, as described on Plate A-6b, are recorded on the boring logs. Soil samples obtained from the split-barrel sampler were visually classified, packaged by the technician, and transported to our laboratory for testing.

2.5 Water Depth

The water depth was measured at each of the boring locations. Water was encountered at a depth of 5 feet below the existing grade in Boring B-4 at the time of drilling operations. The depth of water above mudline in offshore borings varied between 42 and 45 feet. We have assumed that the water surface elevation along the Sabine-Neches Ship Channel was approximately El. 0 feet at the time of our field exploration.

2.6 CPT Soundings

The CPT soundings were conducted using our track-mounted CPT rig. The in-situ soil data was obtained by hydraulically advancing a cylindrical steel rod, with an instrumented probe at the base, vertically into the subsurface soils at a constant rate.[1] The instrumented probe consists of a cone-shapes tip element and a cylindrical-shaped side friction sleeve element. Continuous measurements of penetration resistance at the cone tip and frictional resistance along the friction sleeve were recorded during the penetration. Pore water pressure measurements with depth were also recorded during penetration.

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[1] Jean-Louis Briaud and Jerome Miran, The Cone Penetrometer Test, Report to the Federal Highway Administration, Report No. FHWA-SA-91-043, February 1992.

The CPT field data was saved electronically for further data reduction in the office. The CPT sounding logs are presented in Appendix B. A generalized soil classification chart used for data reduction of the CPT results is also provided in Appendix B.

2.7 Borehole and Sounding Completion

At the completion of drilling and sampling activities, we backfilled the land borings and CPT holes with cement-bentonite grout. The grout was placed in each borehole and CPT hole from the bottom up using a tremie pipe. When grout returned to the surface, we removed the tremie pipe and topped off each borehole by pouring grout from the surface. The water borings drilled for this study were backfilled with cement-bentonite grout and soil cuttings from the bottom up using a tremie pipe.

3.0 LABORATORY TESTING

The laboratory testing program for this geotechnical study was directed primarily towards evaluating the classification properties of the subsurface soils, and the undrained and drained shear strength characteristics of the subsurface cohesive soils. Our laboratory tests were performed in general accordance with the appropriate ASTM standards as tabulated at the end of this section.

3.1 Classification Tests

The classification tests included tests for natural water content, liquid and plastic limits (collectively termed Atterberg limits), and material finer than the No. 200 sieve (percent fines). These tests aid in classifying the soils and are used to correlate the results of other tests performed on samples taken from different borings and/or different depths. The results of the classification tests are provided in the boring logs in Appendix A.

3.2 Shear Strength Tests

We measured the undrained shear strength from selected undisturbed samples of cohesive soils by performing unconfined compression, unconsolidated-undrained triaxial compression tests, or miniature vane shear tests. The natural water content and dry unit weights were determined as routine parts of the shear strength tests. The results of the laboratory shear strength tests, along with the field estimates of shear strength, are presented on the boring logs in Appendix A.

In addition, the undrained and drained shear strength parameters of selected undisturbed samples of cohesive soils were determined by conducting multi-stage consolidated-undrained (CU) triaxial compression tests with pore pressure measurements. The results of the CU triaxial compression tests, summarized in Table 3-1, are presented as Mohr's circles and as stress paths in Appendix C. Water content and dry unit weight were determined as part of the triaxial compression tests.

Table 3-1. Summary of Multi-stage CU Triaxial Compression Results

Boring No.	Depth (ft)	Material Description	Effective Cohesion, c' (psf)	Effective Friction Angle, ϕ' (°)	Cohesion, c (psf)	Friction Angle, ϕ (°)
B-4	20	Sandy Clay, brown and tan with sand pockets	90	32	0	35
B-4	40	Clay, olive gray with silt seams	390	24	610	14
B-5	8	Clay, dark gray, with organic material	140	28	170	23

3.3 Summary

A summary of the types and number of laboratory tests as well as the test method standard performed for this study is provided in Table 3-2.

Table 3-2. Laboratory Test Summary

Laboratory Test	Quantity	Testing Standard
Water Content	42	ASTM D2216
Atterberg Limits	10	ASTM D4318
Percent Passing the No. 200 Sieve	32	ASTM D1140
Miniature Vane Shear	7	ASTM D4648
Dry Unit Weight	30	ASTM D7263
Unconfined Compression	2	ASTM D2166
Unconsolidated-Undrained Triaxial Compression	28	ASTM D2850
Consolidated-Undrained Triaxial Compression	3	ASTM D4767

4.0 GENERAL SITE CONDITIONS

The interpreted site and subsurface conditions based on our field exploration and laboratory testing are discussed in this section. This section also includes a discussion of the depth-to-water in the land based field exploration and the depth of water above mudline at the project site.

4.1 Site Location and Description

The project site is located at the Port of Port Arthur Facility along Sabine-Neches Ship Channel in Port Arthur, Texas. *A Vicinity Map* showing the project site is presented on Plate 1. The proposed expansion will be on the west side of the existing wharf and will include the new Berth 6. The shoreline extension and support structures will extend west of the proposed Berth 6. The borings and CPTs were generally located near the existing shoreline. Based on our review of publically available historical aerial imagery, the site has experienced multiple phases of excavation and dredge placement activities. The subsurface conditions described in our report generally include the results of our review of existing data for the project site and the subsurface conditions encountered during our current geotechnical study.

4.2 Existing Geotechnical Information

As mentioned earlier, we have reviewed the existing geotechnical data for the site provided by LAN and from our internal library (Fugro Report Nos. 0401-3539, 0401-3723, 0401-4351 and 04.10120192) to assist us in evaluation of the information relating to the site geology, soil parameters and site history. LAN provided logs of geotechnical soil borings completed by Gore Engineering, Inc. in 1992 and 1995 and STS Consultants, Ltd. in 1996. Based on our review, the subsurface soil conditions described in previous studies were variable across the project sites, which is generally consistent with the results of our current study for the Berth 6.

4.3 Subsurface Conditions

The subsurface conditions within the depths explored at the project site consist primarily of natural cohesive soils with intermittent layers of cohesionless soils. Detailed descriptions of the soils encountered in the borings and CPT soundings performed for this project are presented on the boring logs in Appendix A and the CPT sounding logs in Appendix B. A generalized subsurface soil stratigraphy is presented on Plate 3. The observed subsurface stratigraphy at the project site is generalized in the following sections.

4.3.1 Stratum I. Stratum I soils are comprised of cohesive and granular fill soils extending from the existing ground surface to El. +8 feet to El. +6 feet. The Stratum I fill soils observed at the site consisted of sandy clay, clay, clayey sand and silty sand.

Measured moisture content in the Stratum I cohesive soils ranged from 7 to 24 percent. Results from liquid limit tests performed on soil samples obtained from Stratum I cohesive soils ranged from 27 to 40, plastic limits ranged from 12 to 14, and plasticity indices ranged from 13 to 28, indicative of medium to highly plastic cohesive soils. Field estimates and laboratory tests indicate

that the undrained shear strength of the Stratum I cohesive soils generally range from soft (250 psf) to stiff (2000 psf). The percentage of material passing the No. 200 sieve measured in tests performed on samples of the Stratum I granular soils was 32 to 42 percent. SPT blow counts indicate that the granular fill soils are generally medium dense, with blow counts ranging from 17 to 25 blows per foot. We observed sand pockets, roots, gravel and shell fragments in the Stratum I fill soils.

4.3.2 Stratum II. Stratum II soils are comprised of firm to very stiff natural cohesive soils and were observed beneath the Stratum I soils to a depth of approximately El. -170 feet, the maximum depth explored in this study. The Stratum II natural cohesive soils observed at the site consisted of clay, silty clay, and sandy clay. Measured moisture content in the Stratum II cohesive soils ranged from 18 to 54 percent. Results from liquid limit tests performed on soil samples obtained from Stratum II ranged from 28 to 86, plastic limits ranged from 12 to 21, and plasticity indices ranged from 10 to 65, indicative of moderately to very highly plastic cohesive soils. Field estimates and laboratory tests indicate that the undrained shear strength of the Stratum II soils generally range from firm (600 psf) to very stiff (greater than 3,500 psf). We observed sand pockets, silt pockets, sand seams, silt seams, shell fragments, organic materials, and ferrous and calcareous nodules in the Stratum II cohesive soils.

4.3.3 Stratum III. Stratum III soils consist of intermittent layers of granular soil encountered within the cohesive soils of Stratum II. The Stratum III granular soils observed at the site consisted of silt, sandy silt, silty sand, and sand. Isolated layers of Stratum III were observed from approximately 23 to 28 feet and 83 to 88 feet in Boring B-1, from 78 to 83 feet in Boring B-2, from 79 to 83 feet and 93 to 108 feet in Boring B-3, and from 16 to 44 feet and below 90 feet in the CPT soundings. The percentage of material passing the No. 200 sieve measured in tests performed on samples of the Stratum III granular soils varied from approximately 8 to 91 percent. SPT blow counts indicate that the natural granular soils are generally medium dense to dense, with blow counts ranging from 20 to 48 blows per foot. We observed clay seams in the Stratum III granular soils.

4.4 Water Depth

Free water was encountered at a depth of 5 feet in Boring B-4 during drilling operations. The CPT test results indicated an increase in pore water pressure below a depth of approximately 20 to 45 feet below the existing grade. The water depth to the mudline was measured at each boring location for the barge-based borings completed for this project. The water depth to the mudline ranged from approximately 42 to 45 feet at the time of our field investigation.

4.5 Seismic Site Class

The Port Arthur area is generally considered an inactive seismic zone. Based on our review of the encountered subsurface conditions in our borings and CPT soundings completed for this project and our review of the seismic site class definitions in Section 1613: *Earth Quake Loads* of the 2012 International Building Code and Chapter 20: *Site Classification Procedure for Seismic Design*

of ASCE 7-10: Minimum Design Loads for Buildings and Other Structures, this project location is classified as Site Class D.

4.6 Variations in Subsurface Conditions

Our interpretations of soil conditions, as described in this report, are based on data obtained from our visual observations, sample borings, laboratory tests, and our experience. Although we have allowed for minor variations in the subsurface conditions, our recommendations may ***not*** be appropriate for subsurface conditions other than those reported herein. It is possible that some undisclosed variations in soil conditions might occur outside the boring locations, especially along the mudline due to depositional channel action of the Sabine-Neches Channel as well as dredging and other channel maintenance activities. We also expect that variations are present between borings as result of discontinuous zones of granular soils that are the result of buried or abandoned drainage features and previous dredge fill placement activities. As such, we recommend careful observations during construction to verify our interpretations. Should variations from our interpretations be found, we recommend that we be notified and authorized to evaluate what, if any, revisions should be made to our recommendations.

5.0 DEEP FOUNDATION RECOMMENDATIONS

We understand that deep foundations will be used to support the proposed Berth 6 and associated mooring and breasting dolphins. We understand that LAN is considering the use of 20-inch, 24-inch, and 30-inch square pre-cast, pre-stressed (PCPS) concrete piles, and 54-inch and 60-inch diameter hollow spun PCPS concrete piles to support the proposed wharf expansion. The proposed relieving platform is currently planned to be supported on 16-inch square PCPS concrete piles. Current plans for the bulkhead wall include driven open-end 48-inch diameter steel pipe piles. This section provides our recommendations for static axial capacity, battered pile capacity, soil parameters for lateral capacity, group effects, estimated settlement, and static and dynamic load testing for driven piles.

5.1 Static Capacity

We analyzed the axial capacity of driven 16-, 20-, 24-, and 30-inch PCPS concrete piles, and 54- and 60-inch diameter hollow spun PCPS piles. The ultimate axial pile capacity curves were computed in general accordance with API 1986[2]. In this method, the ultimate compressive capacity of a driven pile is taken as the sum of the skin friction along the pile and the end bearing capacity at the pile tip. End bearing capacity should be neglected for driven piles when computing ultimate tensile capacities. The weight of the pile is neglected in our computations. We also neglected the soil strength to a depth of 2 pile diameters below mudline to account for soil variability and construction disturbances.

The ultimate axial pile capacity curves for the driven piles installed at the project site are presented on Plates 4a through 4e for piles installed at a mudline elevation of El. -5 feet, on Plates 5a through 5e for piles installed at a mudline elevation of El. -25 feet, on Plates 6a through 6e for piles installed at a mudline elevation of El. -50 feet, and on Plate 7a for 16-inch piles installed at El. +5 feet to support the proposed relieving platform.

We recommend a factor of safety of at least 2 be applied to the ultimate axial capacity of piles loaded in compression (transient and sustained) and transient tension. A factor of safety of 3 should be applied for sustained tension loads. We recommend these factors of safety be used in conjunction with the completion of pile load testing as described in Section 5.6 of this report. The Geotechnical Engineer-of-Record should be contracted to assist with developing pile driving criteria and a quality assurance and quality control testing program. We also recommend that a representative of the Geotechnical Engineer-of-Record be onsite during pile installation activities.

The tensile (uplift) capacity of each pile may be increased by adding the weight of the pile to the computed tensile capacity presented on Plates 4a through 4e for piles installed at a mudline elevation of El. -5 feet, on Plates 5a through 5e for piles installed at a mudline elevation of

[2] American Petroleum Institute. (1986). *Recommended Practice for Planning, Designing, and Construction of Fixed Offshore Platforms.* API Recommended Practice 2A (RP 2A), 16th ed., Washington, D.C.

El. -25 feet, on Plates 6a through 6e for piles installed at a mudline elevation of El. -50 feet, and on Plate 7a for piles installed to support the relieving platform. An effective unit weight of 90 pcf for concrete is typically used for the pile. A factor of safety of at least 1.2 should be applied to the pile weight.

5.2 Battered Piles

We understand that current plans are to use battered piles for the proposed concrete deadman that will support the bulkhead wall. The ultimate pile capacities presented in this report are for vertical piles. We recommend the ***ultimate*** pile capacities for battered piles be calculated as discussed on Plate 8.

5.3 Lateral Capacity

We understand that LAN will perform an analysis of the behavior of laterally loaded piles to be installed at the project site. We understand the analysis will be done using the computer program LPILE. As such, we have provided soil parameters to assist in this analysis. Our recommended parameters for LPILE analysis are presented in Table 5-2. The parameters are based on the encountered soil conditions and the results of our laboratory testing. We recommend that appropriate measures be taken during the LPILE analyses to account for potential changes in mudline elevation.

Table 5-1. Summary of LPILE Parameters

Soil Type	Elevation (feet)		Effective Unit Weight, γ'	Cohesion, c	Friction Angle, ϕ	Stiffness, k (psi/in)		ε_{50}
	Top	Bottom	(pcf)	(psf)	(deg)	Static	Cyclic	(in/in)
Sand (API) (LWA)[1]	12	-5	8	-	35	120	120	-
Stiff Clay w/o free water[2]	-5	-28	58	1,000	-	335	130	0.01
Sand (API)	-28	-43	53	-	25	25	25	-
Stiff Clay w/o free water[2]	-43	-60	58	800	-	265	105	0.014
Stiff Clay w/o free water	-60	-100	58	1,700	-	570	225	0.007
Stiff Clay w/o free water	-100	-110	58	700	-	235	90	0.016
Stiff Clay w/o free water	-110	-170	58	2,000	-	670	250	0.006

Note:

1. If light weight aggregate (LWA) is not used, then we should be notified about the type of backfill to be used so that we can provide appropriate soil parameters.
2. Stiff clay with free water should be used if the pile is driven over water and cohesive soils are anticipated at the mudline.

5.4 Group Effects

The overall allowable load-carrying capacity of a group of piles may, in some cases, be less than the sum of the individual allowable capacities. A reduction in the individual pile capacity, to allow for group effects, is usually not necessary for piles having a center-to-center spacing of 3 to 5 or more pile diameters or widths. The reduction in individual capacity is dependent on several factors including: the configuration of the group, number of piles in the group, pile size, the depth of installation, and the pile spacing. We recommend piles for this project be spaced at least 3 diameters or widths (center-to-center) to reduce substantial group effects. If piles are spaced closer or if pile groups larger than 5-by-5 are anticipated, we would be pleased to review the design and comment on group effects. Piles should also be similarly spaced from existing foundations.

5.5 Settlement Considerations

A detailed settlement analysis was beyond the scope of this study. Based on the anticipated pile spacing and the load conditions described above, we expect settlements to be due almost exclusively to the elastic settlement of the piles themselves. Settlement of the proposed pile-supported structures is expected to be about ¼ to ½ inch, depending on the pile lengths, pile spacing, and actual loading on individual piles. It should be noted that groups of piles will likely settle more than individual piles subjected to the same load per pile. This relationship is dependent on actual soil conditions, pile dimensions, and group configuration. The increase in settlement between individual piles and groups is generally negligible for groups of piles that are less than about 5-by-5. The settlement of pile groups is dependent on several variables including: dimensions of the pile group, pile length, sustained structural loading, and compressibility characteristics of the foundation soils. We would be pleased to perform a detailed pile group settlement analysis on a case-by-case basis under separate cover.

5.6 Pile Load Testing

We recommend that axial load tests be performed in general accordance with ASTM procedures. Load testing could include static axial capacity testing or dynamic pile testing, depending on the types of piles selected for the final design. The advantage of performing a load test can be realized in using a reduced factor of safety applied to the ultimate axial pile capacity for certain loading conditions. The reduced factor of safety can result in shorter pile lengths, shorter pile installation times, and a potential cost savings. Load testing should be conducted in the presence of an experienced Geotechnical Engineer. If static testing is selected, each test pile should be loaded to produce enough movement at the top of the pile to determine the ultimate capacity or at least 3 times the design load. If dynamic testing is selected, we recommend that dynamic testing include testing during restrike driving with a minimum wait of approximately 1 week. We recommend that the Geotechnical Engineer-of-Record be retained to review the results of load tests to: 1) evaluate the load/unload-displacement response of the piles, 2) evaluate the ultimate capacity of test piles, and 3) compare measured capacities and deflections with design criteria. At your request, we can provide members of our professional staff to monitor the recommended load tests.

5.7 Abandonment of Existing Foundations

We understand that the Port of Port Arthur is planning to abandon several existing piles as part of the proposed construction activities. If the piles are located within the footprint of the proposed dock, we recommend that the piles should be cut off 2 to 3 feet above water surface and indicated on the structural drawings. If the piles are not located within the footprint of the proposed dock, we recommend consideration be given to removing the piles with a vibratory hammer. An experienced Pile Driving Contractor should be consulted to help the Port of Port Arthur in assessing if the piles can be removed and aid in the development of removal procedures. The locations should be surveyed and documented and should be evaluated on a case-by-case basis.

6.0 BULKHEAD WALL RECOMMENDATIONS

The proposed Berth 6 expansion includes extension of the existing bulkhead wall and accompanying anchor wall. This section provides the results of our global stability analysis performed for the dredge slope underneath the deck and our recommendations for lateral earth pressures to be used in the design of the proposed bulkhead wall. Current plans are for the bulkhead wall to be a combi-wall system with 48-inch diameter "king-pile" pipe sections spaced on 102-inch centers infilled with steel sheet piling and tied-back to an A-frame anchor wall. The Project Team is considering terminating the steel sheet piling at a shallower depth than the proposed king-piles.

6.1 Introduction

Our scope of services included evaluation of the global stability of the proposed bulkhead wall expansion and the slope stability of the dredge slope in front of the bulkhead wall. We have provided geotechnical soil parameters to assist the Project Team in their design of the bulkhead wall. The ultimate design of the bulkhead wall and dredge slope should satisfy requirements for global and slope stability and be able to resist the anticipated lateral forces. The results of our global stability analysis are provided in the following sections. A summary of geotechnical parameters to assist the Project Team in their design of the bulkhead wall is also provided, along with our recommendations for the bulkhead wall and anchor system.

6.2 Loading Conditions

The selection of geotechnical parameters and our method of analyses are in general accordance with the guidelines outlined in the United States Army Corps of Engineers (USACE) EM 1110-2-2504[3] and the Naval Facilities Engineering Command (NAVFAC) DM 7.2[4], as well as our experience with similar structures and subsurface conditions. The proposed bulkhead wall should be evaluated for both short-term and long-term conditions. Our experience has shown that long-term conditions are often the most critical. For satisfactory performance, the proposed dredge slope underneath the deck and shoreline slope should have an acceptable factor of safety during their entire projected time of service. Factors of safety for all potential loading conditions and modes of failure should be considered. The following paragraphs discuss each stability condition that should be analyzed.

6.2.1 Short-Term (Undrained). Short-term (undrained) loading conditions models the soil condition during and immediately following construction. EM 1110-2-2504 indicates that the end of construction usually represents the critical short-term (undrained) loading condition retaining walls. For this loading condition, any excess pore pressures developed during construction activities have not had the opportunity to dissipate. A factor of safety of at least 2 should be applied to the

[3] Design of Sheet Pile Walls, EM 1110-2-2504, 31 March 1994, United States Army Corps of Engineers.

[4] Foundations and Earth Structures, DM 7.2, 1 September 1986, Naval Facilities Engineering Command.

passive soil pressures for this loading condition according to EM 1110-2-2504. We recommend a minimum factor of safety of 1.3 in general accordance with USACE EM 1110-2-1913[5] for global stability. We performed our analysis with an assumed water surface elevation at El. +1 feet at the bulkhead wall sloping to the ground surface at El. +15 feet behind the face of the wall.

6.2.2 Long-Term (Drained). The long-term (drained) loading condition models the soils after excess pore water pressures have dissipated to post-construction equilibrium and post-construction consolidation of the cohesive soils has taken place. EM 1110-2-2504 indicates that a factor of safety of 1.5 should be applied to the passive soils pressures for long-term conditions. For global stability, we recommend a minimum factor of safety of 1.5 in general accordance with USACE EM 1110-2-1913. Our long-term analysis modeled the depth-of-water using the same assumptions as used in our short term analyses. It should be noted that relatively minor differences in the selected soil parameters for long-term analysis can have a significant impact on the results of our analyses. Discussion relating to soil parameters is presented later in this section.

6.3 Soil Parameters

Undrained soil parameters (undrained cohesion and undrained friction angle) and drained soil parameters (drained cohesion and drained friction angle) were selected for each soil stratum based on the laboratory and field test data collected during our field exploration and laboratory testing, correlations with published papers, our experience with similar projects and subsurface conditions. The soil parameters we used in our evaluation of the proposed bulkhead wall are presented for short-term and long-term conditions in Table 6-1 and Table 6-2. A discussion of our methodology in selecting geotechnical parameters is presented in the following sections.

6.3.1 Undrained Soil Parameters. We selected our undrained soil parameters by reviewing the results of our field exploration and laboratory testing. Granular soil conditions were based on the results of SPT blow counts and laboratory grain size analyses. The SPT and laboratory grain size data were obtained from the boring logs for each location. For stability purposes, the undrained angle of internal friction, ϕ, was based primarily on relative density correlations with SPT blow counts as developed by Terzaghi and Peck[6] as well as Lambe and Whitman[7]. Unit weight values for granular soils were also based on published relative density correlations[4] with SPT blow counts recorded during our field exploration activities.

Undrained soil parameters were similarly developed using a combination of unconfined compressive strength measurements taken during field exploration activities with a hand penetrometer and shear strength measurements with a Torvane. We also used the results of

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5 Design and Construction of Levees, EM 1110-2-1902, 30 April 2000, United States Army Corps of Engineers, Washington, D.C.

6 Terzaghi, K. and Peck, R.B., Soil Mechanism in Engineering Practice, 2nd Ed., John Wiley and Sons, New York, 1967, pp. 22-42, 341.

7 Lambe, T.W. and Whitman, R.V., Soil Mechanics, John Wiley and Sons, New York, 1969, pp. 148-149, 352-373, 423-486.

laboratory unconfined compressive strength, unconsolidated-undrained triaxial compression, and miniature vane tests. Unit weight values were determined based on measured unit weight values in the laboratory. To develop our design soil parameters, we reviewed the boring logs for soil strength, unit weight, and plasticity information at each pertinent boring location relative to the slope. The undrained shear strength parameters for the generalized soil conditions encountered are presented in Table 6-1.

Table 6-1. Undrained Soil Parameters - Bulkhead Wall – Short-Term Conditions

Soil Type	Elevation (feet)		Effective Unit Weight, γ'	Cohesion, c	Friction Angle, ϕ
	Top	Bottom	(pcf)	(psf)	(deg)
Sand (LWA*)	12	-5	8	-	35
Clay	-5	-28	58	1,000	-
Sand	-28	-43	53	-	25
Clay	-43	-60	58	800	-
Clay	-60	-100	58	1,700	-
Clay	-100	-110	58	700	-
Clay	-110	-170	58	2,000	-

* Light Weight Aggregate (LWA)

6.3.2 Drained Soil Parameters. Drained soil parameters were also selected based on our review of the results of field exploration and laboratory testing. Granular soil conditions were based on the results of SPT blow counts and laboratory grain size analyses similar to the undrained condition described above.

We developed our drained shear strength parameters of cohesive soils by evaluating the results of various shear strength tests performed on the recovered soils from the site. Particularly, we considered the results of consolidated-undrained shear strength tests presented in Appendix C. The undrained shear strength parameters for the generalized soil conditions encountered are presented in Table 6-2.

Table 6-2. Drained Soil Parameters – Bulkhead Wall – Long-Term Conditions

Soil Type	Elevation (feet)		Effective Unit Weight, γ'	Cohesion, c	Friction Angle, ϕ
	Top	Bottom	(pcf)	(psf)	(deg)
Sand (LWA*)	12	-5	8	-	35
Clay	-5	-28	58	100	22
Sand	-28	-43	53	-	25
Clay	-43	-60	58	90	18
Clay	-60	-100	58	200	22
Clay	-100	-110	58	90	18
Clay	-110	-170	58	200	22

* Light Weight Aggregate (LWA)

6.4 Slope and Global Stability

To evaluate the stability of the proposed sheet pile wall and the dredge slopes as presented herein, we performed analyses on both the slope and global stability considering short-term and long-term conditions. We performed our stability analysis using the computer program *Slide*[8]. *Slide* randomly generates trial failure surfaces through a designed slope and evaluates the factor of safety for each trial failure surface. The program allows a large number of potential shear surfaces to be investigated to determine the critical failure surface for each of the analyzed slope configurations. We used the Simplified Bishop method in *Slide* to evaluate the global stability of the sheet pile wall. This method uses two-dimensional limit equilibrium analysis to determine the factor of safety for the slope. The computed factor of safety is the ratio of the forces resisting movement to the forces driving movement.

Global stability analyses should consider, at a minimum, static forces including soil, water, and surcharge loads. The analyses should also address the effects of dynamic forces, *e.g*. wind, waves, and vessel traffic, on the global stability. The effects of “extreme” events such as tropical storm or hurricane events may also be considered. The final dock design should be such that global stability is provided.

Based on information provided by LAN, we understand that the proposed bulkhead wall has a top elevation at approximately El. +14.5 feet and a mudline at approximately El. -20 to -25 feet. The dredge slope in front of the wall slopes downward to an ultimate dredge elevation at approximately

[8] *Slide 6.008* – 2D limit equilibrium slope stability analysis. Roc Science Slide

El. -48 feet. The proposed dredge slope will include slope protection to protect against possible scour and erosion. We evaluated the proposed bulkhead both with and without a potential 25-foot wide pile-supported relieving platform that is being considered behind the bulkhead. Our analysis included a surcharge load behind the proposed bulkhead of both 250 psf and 1,000 psf.

Soil parameters were selected based on the soil conditions encountered in our soil borings as summarized in Table 6-1 and Table 6-2. The results of our analysis for the soil conditions encountered are summarized below in Table 6-3 and Table 6-4. Graphical representations of our stability analyses output are included in Appendix D. We recommend a minimum factor of safety of 1.3 for the global stability of the sheet pile wall for short-term loading condition and 1.5 for long term loading condition. We understand that the Project Team is considering terminating the steel sheet piling at a shallower depth than the king-pile portion of the proposed bulkhead wall. We recommend that the steel sheet pile section extend to a depth to provide a minimum factor of safety for global stability of 1.0 for short-term conditions and 1.1 for long-term conditions.

Based on our results, we recommend the design dredge slope be sloped with a combination of 2-horizontal to 1-vertical from the top of the slope to El. -28 feet and at 3-horizontal to 1-vertical from El. -28 feet to the bottom of the slope.

Table 6-3. Summary of Global Stability Analysis – Short-Term Conditions

Bulkhead Wall Length (feet)	Bulkhead Wall Tip Elevation (feet)	Surcharge (psf)	Relieving Platform Included	Computed Factor of Safety
65	-50	1,000	No	1.0
74	-59	1,000	No	1.0
76	-61	250	No	1.3
			Yes	1.3
		1,000	No	1.2
			Yes	1.2
100	-85	250	No	1.8
		1,000	No	1.5

Table 6-4. Summary of Global Stability Analysis – Long-Term Conditions

Bulkhead Wall Length (feet)	Bulkhead Wall Tip Elevation (feet)	Surcharge (psf)	Relieving Platform Included	Computed Factor of Safety
65	-50	1,000	No	0.9
74	-59	1,000	No	1.1
76	-61	1,000	No	1.1
100	-85	1,000	No	1.5

We understand that the final design of the proposed bulkhead wall has not been completed by the Project Team at the time of this report. We recommend performing global stability analyses once the final size and depth of the sheet pile wall and the location of the anchor system have been determined. The final design should be evaluated for both short-term and long-term conditions.

6.5 Bulkhead Wall Analysis

Our recommend soil parameters for the design of the bulkhead wall are provided in Table 6-1 and Table 6-2. Undrained soil parameters (undrained cohesion and undrained friction angle) and drained soil parameters (drained cohesion and drained friction angle) were selected for each soil stratum based on the laboratory and field test data collected during our field exploration and laboratory testing, correlations with published papers, our experience with similar projects and subsurface conditions. The soil parameters we recommend using to determine the earth pressures are presented for short-term and long-term loading conditions. We have also provided active and passive earth pressure coefficients.

6.5.1 Factors of Safety. We recommend performing the bulkhead wall analyses in general accordance with the guidelines set forth in USACE EM 1110-2-2503. We recommend using a factor of safety of 2 for passive pressures under undrained (short-term) loading conditions, and a factor of safety of 1.5 for passive pressures under drained (long-term) loading conditions when determining bulkhead wall penetrations. For determining maximum bending moments and anchor loads, we recommend a factor of safety of 1.0 be used for the drained loading condition. We expect the Structural Engineer will apply adequate factors of safety to the structural design of the wall. We also recommend that when determining loads imparted on the sheet pile wall, appropriate surcharges are used behind the wall.

We performed global stability analyses based on the drawings provided to us by the Project Team. We evaluated the combi-wall system based on the current proposed design using a 48-inch diameter king pile spaced at 102-inches on center infilled with steel sheet pile. The global stability analyses indicated that the king-piles have a minimum tip elevation of El. -85 feet to satisfy the required factors of safety for short-term and long-term loading conditions. Our analyses indicate that the infilled sheet pile section have a minimum tip elevation of El. -59 feet to satisfy the

minimum required factors of safety for the sheet pile wall section. Internal design requirements may control the design of the bulkhead wall. The internal design is being completed by other members of the Project Team. We recommend performing additional analyses if the loading conditions, proposed bulkhead wall design, or cross-section geometry are different than what has been analyzed herein.

6.5.2 Anchor System. The proposed sheet pile wall will require an anchorage system. We understand that the anchorage system will likely consist of a combination of steel tieback cables connected to a series of A-frames, *i.e.* opposing batter piles with a concrete pile cap. We recommend using structural sand, crushed stone or light weight aggregate as the backfill material. We also recommend using a geotextile fabric separator, such as Mirafi 180N or other approved equivalent, be placed between the granular fill and natural soils. We recommend that the passive resistance elements be located outside of the active wedge of the sheet pile wall, as well as the passive wedge of other deadmen or anchorage components. The active wedge is defined by a line extending up at an angle of 45 degrees from where the soil intersects the front face of the sheet pile wall. Additionally, we recommend that the anchorage system components be installed prior to the dredging activities in front of the sheet pile wall. The dredging activities should not be completed until the required anchorage is in place.

The consolidation of the in-situ soils behind the proposed bulkhead should be evaluated. Generally, higher design loads and higher site grade raise activities will cause greater settlements. We did not explore or evaluate the soils behind the proposed footprint of the proposed Berth 6 expansion. As such, our evaluation is based solely on the results of our geotechnical soil borings and CPT soundings completed in the area. If a detailed settlement analysis is required for the proposed construction and site grade raise, we recommend that consideration be given to performing additional field exploration and laboratory testing.

The primary goal of any of the potential construction activities should be to mitigate overall settlement of the near surface compressible cohesive soils. Generally, we observed firm to stiff cohesive soils to a depth of 6 to 10 feet below existing grade in our land-based borings and CPTs. If present, these soils are likely to experience settlements due to the site grade raise and design distributed load of 1,000 psf. This settlement, if not properly accounted for, could result in excessive movements of the tieback anchors. This movement could lead to additional stresses on the anchors and anchor connections. At a minimum, we recommend that each tieback anchor be installed within an 8-inch diameter Schedule 80 PVC sleeve. If settlement in excess of 6 inches is expected, then the diameter of the PVC sleeve should be sized large enough to accommodate the anticipated settlements. Other potential options to mitigate the expected settlement are pre-loading, soil improvement, and soil removal and replacement behind the bulkhead wall.

7.0 BELOW GRADE STRUCTURES

We have provided our recommendations on allowable net bearing pressure and lateral earth pressures for walls for the proposed stormwater collection box to be constructed approximately 12 feet below grade. We understand the footprint of the collection box is approximately 12 feet wide and 60 feet long and generally located between CPT-7 and Boring B-4.

7.1 Allowable Net Bearing Pressure

From a geotechnical perspective, the performance of a foundation system for the stormwater collection box should provide an adequate factor of safety against shear failure of the foundation soils and reduce the potential for excessive settlements due to overstressing of the underlying foundation soils. The collection box foundation should be designed such that the applied bearing pressures do **not** exceed the allowable net bearing pressure of the underlying soils.

The allowable net bearing pressure for the proposed collection box is a function of, among other items, the bearing surface, the strength of the foundation soils, the location of the foundation, the shape of the foundation, and the recommended factor of safety. The collection box foundation should be proportioned so the maximum contact pressure under dead, live, and transient loads, does **not** exceed the allowable net bearing pressure of the foundations soils. Total loading conditions as described in this report refers to the combination of properly factored dead and live loads. Transient loading conditions refer to the combination of dead, live, and infrequent transient loads.

For total loading conditions we recommend the net bearing pressure be limited to 2,000 psf. For transient loading conditions we recommend the net bearing pressure be limited to 2,600 psf. Allowable net bearing pressure, as used in this report, is defined on Plate 9. To calculate values of W_e, W_s, and W_f from Plate 9, use effective unit weights of 60 pcf for soil and 90 pcf for concrete.

The presented allowable net bearing pressures are for the collection box supported on undisturbed, competent firm to stiff cohesive soils. If wet, weak, or disturbed soils are encountered at the foundation depth then we recommend that the Geotechnical Engineer be consulted. The allowable net bearing pressures presented in this report include a factor of safety ranging between 1.5 to 3 with respect to shear failure of the foundation soils. The recommended net bearing pressures do not limit settlement. We anticipate that the proposed stormwater collection box may experience total consolidation settlements on the order of approximately 1 to 2 inches. A detailed settlement analysis was beyond the scope of this study.

7.2 Lateral Earth Pressure on Below Grade Structures

Below grade walls should be designed to withstand permanent lateral earth pressures resulting from a combination of soil pressure, hydrostatic pressure, and surcharge loads. The distribution of lateral earth pressures on permanent non-yielding below grade walls is presented on Plate 10. We estimate that typical surcharge loads for general design purposes are on the order of 250 to 500 psf. Surcharge loads should be evaluated if additional heavy loads are going to be present in

the vicinity of the proposed below grade walls. We recommend that the allowable net passive pressure be taken as 800 psf for properly placed and compacted structural clay fill and as 600 psf for natural cohesive soils. We recommend that passive pressure be neglected to a depth of 5 feet unless area paving or other similar surface cover is provided.

It is possible a sloped excavation may be used to construct the proposed below grade structures. If this is the case, care should be taken during backfill operations not to over compact the backfill soils. Over compaction may induce significant stresses on walls. We recommend that compaction of the backfill soils not exceed 98 percent of the maximum dry unit weight of the placed soils. Hand held compaction equipment should be used to compact backfill within 4 to 8 feet of below grade walls. Settlement due to self-weight should be expected if compressive fill soils are used as backfill. Our experience indicates that properly placed and compacted fill soils may settle on the order of 1 to 2 percent of the fill height under self-weight.

If the wall is to be maintained in a relatively dry condition, backfill behind the walls should include at least 18 inches of free draining granular soils, or other engineered drainage system, along the wall extending from the base of the wall up to within about 3 feet of the surface. The granular soils or drainage system should collect the water and remove it from behind the walls with a discharge location or sumps and pumps. A 3-foot thick clay cover should be placed over the free draining granular soils or drainage system to reduce the potential for surface water entering the drainage soils or system behind the wall. Surface drainage should be provided away from all below grade structure walls.

8.0 SHORELINE PROTECTION RECOMMENDATIONS

The proposed Berth 6 expansion project also includes construction of approximately 1,200 lineal-feet of new shoreline stabilization between the end of the new dock and the State Highway 82 Bridge using articulated block mats anchored to the existing shoreline. This section provides the results of our global stability analyses and geotechnical recommendations for the proposed shoreline stabilization.

8.1 Soil Parameters

Based on drawings provided by LAN, we have evaluated the global stability of Cross Sections 3 and 4. We developed soil parameters for our analysis based on the results of our geotechnical field exploration activities and laboratory testing program. A summary of the drained and undrained soil parameters used in our slope stability analyses is presented in Table 7-1 and Table 7-2. The cross-sections used in our analyses are presented in Appendix E.

Table 8-1. Soil Parameters – Section 3

Generalized Soil Type	Elevation (feet)	Total Unit Weight (pcf)	Undrained (Short-Term)		Drained (Long-Term)	
			Cohesion, c (psf)	Friction Angle, ϕ (°)	Cohesion, c' (psf)	Friction Angle, ϕ' (°)
Fill/Stiff Clay 1	+6 to -28	120	1,000	-	100	22
Sand	-28 to -43	115	-	25	-	25
Firm Clay	-43 to -60	120	800	-	90	18
Stiff Clay 2	-60 to -100	120	1,700	-	200	22

Table 8-2. Soil Parameters – Section 4

Generalized Soil Type	Elevation (feet)	Total Unit Weight (pcf)	Undrained (Short-Term)		Drained (Long-Term)	
			Cohesion, c (psf)	Friction Angle, ϕ (°)	Cohesion, c' (psf)	Friction Angle, ϕ' (°)
Stiff Clay 1	+15 to -28	120	1,000	-	100	22
Sand	-28 to -43	115	-	25	-	25
Firm Clay	-43 to -60	120	800	-	90	18
Stiff Clay 2	-60 to -100	120	1,700	-	200	22

8.2 Methodology

To evaluate the global stability of the proposed slopes as presented herein, we performed slope stability analyses considering undrained (short-term) and drained (long-term) conditions. We performed our slope stability analyses using the computer program *Slide.* The *Slide* computer program randomly generates trial failure surfaces through a designed slope and evaluates the factor of safety for each trial failure surface. The program allows a large number and shape of potential shear surfaces to be investigated to determine the critical failure surface for each of the analyzed slope configurations.

We used the Spencer's method and Bishop's Simplified method in *Slide* to evaluate the global stability. Each method uses two-dimensional limit equilibrium analysis to determine the factor of safety for the slope. The computed factor of safety is the ratio of the shear strength of the soil to the shear stress required for equilibrium along the potential failure surface.

We performed the stability analyses for the proposed slopes based on cross-sections provided by LAN. For Section 3, the water elevation was assumed at El. +6 and for Section 4, the water elevation was assumed at El. +15. A surcharge load of 250 psf was applied on the top of the slope for both cases.

8.3 Stability Analysis Results and Recommendations

The results of our global slope stability analyses for undrained (short-term), and drained (long-term) loading conditions for the proposed shore line protection are summarized in Table 8-3. The graphical results of our stability analyses are presented in Appendix D. For short-term and long-term conditions, the USACE EM 1110-2-1913 requires a minimum factor of safety (F.O.S.) of 1.3 and 1.4, respectively. Our analyses were based on preliminary design drawings provided by LAN. We recommend that cross-section and survey data be provided during the final design to confirm the results of our global stability analyses.

It should be noted that our analyses assume slopes maintain their geometries as analyzed. This assumes no scour, erosion, or dispersion occurs. It is possible that slope geometries can change over time due to seepage, flood, and runoff events. We recommend a monitoring and maintenance program be established to repair and distress to the slopes. Considerations for scour and erosion protection are provided in Section 8.4 of this report.

Table 8-3. Global Stability Analyses Results

Cross Section	Short-Term Conditions (Undrained)		Long-Term Conditions (Drained)	
	Required Factor of Safety	Calculated Factor of Safety	Required Factor of Safety	Calculated Factor of Safety
Section 3	1.3	1.97	1.4	1.54
Section 4	1.3	1.31	1.4	1.4

8.4 Erosion Protection

Erosion protection should be provided in front of the proposed sheet pile wall and along the slope of new shoreline due to the presence of granular soils and soft to firm cohesive soils. The loss of soil in front of the wall could jeopardize the performance and integrity of the bulkhead. The following section provides our geotechnical recommendations and considerations to guide the design of erosion control features of the proposed sheet pile bulkhead. The discussions are based our understanding that the exposed soils will not be exposed to excessive prop wash or bow thrusters from marine vessels.

8.4.1 Selection of Erosion Protection Methods. It is our opinion that properly placed concrete riprap and/or articulated concrete erosion control mats are the most applicable alternatives for erosion control at this site.

8.4.2 Articulated Concrete Erosion Control Mats. The objective of the articulated concrete erosion control mats is to provide soil retention along the dredge slopes and armor the slopes from channel influences to help reduce erosion and retain the underlying bank soils. The mats should be articulated, cable-tied, cellular mattresses. The final design of the mat layout is usually a proprietary function of the mat supplier and contractor. However, we recommend that we be given an opportunity to review the design for compliance with our geotechnical considerations expressed herein. The mats should be property anchored using trench anchoring techniques or other anchoring methods to withstand the channel and prop wash energy. Anchorage will be a critical component of the long-term performance of the erosion control mats. The erosion control mat design should consider any locations where pipeline, outfalls, or other structures exist along the slopes.

The cables for the mats should be non-corrosive. We anticipate that the mattresses may be placed either from the high bank or within the channel using a crane and laydown area. The mattresses should be placed on the slopes on top of properly placed and anchored geotextile according to the manufactures specifications. Each mat section should be properly articulated to accommodate changes in slope topography.

8.4.3 Riprap. Concrete or stone riprap may also be used for erosion protection either in lieu of or in combination with the articulated concrete mats. Riprap may be used to protect slope soils and to supplement the erosion protection provided by the erosion control mats. The riprap should be selected and sized accordingly for the channel velocities and hydraulic conditions. A geotextile separator should be used between the riprap and the underlying bedding subgrade. The Geotechnical Engineer-of-Record should be given the opportunity to review the riprap specifications and design layout.

8.4.4 End Protection. The stabilization design should consider the influence of end forces from channel flow. Erosive forces including eddying, hydraulic shear, and uplift forces, can adversely affect the erosion control mats. Additionally, these conditions can also increase the potential for

bank erosion in the adjacent areas outside the stabilized sections. The overall erosion protection plan should address this concern.

8.4.5 Maintenance and Performance. A detailed periodic maintenance program should be developed as part of the erosion protection program. The program should include measures to periodically evaluate the performance of the erosion control features. Provisions should be given for reporting and documentation of observed distress. Repair instructions should also be included in the maintenance program. We also recommend that the program require inspection of the channel after events, *i.e.* tropical storms or vessel impact.

9.0 CONSTRUCTION CONSIDERATIONS

Recommendations regarding driven pile installation, site preparation, excavations, and fill placement activities are provided in the following sections.

9.1 Driven Pile Installation

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

9.1.1 Pile Drivability. We recommend that wave equation analyses be performed to select the proper hammer and cushioning to be used for pile driving activities at the site. Based on the subsurface soil conditions at the site, granular soils are anticipated between El. -78 feet and El. -108 feet. This will result in increased capacity and potential drivability concerns during installation. The hammer should provide sufficient energy to successfully install the proposed piles without causing driving stresses beyond the structural capacity of the piles. We are available to provide such geotechnical consultation once the final design has been developed. We also recommend that consideration be given to using fixed leads during pile driving operations.

9.1.2 Pile Driving Specification. Detailed pile driving specifications should be prepared by the design engineer in conjunction with the Geotechnical Engineer. The specifications should cover the project requirements for furnishing and installing the piles including: the scope of work, necessary submittals, piling details, equipment requirements, installation requirements and tolerances, capacity evaluation, and construction records. The specification should require the contractor to submit a complete package detailing the proposed piling equipment and installation procedures for approval prior to mobilization to the site. The complete package should also include the results of the wave equation analyses to evaluate the proposed pile driving hammer and cushion system for approval prior to mobilization to the site.

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

9.1.3 Pile Driving Equipment. Production piles should be driven using a hammer of adequate size in as nearly a continuous operation as feasible, without interruption, if possible. Pile driving

hammers may be diesel, steam, or air operated. The use of a drop hammer, with a light ram and a large stroke, is discouraged since this type of hammer can produce exceedingly high and damaging stresses during driving.

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

9.1.4 Driven Pile Installation. Production piles should be installed to a penetration criterion based on the pile capacity versus penetration curves presented on Plates 4 through 6. The penetration criteria should be developed in conjunction with the pile driving criteria to avoid pile damage. The selection of a particular length and particular criteria depends on the pile size, available length, and capacity requirements, in addition to the soil properties. We recommend restriking selected production piles periodically to determine if the driving resistance and pile capacity increase or decrease with time.

We recommend surveying the production piles to detect possible vertical and/or horizontal movement (commonly referred to as heave) that can result from soil displacement when driving adjacent piles. Piles which heave after driving adjacent piles should be redriven to at least their original penetration and final driving resistance. Pilot holes will reduce heave, if unacceptable pile and ground movements are experienced. We do not recommend the use of jetting to aid pile installation at this site without the consultation of the Geotechnical Engineer-of-Record.

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

9.1.6 Dynamic Pile Testing and CAPWAP Analysis. We recommend that design pile capacities be verified during installation by dynamic methods utilizing a Pile Driving Analyzer (PDA). The PDA can verify hammer performance, driving stresses, hammer-to-pile alignment, pile damage, and pile capacity during driving. Given the nature of the soils at the site, we expect that pile capacity will increase with time. Re-strike testing monitored with the PDA in combination with the computer program CAPWAP, (**CA**se **P**ile **W**ave **A**nalysis **P**rogram) can provide an estimate on the pile capacity increase and aid in predicting long-term pile capacity. We recommend that re-strike testing be performed after a minimum of at least 7 days from pile installation.

9.2 Shallow Open-Cut Foundation Excavations

Excavation safety systems should be in accordance with federal OSHA Standards, 29 CFR Part 1926 (Revised July 1992), Subpart P, Excavations. Details for open-cut slopes and excavation shoring based on soil type and groundwater conditions are provided in the latest amended OSHA federal regulations.

Excavations should be designed in accordance with all applicable local, state, and federal trenching regulations. Based on our interpretation of the regulations and the subsurface conditions indicated in the borings drilled at this site, we classify the natural clay as Type B soil and the surficial fill material and granular soils as Type C soil. Excavations deeper than about 4 feet in Type B soils should be either braced or sloped back no steeper than 1-horizontal to 1-vertical, while excavations in Type C soils should be braced or sloped back no steeper than 1.5-horizontal to 1-vertical. Flatter slopes or bracing should be used in either case if sloughing or raveling is observed.

Foundation soils exposed by the excavations should be protected from disturbance due to construction activities. We recommend that the foundations be placed the same day the excavation is completed. The foundation soils should be protected from moisture content fluctuations that could result in shrink-swell movements of the foundation soils. Good surface drainage away from all excavations should be established to prevent surface runoff from either flooding the excavations or ponding around the completed foundations. In addition, granular foundation soils should be relatively dry and undisturbed prior to placement of the foundations.

We recommend that the Geotechnical Engineer-of-Record or their qualified representative be present on site to observe foundation excavations and the suitability of the exposed soils for foundation support. We also recommend that the foundation concrete and concrete placement be monitored during construction.

9.3 Re-grading

Prior to placement of new fill for the slope improvements, we recommend that the contractor perform clearing, grubbing, and stripping activities to remove, debris, and other deleterious material.

Re-grading activities should extend along the entire footprint of the slope improvements. Care should be taken to mitigate the potential for erosion of the existing and nearby slopes after re-grading activities. Re-grading should also include repair of areas that may have experienced erosion, sloughing, or localized instability and the removal and replacement of poor or unsuitable surficial soils.

9.4 Selection and Placement of Fill Soils

We recommend using low plasticity cohesive soils (structural clay fill) for the placement of fill in the construction of new shoreline stabilization. The high plasticity on-site soils may be modified via lime-stabilization for use as structural (or select) clay fill. If high plasticity cohesive soils are used,

the Owner should be willing to accept the increase risk of desiccation cracking. Our experience has shown that highly plastic clays can be difficult to work, making proper compaction difficult to attain. In all cases, the suitability of potential fill material should be verified using location-specific classification tests.

Regardless of the material used for slope construction, a maintenance program will need to be established to repair any sloughing or erosion that occurs to the slope over the life of the shoreline.

9.4.1 Light Weight Aggregate. Expanded Shale, Clay, and Slate (ESCS) light weight aggregates are approximately half the weight of fills that are commonly used. We understand that light weight aggregate will be placed next to the sheet pile wall from El. +15 to El. -5 feet. The light weight aggregate will have a bulk density less than 70 pounds per cubic feet with a friction angle ranging from 35° to 45°. Light weight aggregate shall meet the requirements of ASTM C 330.

9.4.2 Structural Clay Fill. Structural clay fill should be used for slope construction. We recommend structural clay fill should have a liquid limit of less than 50 and a plasticity index between 15 and 30, with more than 60 percent of the material passing the No. 200 sieve. Structural clay fill should be free of deleterious matter and should have an effective clod diameter less than 3 inches. We do ***not*** recommend mixing sand with high plasticity clay to develop clay fill.

If fill material is selected and used that does not meet the plasticity and gradation requirements recommended herein, the risk of developing premature zones of failure, *e.g.* shrinkage cracks, is increased. The shrinkage cracks could lead to instability of the fill or sloughing when subjected to design water levels or wet weather.

Clay fill should be placed in 6- to 8-inch-thick loose lifts and uniformly compacted to 95 percent of the maximum dry density at a moisture content of 1 percent “dry” to 3 percent “wet” of optimum as determined by ASTM D698 (Standard Proctor). Clay fill should be compacted by a sheepsfoot or padfoot type roller, or by alternative methods that provide a “kneading” compaction equivalent to the sheepsfoot or padfoot roller. We recommend that confirmatory laboratory tests be performed on potential fill soils prior to their use and during fill placement.

If “wet” weather or extended “dry” periods deteriorate the exposed surface whereby a good bond cannot be formed between successive lifts, the earthwork Contractor should prepare the surface as necessary. This preparation may include removing or scarifying the top 2 to 3 inches of the underlying material before placing the next lift.

9.4.3 Lime-Stabilization. Lime-stabilization may be used to modify onsite cohesive materials to meet plasticity requirements. Laboratory tests should be conducted at the time of construction to determine the optimum lime content. The optimum lime content is the amount of lime necessary to achieve a pH of 12.4 (which represents lime fixation), while trying to achieve a plasticity (PI) of less than 20. Organics, chemical fertilizers, and some clay minerals can modify the amount of lime necessary for lime fixation. We recommend that a lime series be performed using the specific soil samples and proposed lime additive.

Lime-stabilization should be done in accordance with the Lime Association recommendations. Key items for lime-stabilizing the clay soils include placing the proper percentage of lime, thoroughly mixing the lime into the clay soils, bringing the stabilized soil to the proper moisture content, allowing the stabilized soil to cure for at least 48 hours, adjusting the moisture content from 1 percent “dry” to 3 percent “wet” of optimum moisture content, pulverizing the soils again until the lime is thoroughly blended, then placing the stabilized soil in accordance with the recommendations discussed herein. Lime-stabilized clay fill should be placed in 6- to 8-inch-thick loose lifts and uniformly compacted to 95 percent of the maximum dry density as determined by ASTM D698 (Standard Proctor).

The moisture-density relationship should be established based on a material sample obtained on-site after stabilization with lime. A combination of sheepsfoot or padfoot rollers and pneumatic rollers is recommended to compact the lime-stabilized clay fill.

9.4.4 Crushed Stone Fill. Crushed stone fill should be generally consistent ASTM C33 No. 67 stone, and be placed in loose lifts not more than 6- to 8-inches and uniformly compacted to 98 percent of the maximum dry density at a workable moisture content as determined by TxDOT Test Method TX-113-E. Crushed concrete may be used in lieu of crushed stone, provided the crushed concrete contains no rebar or steel debris.

10.0 GEOTECHNICAL DESIGN REVIEW AND CONSTRUCTION SURVEILLANCE

10.1 Role of Geotechnical Engineer of Record

As defined by national professional engineering organizations (*e.g.*, ASFE/Geoprofessional Business Organization and the American Society of Civil Engineers - Geo-Institute) the Geotechnical Engineer-of-Record (GER) is the licensed professional engineer who signs and seals a project's documents relating to geotechnical engineering aspects[9]. The GER is responsible for planning and executing the subsurface exploration for the project and providing the geotechnical recommendations needed to assist the Design Team.

Geotechnical engineering is practiced using the observational method. The GER extrapolates the site's subsurface conditions based on the results of the subsurface conditions observed during the subsurface exploration program. Consequently, the recommendations presented in the geotechnical engineering report are not entirely final. The GER can finalize them only by discerning the actual conditions through field observations during excavation and other construction activities. Experience has shown that such observations, by the GER, result in significant time and cost savings considering the extent of exploration that would otherwise be necessary to manage the uncertainty in subsurface conditions.

The recommendations presented in our geotechnical report represent our professional judgment based on our observations and experience, which is unique to this project. For us to confirm the adequacy of our recommendations, we believe it is imperative that we (Fugro) have an onsite presence for the geotechnical components of the ongoing construction. Onsite confirmation of our interpretations of the subsurface conditions, by our staff, is vital to successful construction and performance of the geotechnical elements of the site design. We believe that such construction phase services are an important component of loss prevention on construction projects.

10.2 Review of Design Drawings

We should be given the opportunity to review the drawings and specifications throughout the design process. The purpose of or review is to evaluate whether our recommendations are adhered to accordingly for critical geotechnical elements of the project. We can provide a proposal to address the scope of our design drawing review under separate cover.

10.3 Geotechnical Observations During Construction

As noted above, the geotechnical aspects of construction contain inherent uncertainties that can result in unanticipated soil and/or groundwater conditions being encountered during construction. Qualified geotechnical personnel observing onsite construction can monitor construction activities and may aid in recognizing unanticipated subsurface conditions and reconciling these conditions with design recommendations.

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[9] National Practice Guideline for the Geotechnical Engineer of Record, ASFE, 2009.

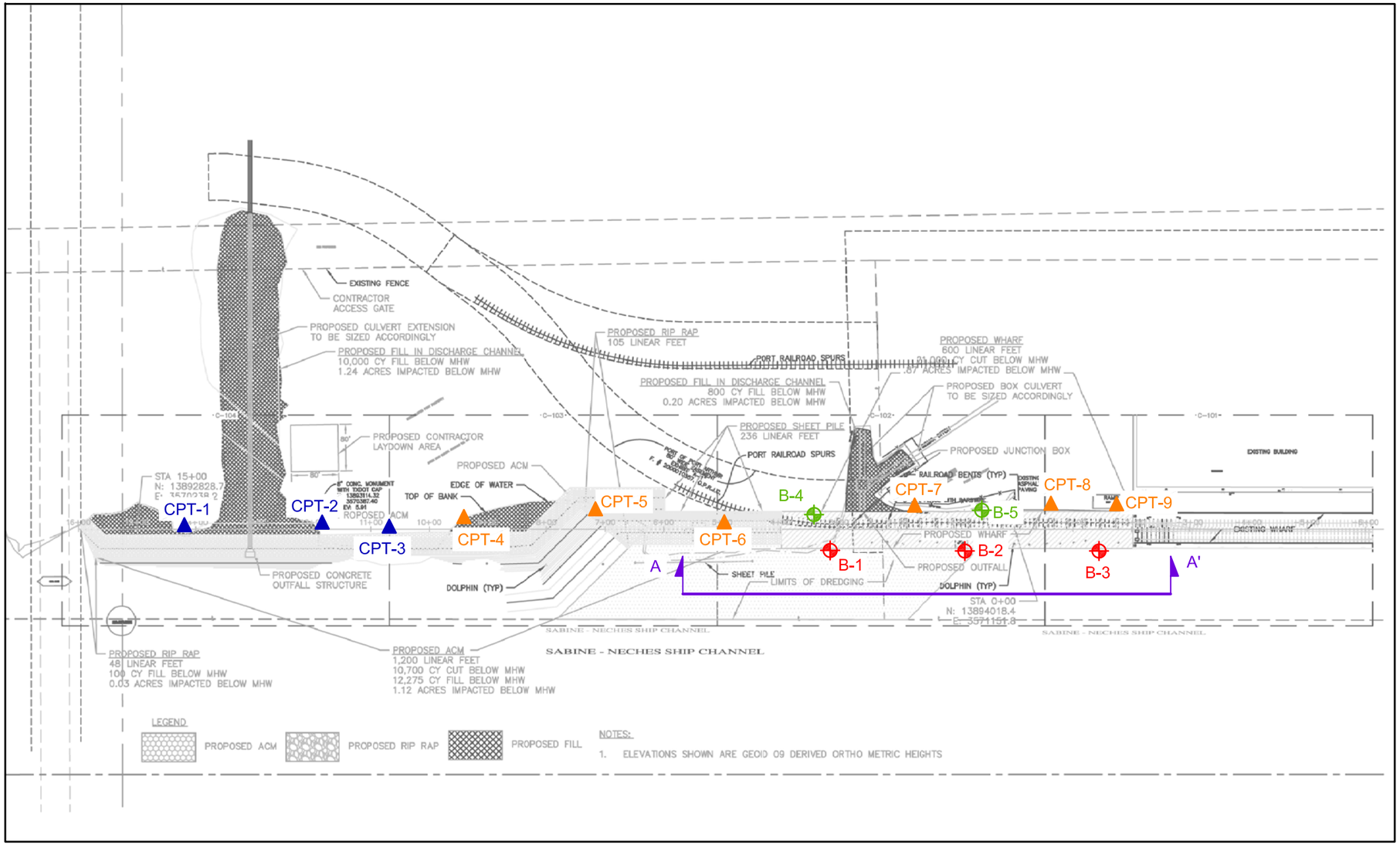
We should be retained during the site preparation, earthwork, and foundation excavation and construction phases to provide material testing and construction surveillance to:

- Observe compliance with the design concepts, specifications, and recommendations,
- Observe subsurface conditions during construction, and
- Perform quality control tests.

ILLUSTRATIONS

VICINITY MAP

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS



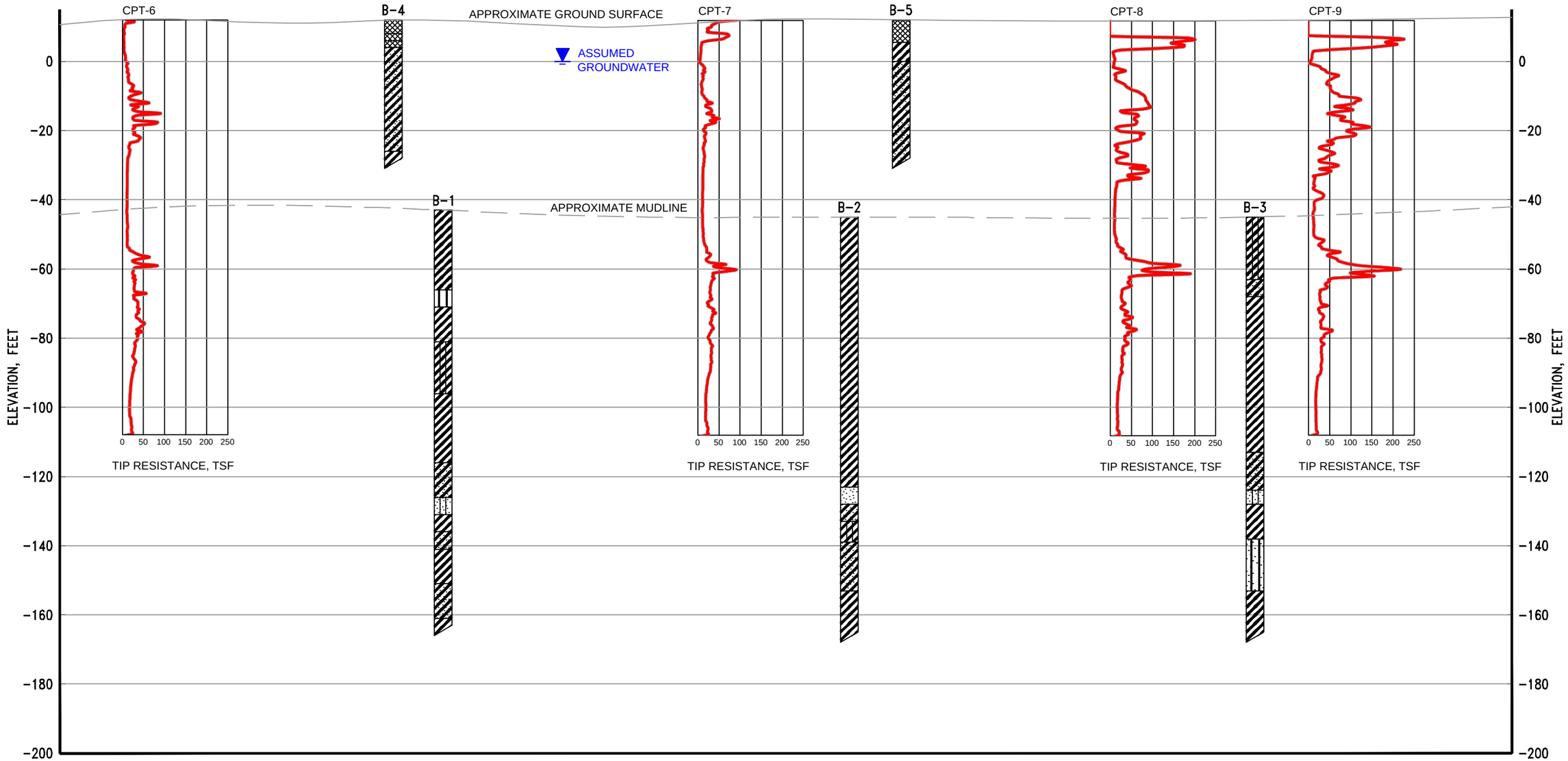
LEGEND

B-1	GEOTECHNICAL BORING (120' DEPTH)
B-4	GEOTECHNICAL BORING (40' DEPTH)
CPT-1	CONE PENETRATION TEST (40' DEPTH)
CPT-4	CONE PENETRATION TEST (120' DEPTH)
A A'	SUBSURFACE PROFILE

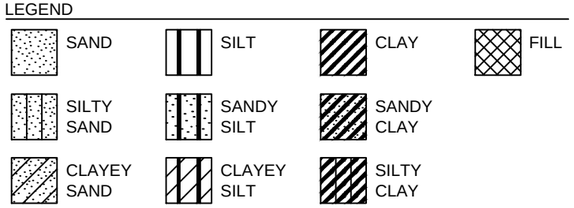
NOTES:
1. BORING AND CPT LOCATIONS ARE APPROXIMATE.
2. BASE DRAWING NO. (C-100) PROVIDED BY LOCKWOOD, ANDREWS & NEWMAN, INC.

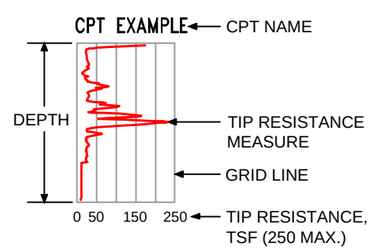
PLAN OF BORINGS
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

0 200 400 FEET



LEGEND





NOTES:

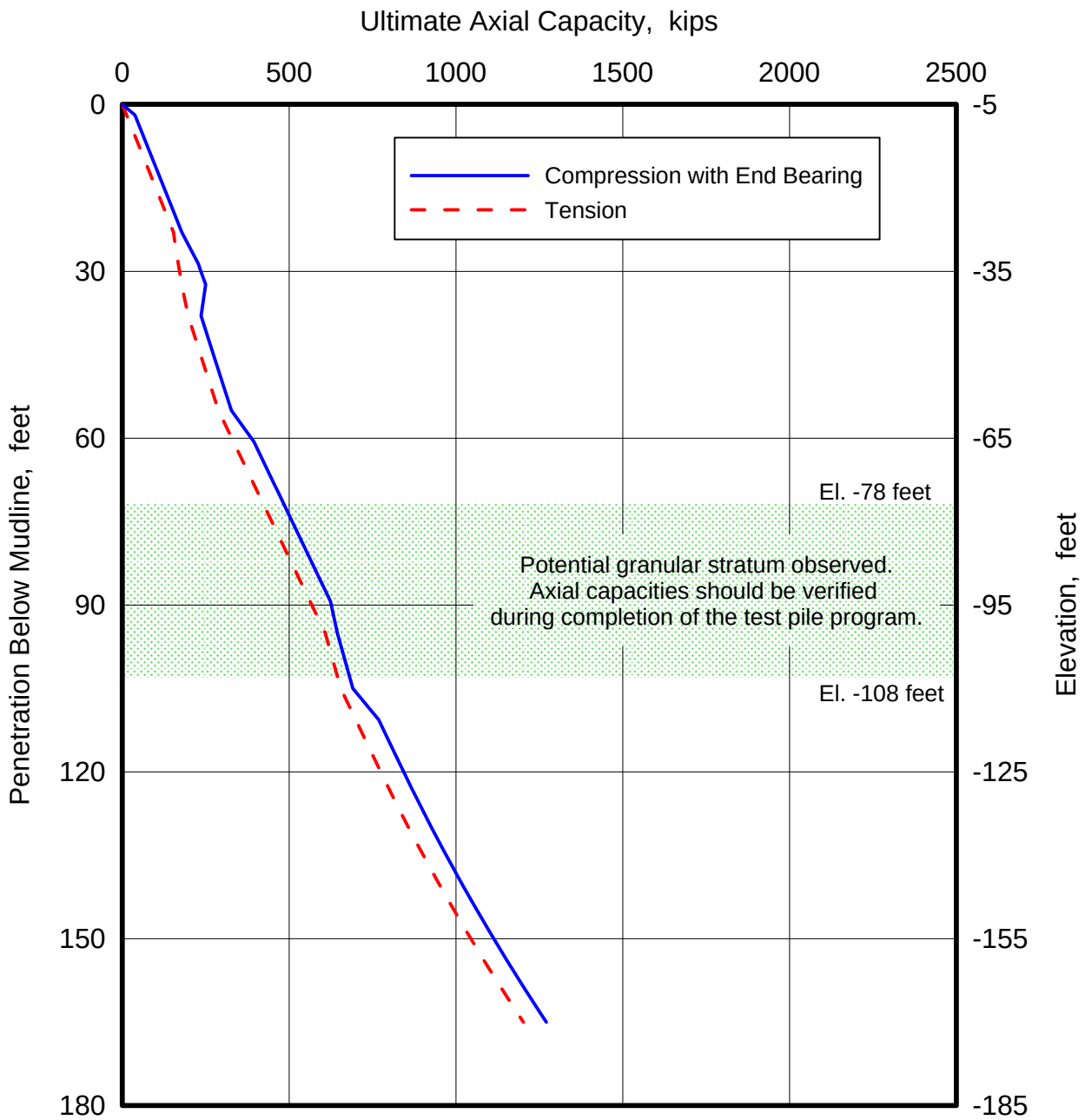
1. DATA CONCERNING SUBSURFACE CONDITIONS HAVE BEEN OBTAINED AT BORING AND SOUNDING LOCATIONS ONLY. ACTUAL CONDITIONS BETWEEN BORING AND CPT SOUNDING LOCATIONS MAY VARY FROM THE GENERALIZED SOIL PROFILE SHOWN HERE.
2. ELEVATIONS ARE APPROXIMATE.
3 BORINGS B-1, B-2, AND B-3 WERE DRILLED WITHIN THE SABINE-NECHES CHANNEL USING A BARGE.

ASSUMED GROUNDWATER AT THE TIME OF FIELD ACTIVITIES

GENERALIZED SUBSURFACE PROFILE (A-A')

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

VERTICAL SCALE
0 30 60 FEET

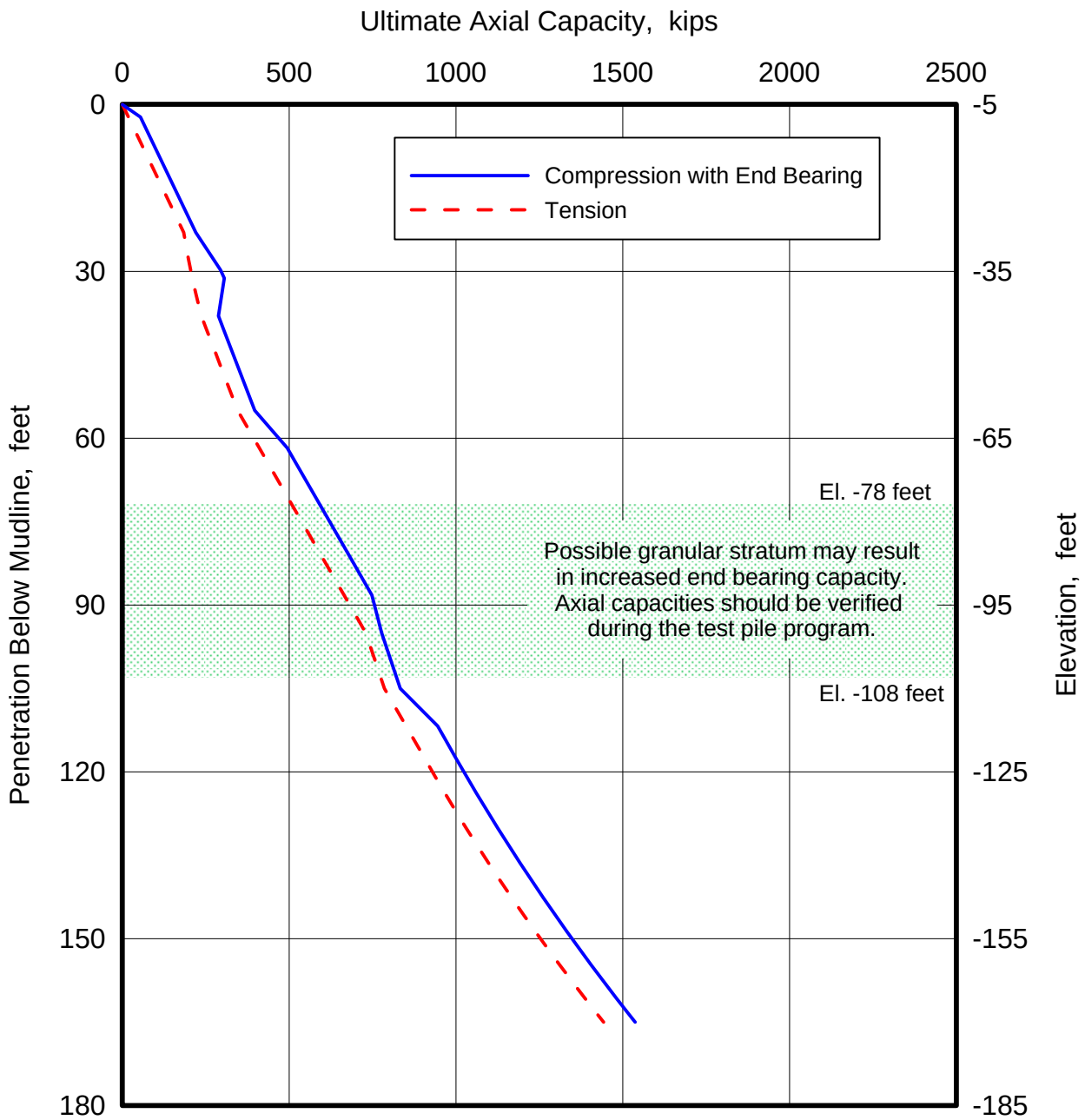


Notes:

1. These curves represent *ultimate* values of a driven square pre-cast, pre-stressed (PCPS) concrete pile for compression with end bearing and tension. Factors of safety for compressive and tensile loading are discussed in the report text.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -5 FEET
20-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS



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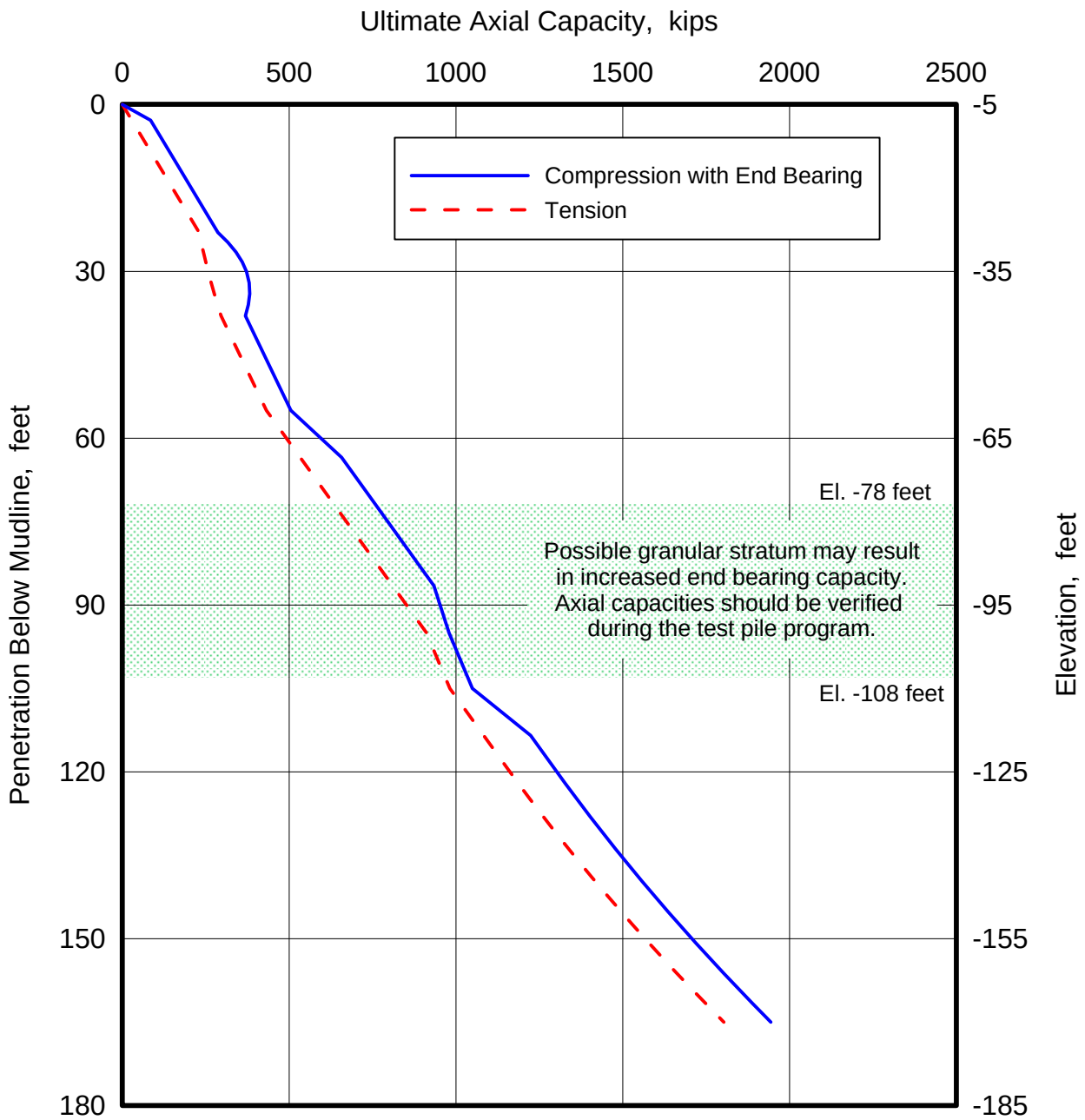
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -5 FEET
24-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 4b



Notes:

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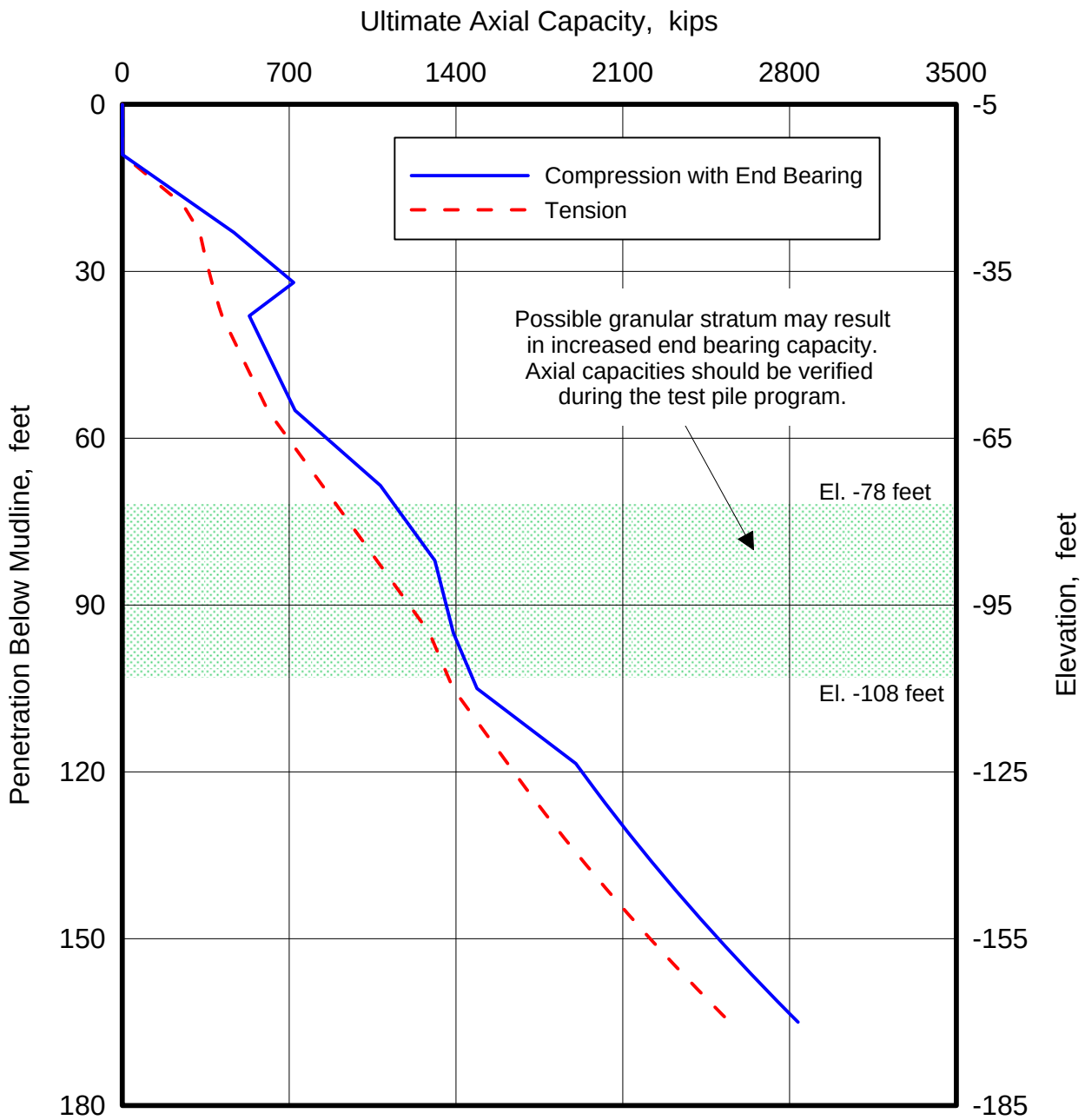
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -5 FEET
30-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 4c



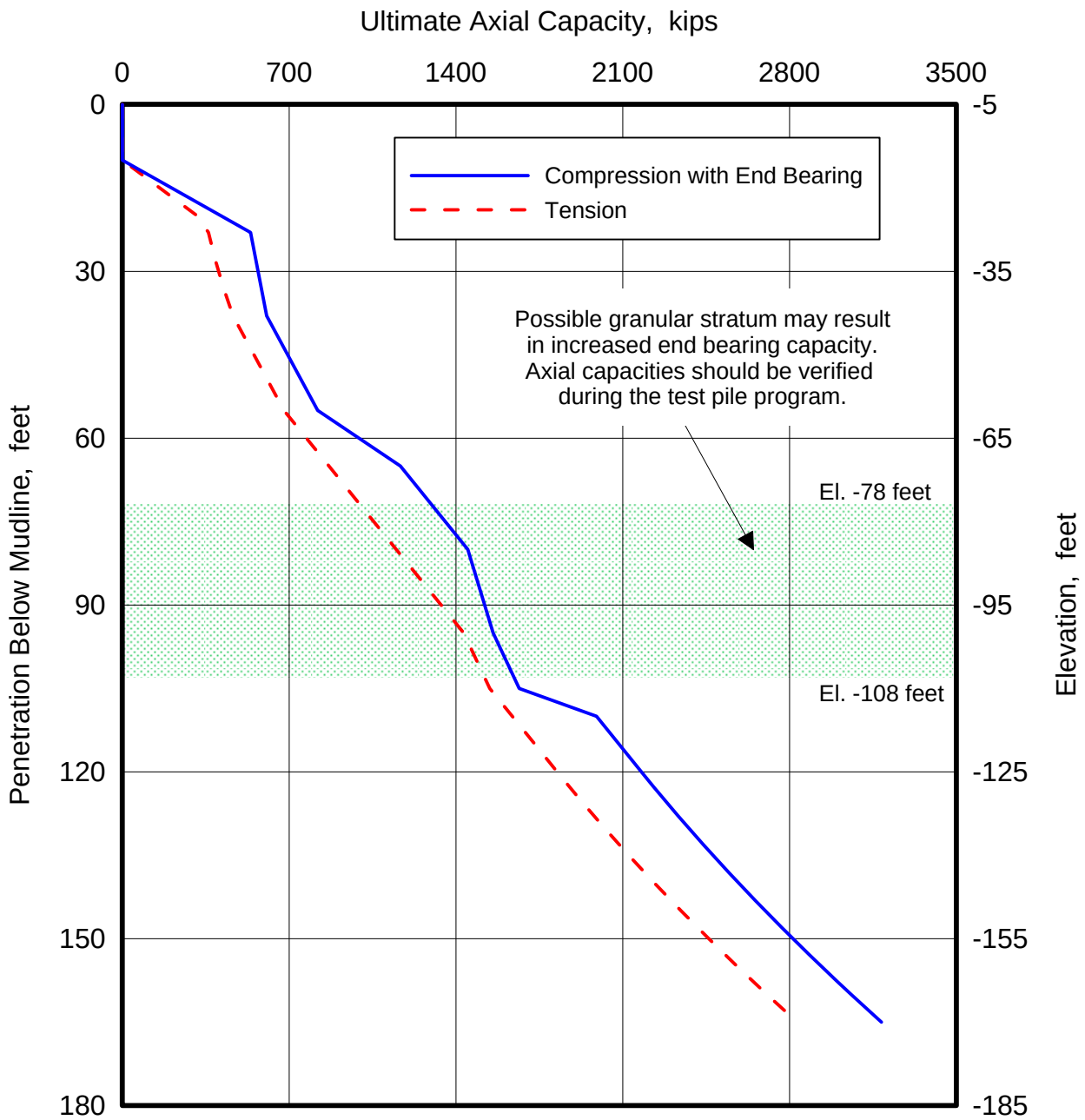
Notes:

1. These curves represent *ultimate* values of a driven hollow spun pre-cast, pre-stressed (PCPS) concrete pipe pile for compression with end bearing and tension. Factors of safety for compressive and tensile loading are discussed in the report text.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -5 FEET
54-INCH DIAMETER x 5-INCH WALL HOLLOW SPUN PCPS PILES

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 4d



Notes:

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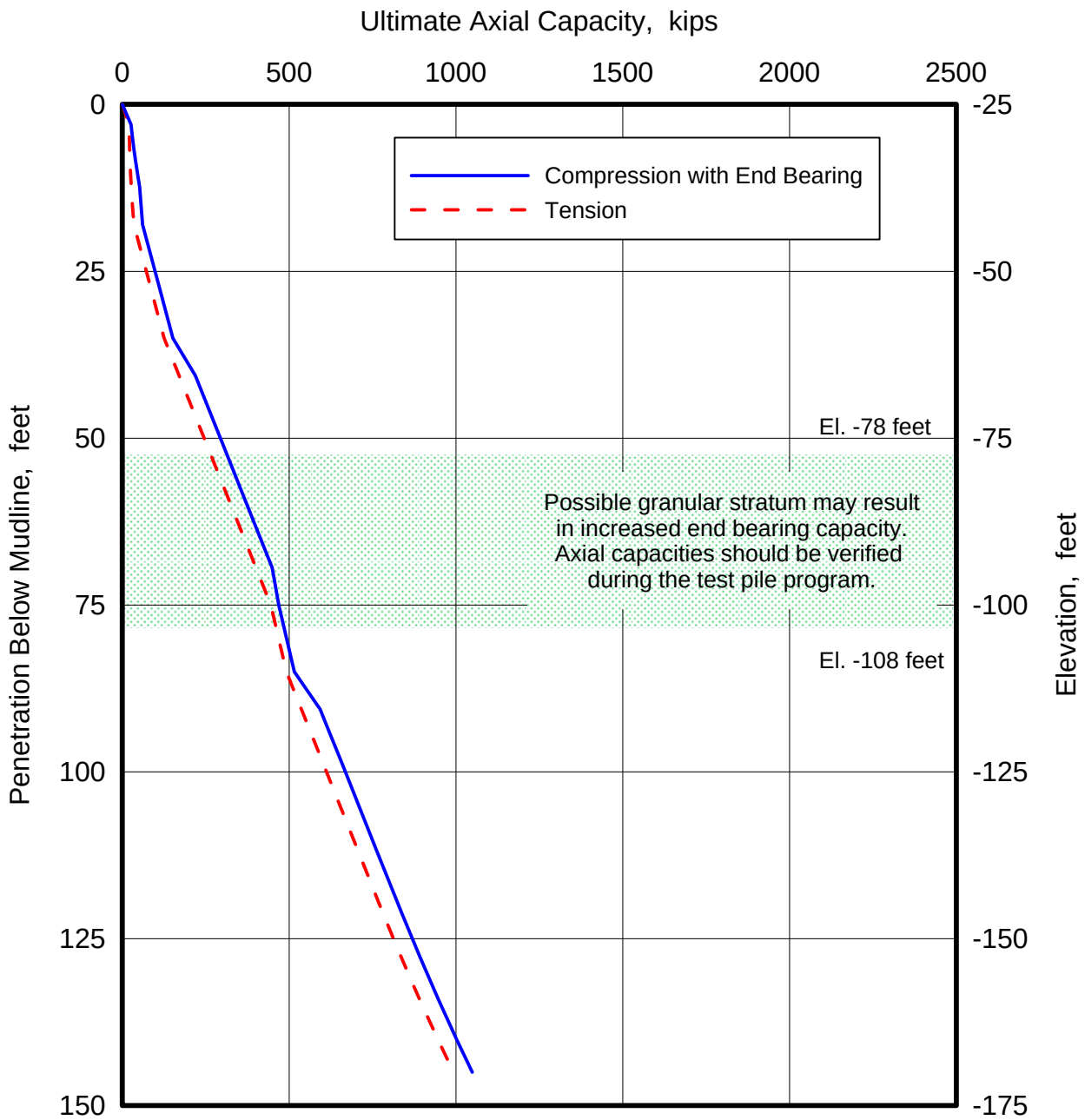
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -5 FEET
60-INCH DIAMETER x 6-INCH WALL HOLLOW SPUN PCPS PILES

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 4e



Notes:

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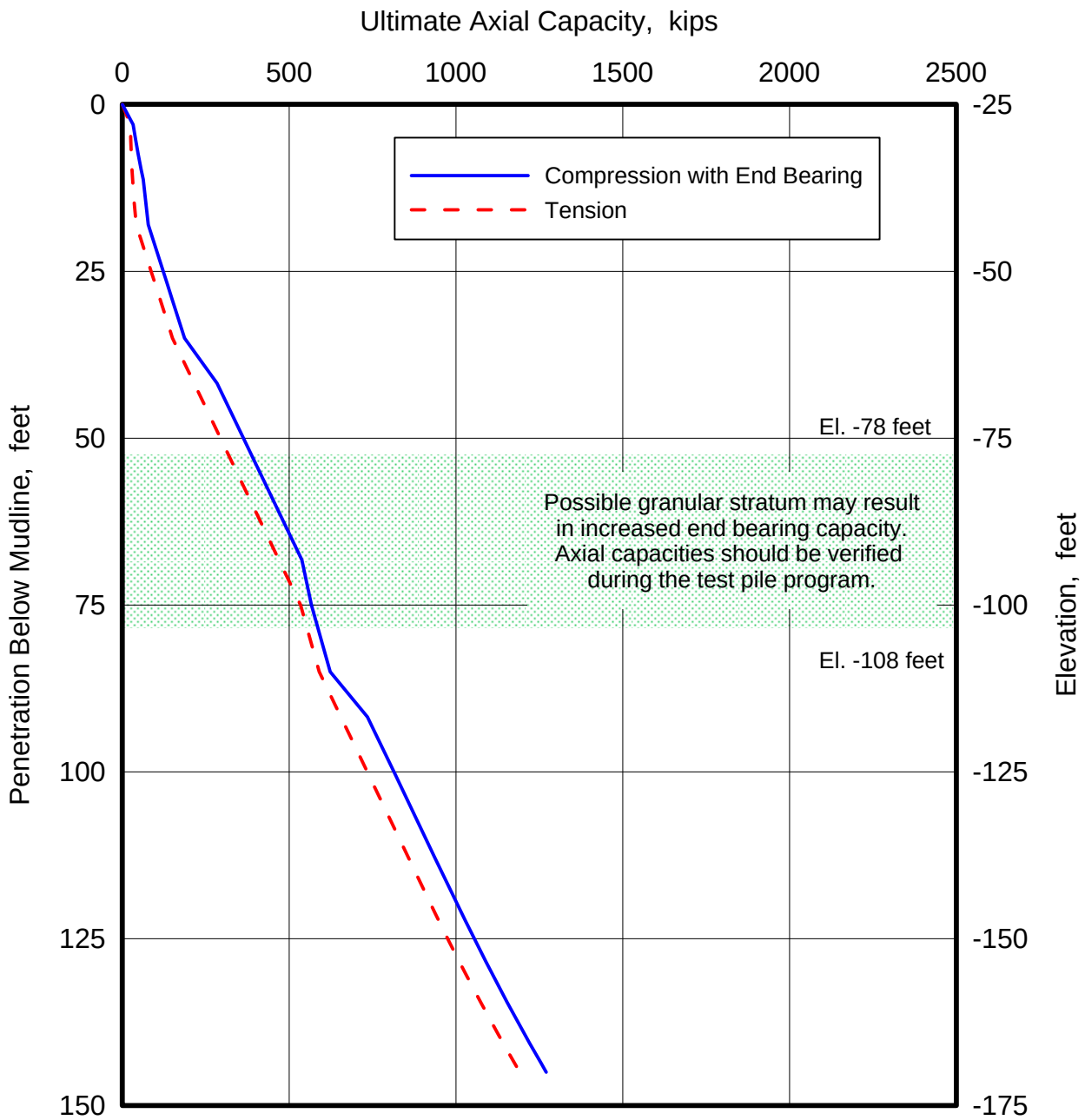
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -25 FEET
20-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 5a



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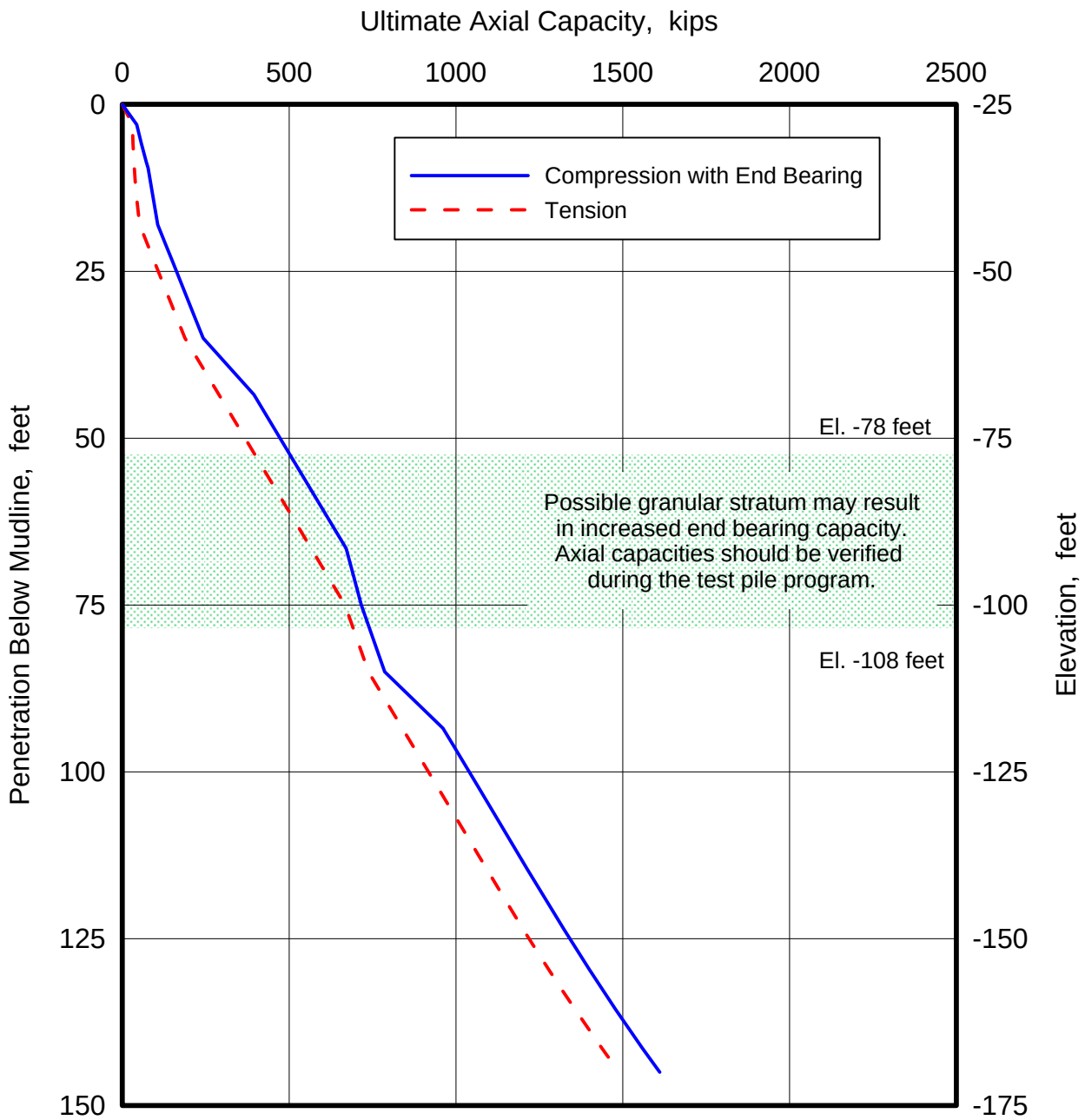
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -25 FEET
24-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 5b



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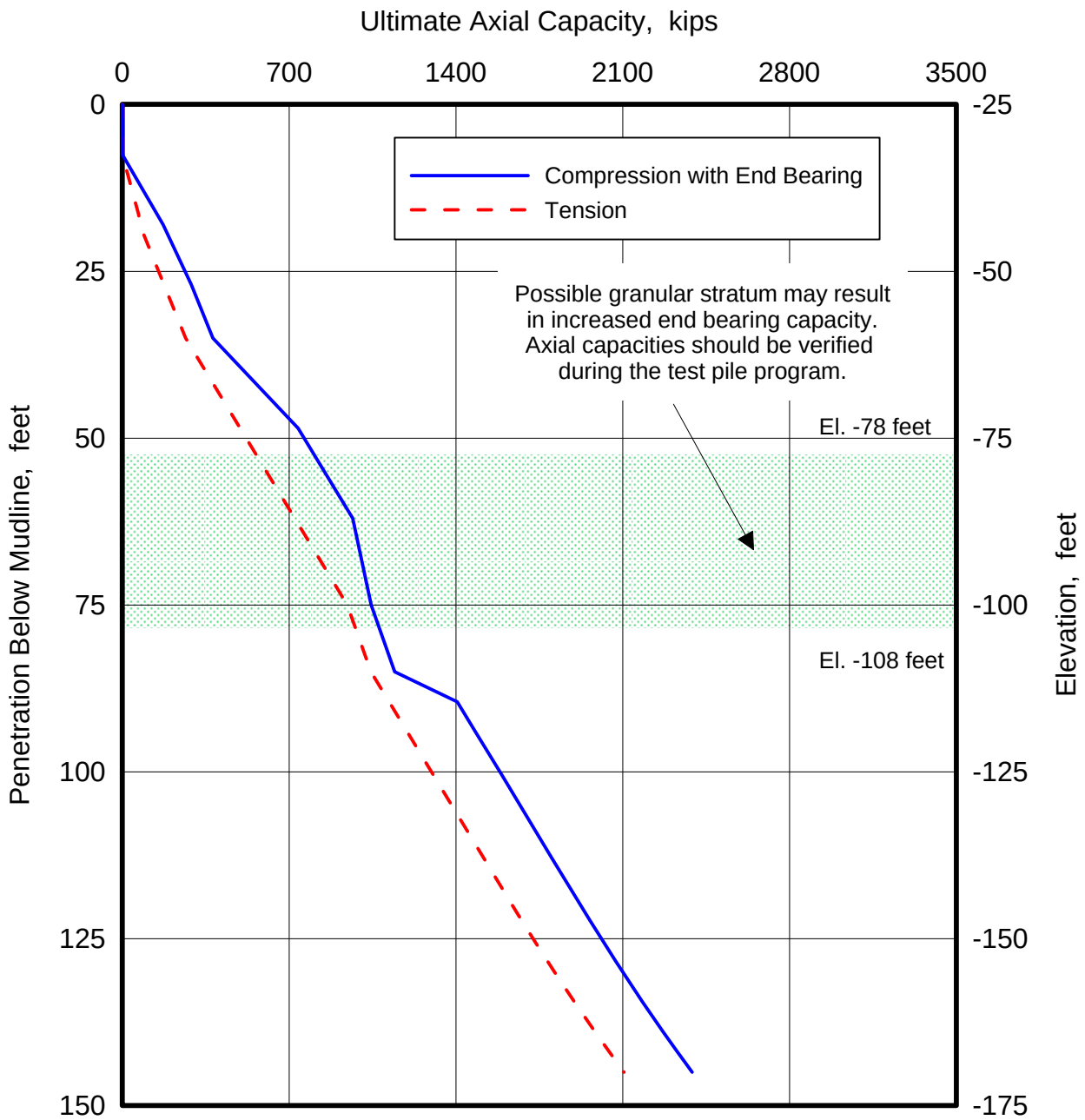
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -25 FEET
30-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 5c



Notes:

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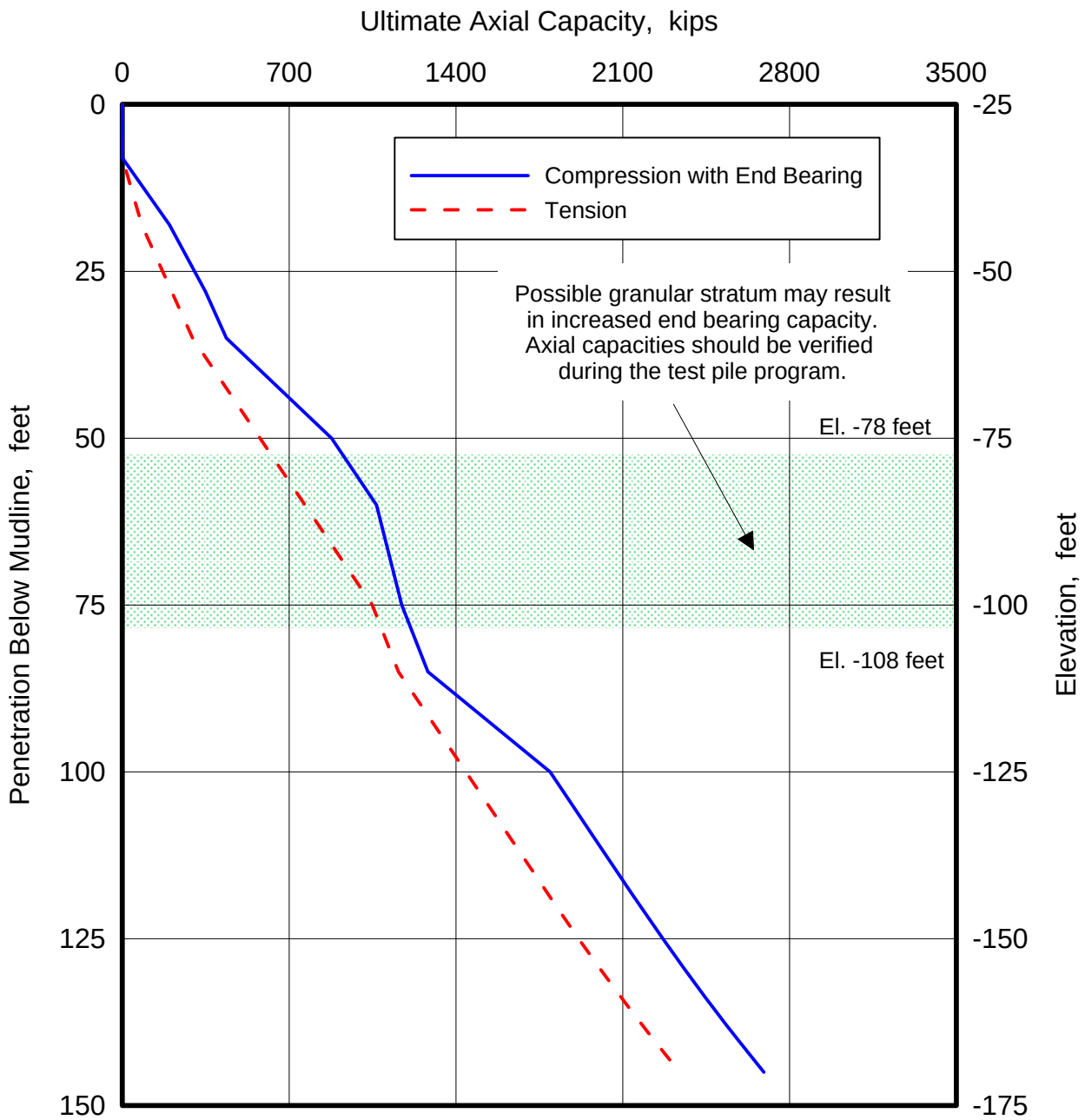
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -25 FEET
54-INCH DIAMETER x 5-INCH WALL HOLLOW SPUN PCPS PILES

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 5d



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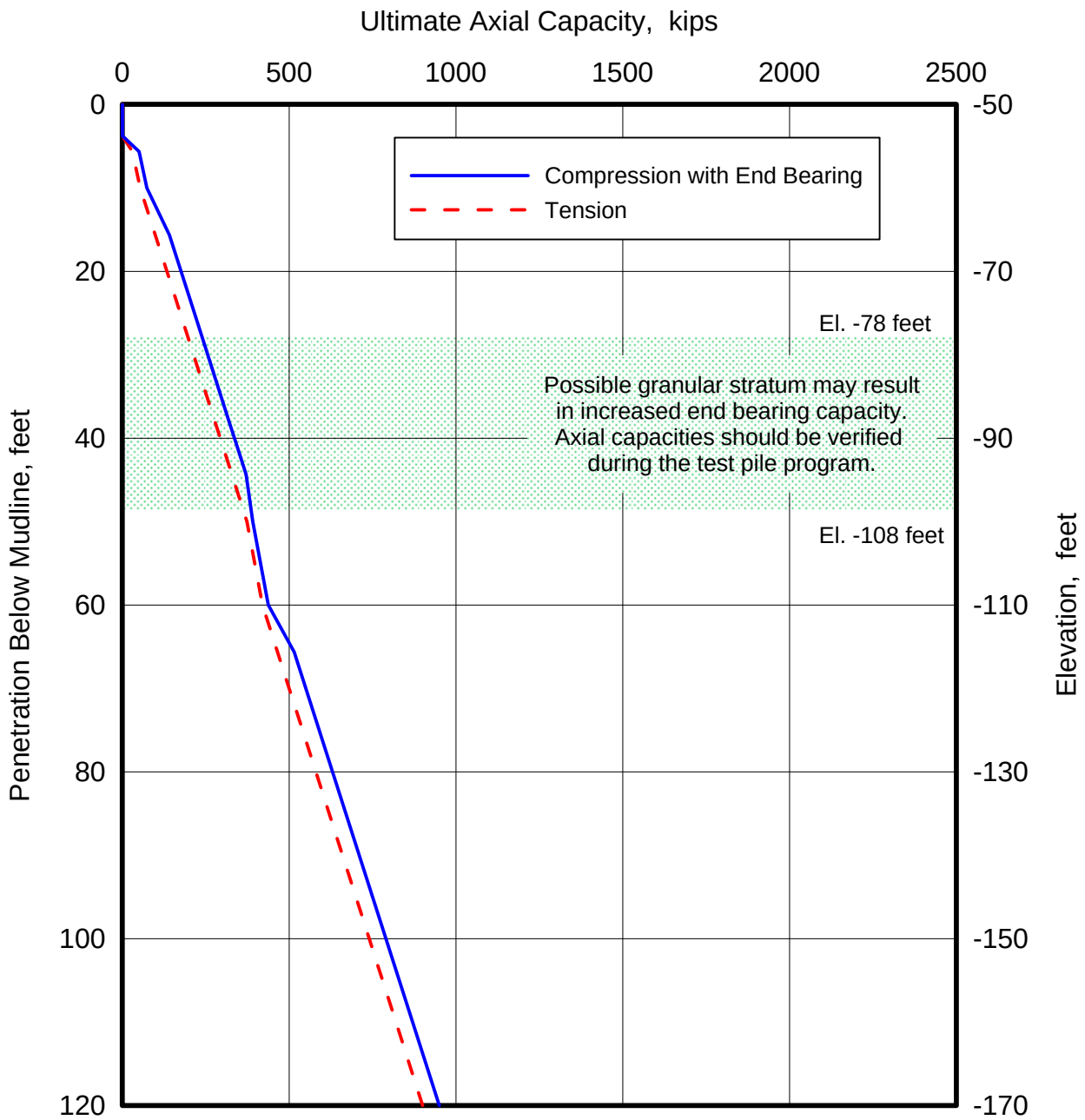
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -25 FEET
60-INCH DIAMETER x 6-INCH WALL HOLLOW SPUN PCPS PILES

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 5e



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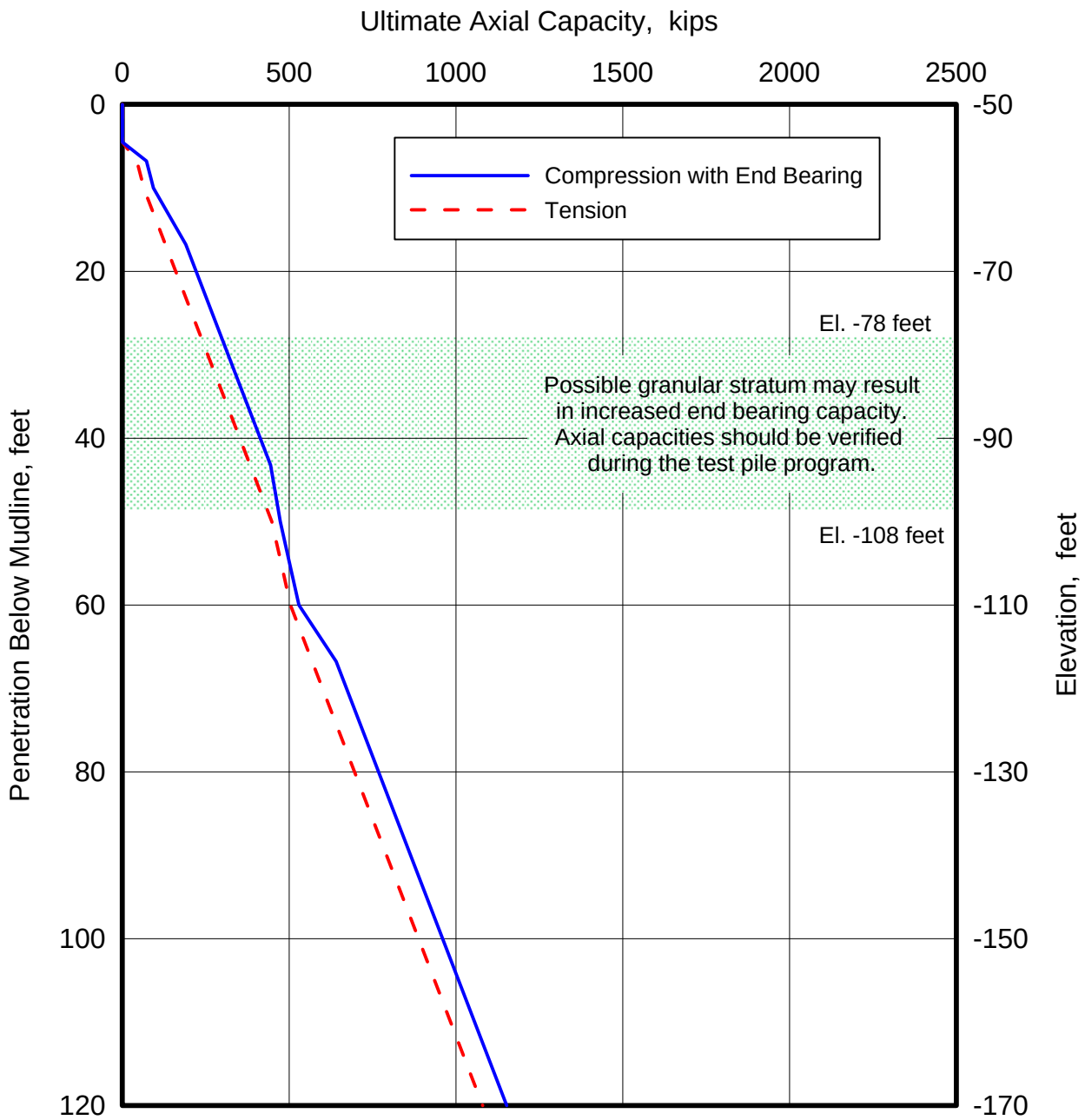
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -50 FEET
20-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 6a



Notes:

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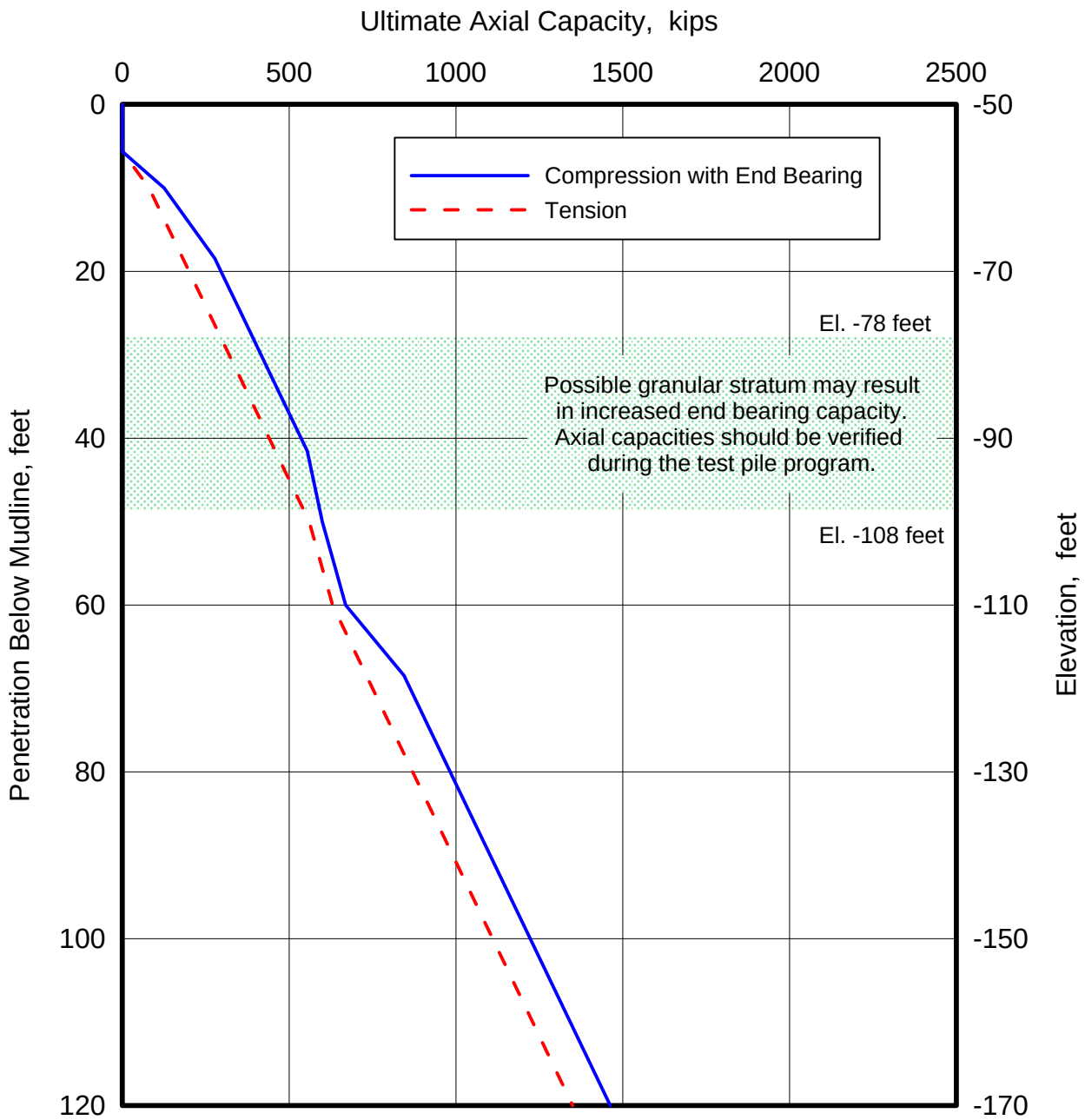
ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -50 FEET
24-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 6b



Notes:

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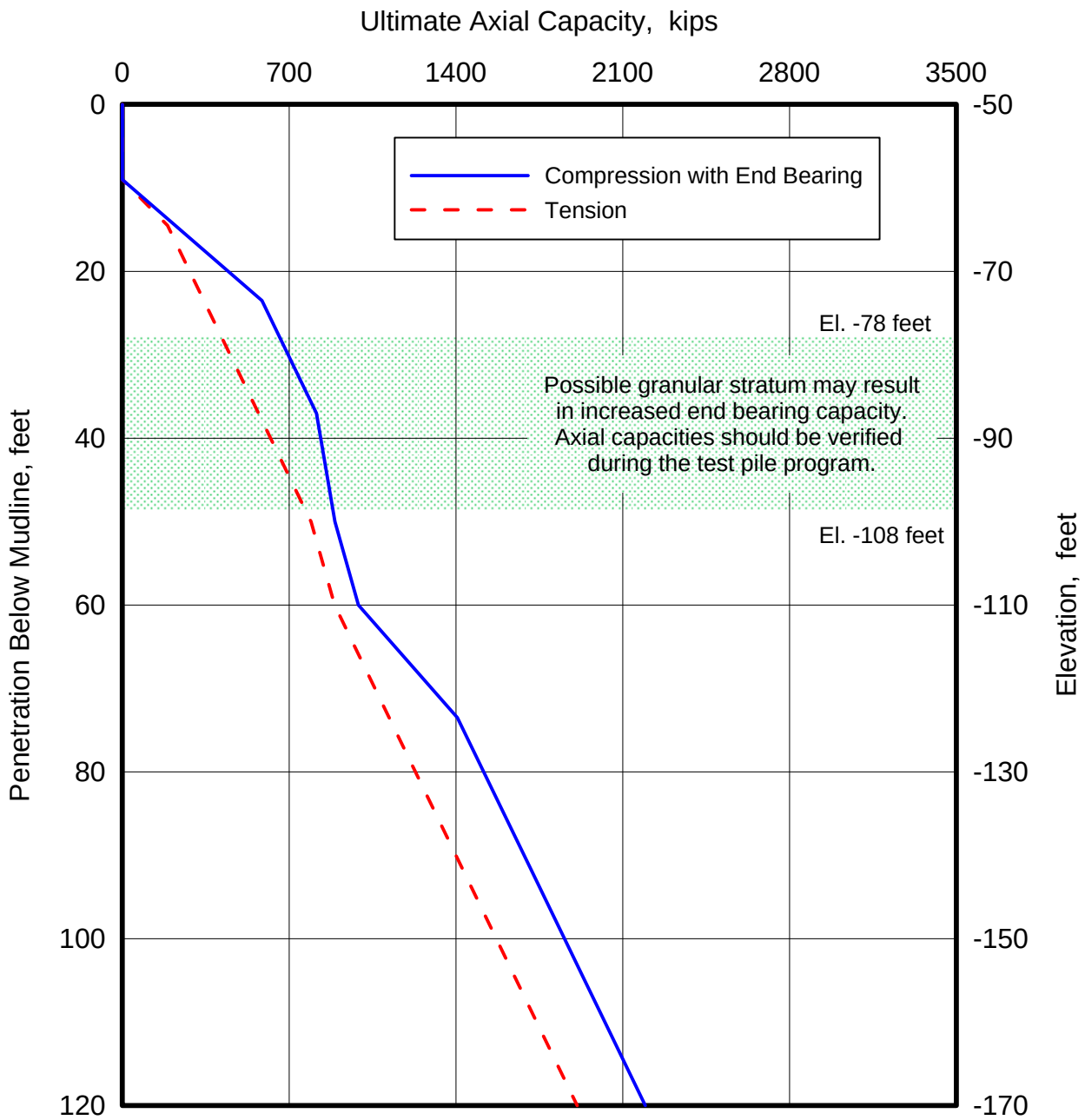
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -50 FEET
30-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 6c

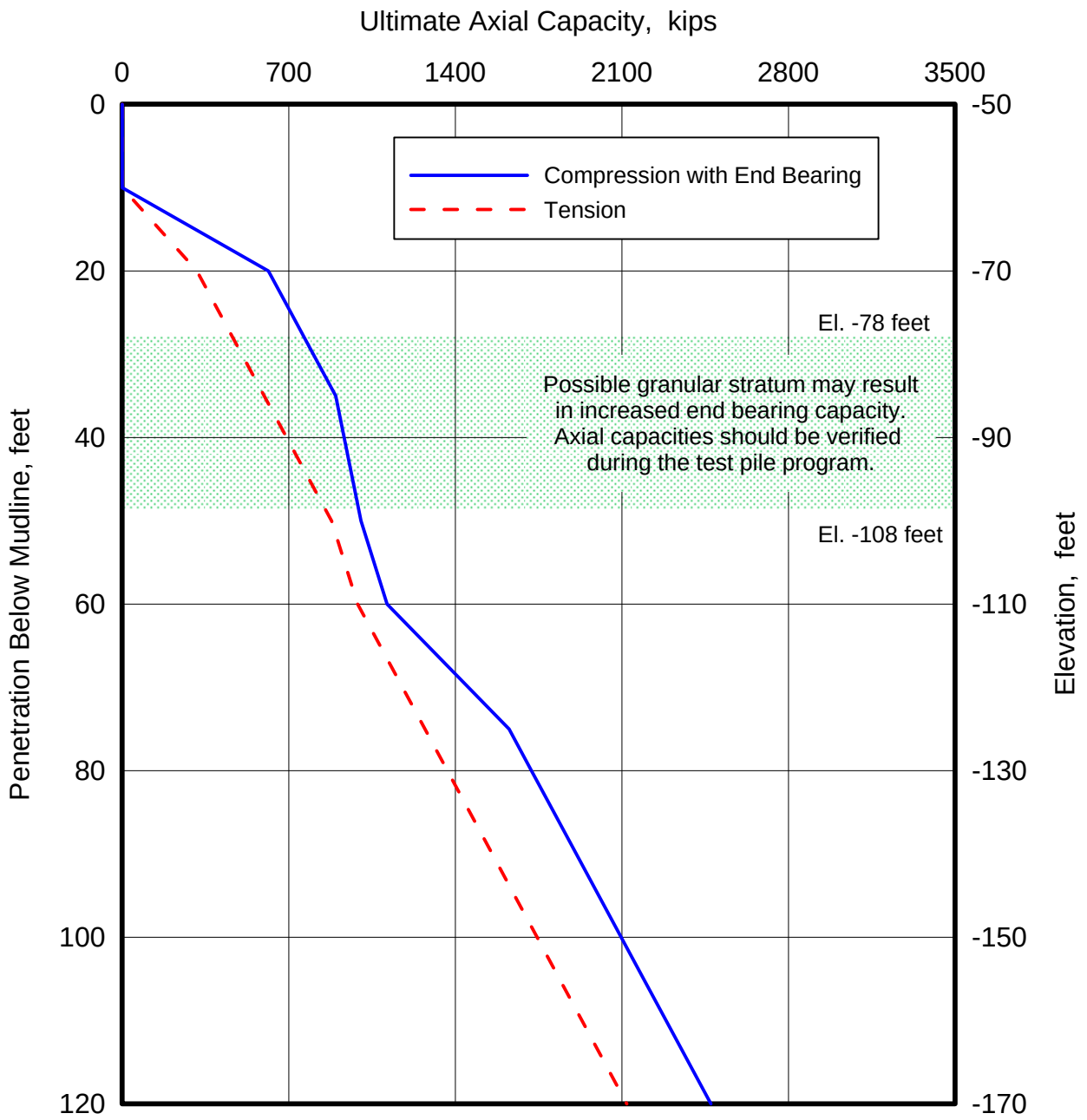


Notes:

1. These curves represent *ultimate* values of a driven hollow spun pre-cast, pre-stressed (PCPS) concrete pipe pile for compression with end bearing and tension. Factors of safety for compressive and tensile loading are discussed in the report text.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -50 FEET
54-INCH DIAMETER x 5-INCH WALL HOLLOW SPUN PCPS PILES

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS



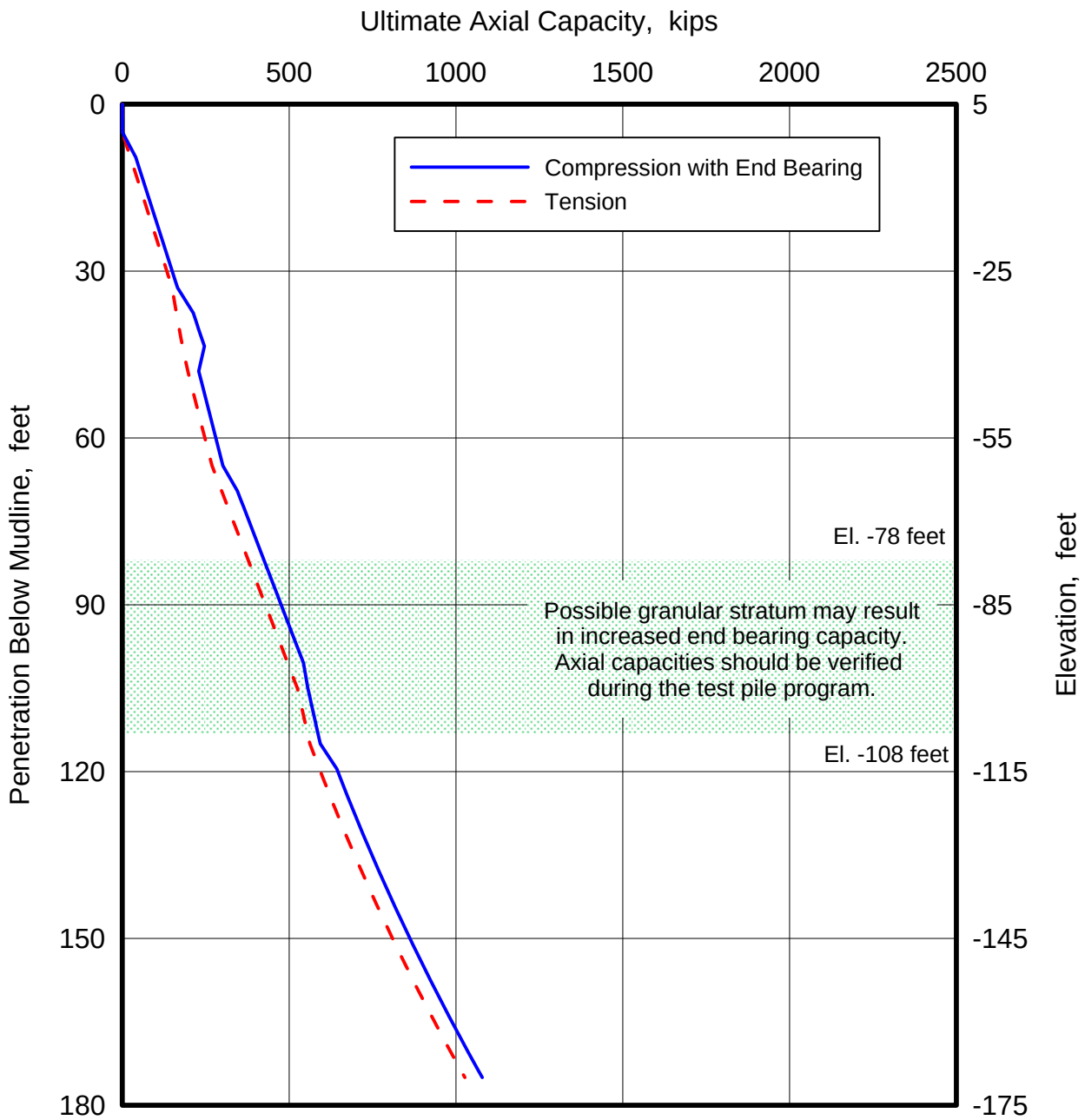
Notes:

1. These curves represent *ultimate* values of a driven hollow spun pre-cast, pre-stressed (PCPS) concrete pipe pile for compression with end bearing and tension. Factors of safety for compressive and tensile loading are discussed in the report text.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN. We have assumed a water surface elevation of El. 0 feet.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - DREDGE EL. -50 FEET
60-INCH DIAMETER x 6-INCH WALL HOLLOW SPUN PCPS PILES

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 6e



Notes:

1. These curves represent *ultimate* values of a driven square pre-cast, pre-stressed (PCPS) concrete pile for compression with end bearing and tension. Factors of safety for compressive and tensile loading are discussed in the report text.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Elevations presented herein are estimated based on site elevation data provided by LAN.
4. Pile installation guidelines are discussed in the report text.

ULTIMATE AXIAL CAPACITY CURVES - RELIEVING PLATFORM - EL. +5 FEET
16-INCH SQUARE PCPS CONCRETE PILE

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

PLATE 7a

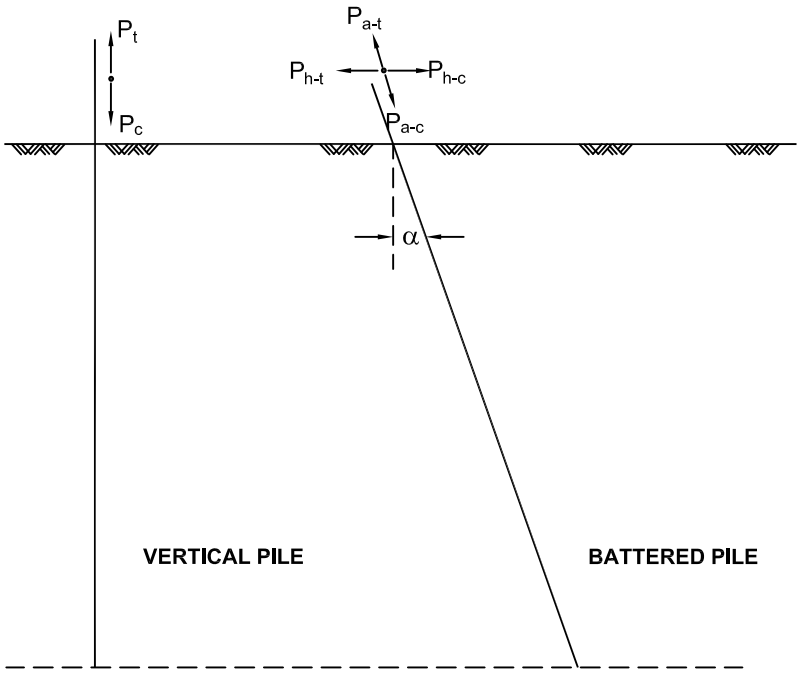
STEPS FOR COMPUTING BATTERED PILE CAPACITY

1. Determine ultimate tensile capacity (P_t) and ultimate compressive capacity (P_c) for vertical piles using curves presented in this report.
2. Compute ultimate end bearing capacity of vertical pile as $P_e = P_c - P_t$.
3. Compute skin friction component of battered pile (P_s) by multiplying P_t by the secant of the batter angle (sec α).

4a. For battered piles loaded in compression, compute the ultimate axial capacity of the pile (P_{a-c}) by adding the skin friction (P_s) and end bearing (P_e) capacities.

4b. For battered piles loaded in tension, the ultimate axial capacity of the pile (P_{a-t}) is equal to the skin friction capacity (P_s).

5a & 5b. To compute the horizontal capacity (P_h) of the battered pile, multiply the ultimate axial capacity by the sine of the batter angle (sin α).



$P_s = P_t \text{ SEC } \alpha$

$P_e = P_c - P_t$

$P_{a-c} = P_s + P_e$

$P_{a-t} = P_s$

$P_{h-c} = P_{a-c} \text{ SIN } \alpha$

$P_{h-t} = P_{a-t} \text{ SIN } \alpha$

NOTES:

1. Factor of safety of at least 2.0 recommended for compression and temporary tension.
2. Factor of safety of at least 3.0 recommended for sustained tension.
3. Vertical and battered piles must penetrate to equal elevations for this method to be applicable.
4. See text for other recommendations for battered piles.
5. Additional horizontal capacity can be developed due to flexural stiffness of battered pile. The method shown here does not include the flexural capacity of battered piles.

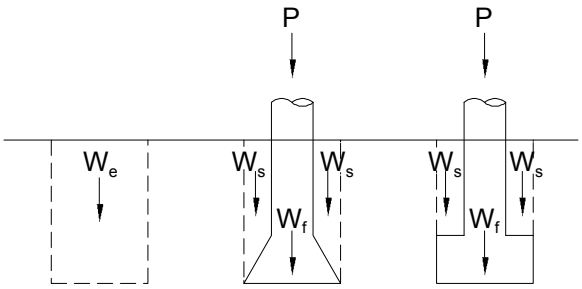
ULTIMATE CAPACITY OF BATTERED PILES

COMPUTATION METHOD

FOUNDATION DESIGN CRITERIA

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

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



SYMBOLS

TERMS AND SYMBOLS

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

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

W_s = Weight of soil located above foundation.*

W_f = Weight of foundation.*

A = Area of base of foundation.

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

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

BEARING PRESSURES

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

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

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

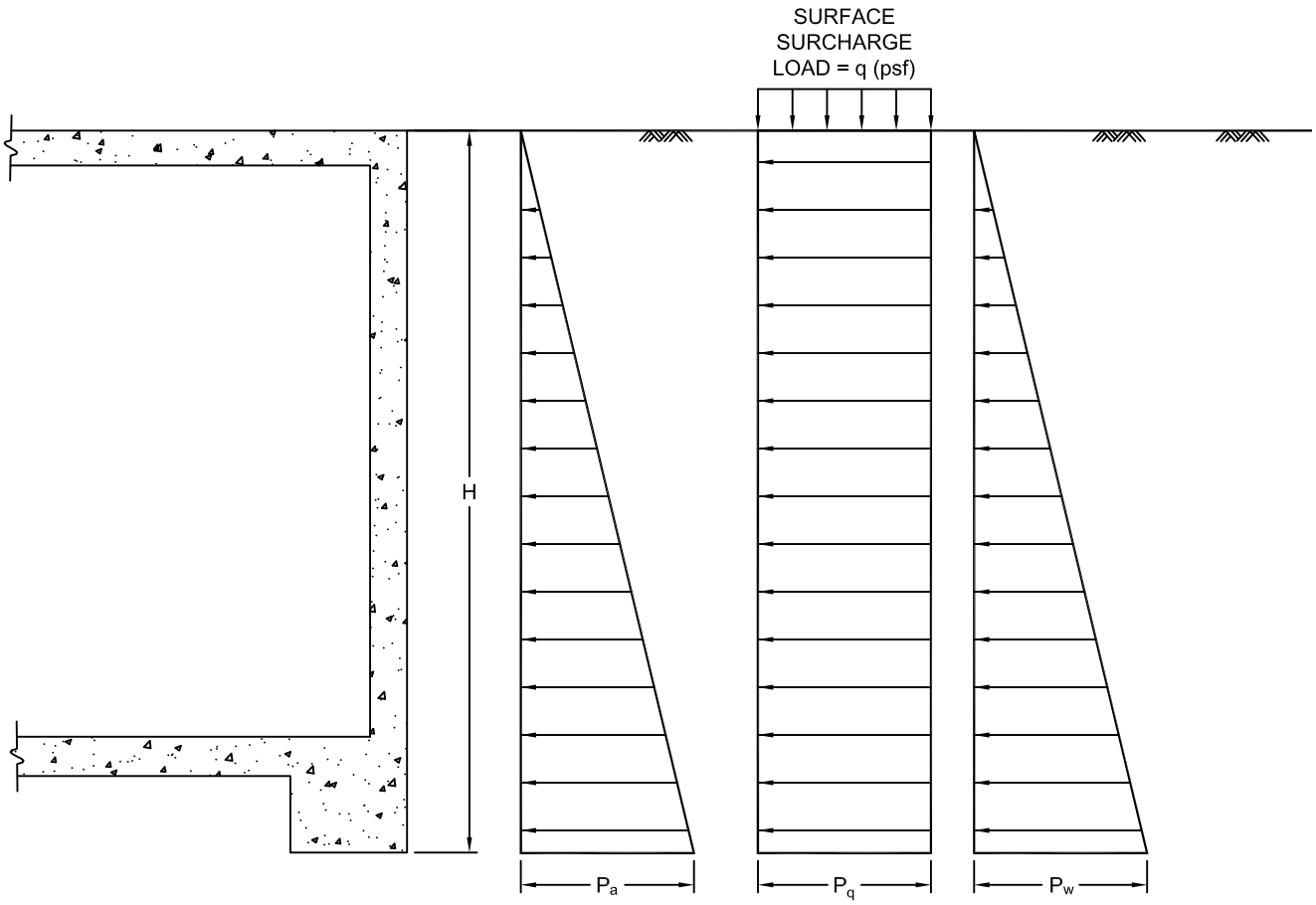
$$p' = 1/A\ (P + W_s + W_f - W_e)$$

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

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

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

COMPUTATION OF BEARING PRESSURES
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS



NON-YIELDING WALLS
Soil: P_a = 42 (H), psf
Surcharge: P_q = 0.5 (q), psf
Water: P_w = 63 (H), psf

H = Excavation Depth, ft
q = Surcharge Load, psf

NOTES:

1. The lateral earth pressures shown above are for soils in contact with permanent below grade walls.
2. Lateral earth pressures assume hydrostatic pressures that develop behind the wall are short term.
3. These represent ultimate values. Structural Engineer should apply approximate safety or design factors. See report text for additional details.

LATERAL EARTH PRESSURES FOR BELOW GRADE WALLS
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS
(NOT TO SCALE)

APPENDIX A

GEOTECHNICAL SOIL BORING LOGS

LOCATION: See Plate 2
COORDINATES: 29°51'24.4" N
93°56'34.3" W

SURFACE EL.: Approximately -43'

DEPTH, FT	BLOWS PER FOOT	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	STRATUM DESCRIPTION
	4			90					CLAY, firm, gray, with silt pockets
5									
									- firm to stiff, 8' to 18'
10				98	49	86	21	65	
									- with organic materials, 13' to 18'
15					46				
									- stiff to very stiff, with calcareous nodules below 18'
20			106		22	54	13	41	
		23.0							SILT, light gray, with sand and clay seams
25				88					
		28.0							CLAY, stiff, brown and gray
30	12			93					
									- very stiff below 33'
35									
		38.0							SILTY CLAY, stiff, brown and gray - with silt seams and silt pockets to 43'
40			99		27				
									- very stiff, 43' to 48'

SHEAR STRENGTH

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

KIPS PER SQ FT: 0.5 1.0 1.5 2.0 2.5

NOTES:

1. Mudline encountered approximately 43 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 7, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-1

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'24.4" N
93°56'34.3" W

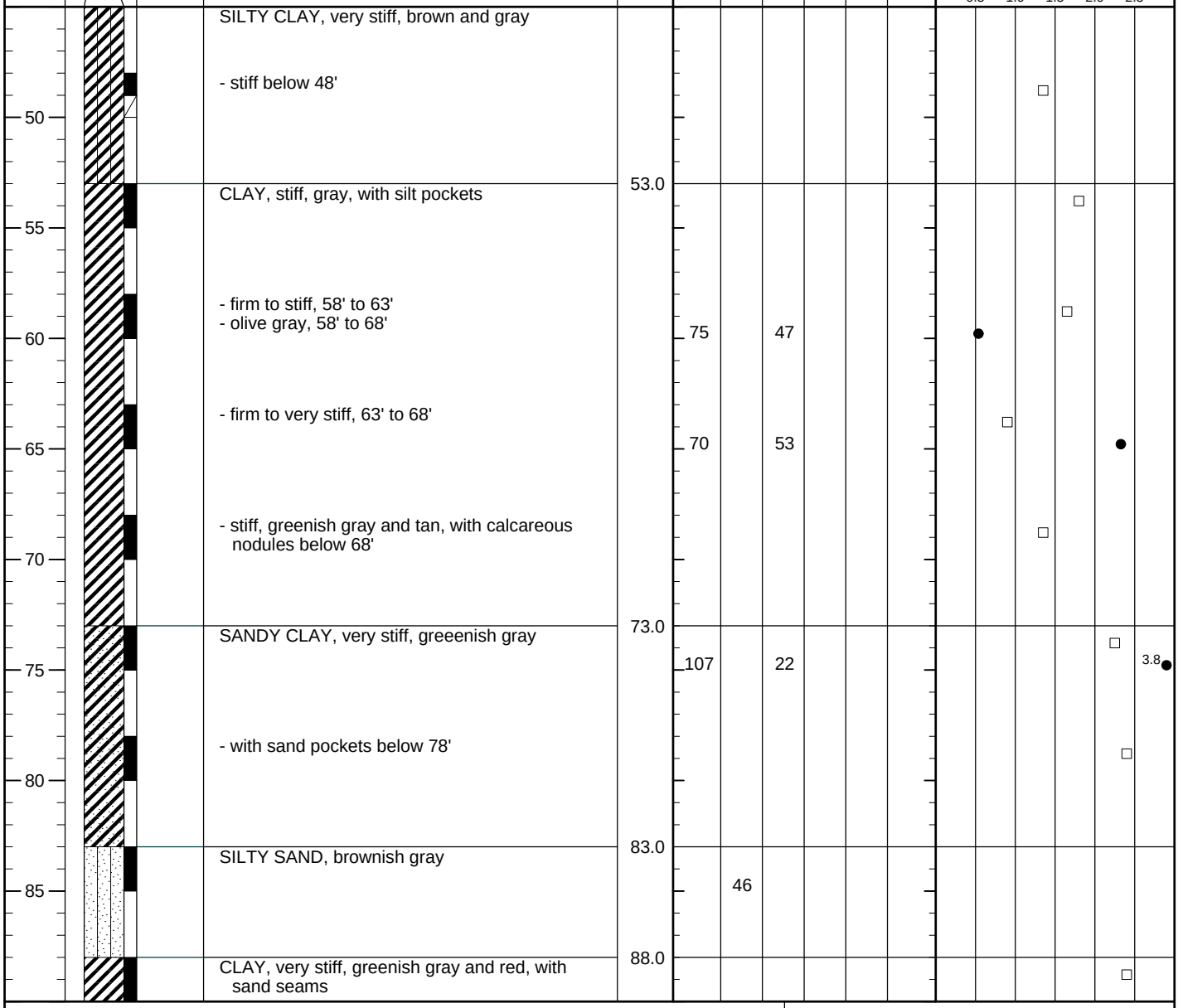
SURFACE EL.: Approximately -43'

DEPTH, FT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
	SILTY CLAY, very stiff, brown and gray							
	- stiff below 48'							
		53.0						
	CLAY, stiff, gray, with silt pockets							
	- firm to stiff, 58' to 63'							
	- olive gray, 58' to 68'		75		47			
	- firm to very stiff, 63' to 68'		70		53			
	- stiff, greenish gray and tan, with calcareous nodules below 68'							
		73.0						
	SANDY CLAY, very stiff, greeenish gray		107		22			
	- with sand pockets below 78'							
		83.0						
	SILTY SAND, brownish gray			46				
		88.0						
	CLAY, very stiff, greenish gray and red, with sand seams							

SHEAR STRENGTH

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

KIPS PER SQ FT
0.5 1.0 1.5 2.0 2.5



NOTES:
1. Mudline encountered approximately 43 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 7, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-1
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'24.4" N
93°56'34.3" W

SURFACE EL.: Approximately -43'

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					CLAY, very stiff, greenish gray and red, with sand seams	93.0						
95					SANDY CLAY, stiff to very stiff, olive gray and brown, with sand pockets - with SAND layer, 97' to 98'	98.0	97		27			
100				22	CLAY, very stiff, olive gray and red, with silt pockets	108.0		99				
105												
110					SANDY CLAY, very stiff, gray and greenish gray, with sand pockets	118.0						
115												
120					CLAY, very stiff, brown	120.0						
125												
130												

SHEAR STRENGTH

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

KIPS PER SQ FT: 0.5 1.0 1.5 2.0 2.5

NOTES:
1. Mudline encountered approximately 43 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 7, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-1
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'26.2" N
93°56'32.2" W

SURFACE EL.: Approximately -45'

DEPTH, FT	STRATUM DEPTH, FT	STRATUM DESCRIPTION	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		CLAY, firm to stiff, olive gray - with sand seams and shell fragments to 13'						
5				86	41	86	21	65
		- stiff, 8' to 13' - greenish gray, with organic materials, 8' to 18'						
10			101		25			
		- very stiff, with sand pockets, 13' to 33'						
15								
		- olive gray, 18' to 23'						
20			105	87	21			
		- brown and gray, slickensided, 23' to 28'						
25								
		- light brown and gray, 28' to 33'						
30								
		- stiff, brown, 33' to 38'						
35			91	100	32			
		- very stiff, brown and gray, 38' to 48' - with sand seams, 38 to 53'						
40								

SHEAR STRENGTH

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

KIPS PER SQ FT
0.5 1.0 1.5 2.0 2.5

NOTES:
1. Mudline encountered approximately 45 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 9, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-2
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'26.2" N
93°56'32.2" W

SURFACE EL.: Approximately -45'

DEPTH, FT	STRATUM DEPTH, FT	STRATUM DESCRIPTION	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		CLAY, very stiff, brown and gray - with sand seams to 53'						
		- stiff, gray, 48' to 53'						
50								
		- firm to very stiff, 53' to 58' - olive gray, 53' to 63'						
55			74	100	48			
		- stiff to very stiff, 58' to 73'						
60			69	100	54			
		- greenish gray below 63'						
65								
70			98		27			
		- very stiff below 73'						
75								
	78.0	SAND, greenish gray, with silt						
80				8				
	83.0	SANDY CLAY, stiff, greenish gray, with sand pockets						
85			107		20			
	88.0	SILTY CLAY, very stiff, greenish gray and dark brown		100				

CLASSIFICATION

SHEAR STRENGTH

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

KIPS PER SQ FT
0.5 1.0 1.5 2.0 2.5

NOTES:
1. Mudline encountered approximately 45 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 9, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-2
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'26.2" N
93°56'32.2" W

SURFACE EL.: Approximately -45'

DEPTH, FT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
	SILTY CLAY, very stiff, greenish gray and dark brown							
	- with SAND layer, 93' to 94'	94.0						
95	SANDY CLAY, stiff, greenish gray and dark brown							
	- greenish gray and dark brown below 98'							
100								
105		108.0						
	CLAY, stiff to very stiff, gray, with sand pockets							
110			109	87	20			
115								
	- very stiff, greenish gray below 118'							
120		120.0						
125								
130								

SHEAR STRENGTH

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

KIPS PER SQ FT
0.5 1.0 1.5 2.0 2.5

NOTES:
1. Mudline encountered approximately 45 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 9, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-2
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'27.9" N
93°56'31.0" W

SURFACE EL.: Approximately -45'

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					SILTY CLAY, stiff, brownish gray							
5								98	52			
					- with shell fragments, 8' to 13' - very stiff, greenish gray below 8'							
10							108		18			
					- with sand pockets below 13'							
15												
					SANDY CLAY, stiff, greenish gray	18.0						
20							106	68	21			
					CLAY, very stiff, gray and brown, with sand seams	23.0						
25												
					- brown and gray, 28' to 33' - stiff, 28' to 43'							
30							108	79	20			
					- brown, 33' to 38'							
35												
					- brown and tan, 38' to 43'							
40				9				99				
					- very stiff, olive gray and dark gray, 43' to 48'							

SHEAR STRENGTH

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

KIPS PER SQ FT
0.5 1.0 1.5 2.0 2.5

NOTES:
1. Mudline encountered approximately 45 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 11, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-3
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'27.9" N
93°56'31.0" W

SURFACE EL.: Approximately -45'

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					CLAY, very stiff, olive gray and dark gray, with sand seams							
					- olive gray, 48' to 63'							
					- stiff to very stiff, 48' to 58'							
50							80		42			
55							75		47			
					- firm to stiff, 58' to 63'							
60							69		53			
					- stiff, greenish gray, with shell fragments and organic materials below 63'							
65							81		40			
						68.0						
					SANDY CLAY, stiff, greenish gray							
					- with organic materials to 73'							
70												
75												
					- very stiff below 78'							
						79.0						
					SILTY SAND, medium dense, greenish gray							
80				20				26				
						83.0						
					CLAY, stiff, gray, with sand pockets							
85				20				93				
					- stiff to very stiff, dark brown and greenish gray below 88'							
							98		27			

CLASSIFICATION

SHEAR STRENGTH

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

KIPS PER SQ FT: 0.5 1.0 1.5 2.0 2.5

NOTES:
1. Mudline encountered approximately 45 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 11, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-3

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'27.9" N
93°56'31.0" W

SURFACE EL.: Approximately -45'

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
					CLAY, stiff to very stiff, dark brown and greenish gray, with sand pockets								
					SANDY SILT, dense, dark brown	93.0							
95				31				79					
100				48	- dark brown and greenish gray, with clay seams below 98'			91					
105													
					CLAY, very stiff, dark brown, greenish gray, and red	108.0							
110													
115													
					- greenish gray below 118'								
120						120.0							
125													
130													

CLASSIFICATION

SHEAR STRENGTH

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

KIPS PER SQ FT: 0.5 1.0 1.5 2.0 2.5

NOTES:

1. Mudline encountered approximately 45 feet below the water surface.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: December 11, 2012
TOTAL DEPTH: 120'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: Mudline to 120'
BACKFILL: Soil Cuttings
LOGGER: E. Schulak

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LOG OF BORING NO. B-3

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'24.8" N
93°56'36.0" W

SURFACE EL.: Approximately +12'

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					FILL: SANDY CLAY, firm to stiff, dark brown and tan, with sand pockets and shell fragments			29	7	40	12	28
					- dark gray below 2'				24			
					FILL: CLAYEY SAND, dark gray, with roots	4.0						
5												
					FILL: CLAY, very soft to soft, dark gray, with sand	6.0						
				2				75				
					SANDY CLAY, firm to stiff, gray	8.0						
					- with organic material to 14'		101		24	53	12	41
10					- gray and tan, 10' to 18'							
									25			
					- stiff, 12' to 23'							
							106		22			
					- with calcareous nodules and ferrous nodules, 14' to 18'							
15												
					- brown and tan, with sand pockets, 18' to 23'							
							104	66	23	28	18	10
20												
					- with sand seams, 23' to 28'							
					- brown, 23' to 33'							
25					- firm to stiff below 23'		104	74	23			
					- with sand pockets below 28'							
30												
					- olive gray and gray, 33' to 38'							
35							99		27			
					CLAY, firm, olive gray, with silt seams	38.0						
40						40.0	84	100	37	60	17	43

SHEAR STRENGTH

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

KIPS PER SQ FT
0.5 1.0 1.5 2.0 2.5

NOTES:
1. ▽: Water First Noticed. ▼: Depth To Water after 15 minutes.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: October 18, 2012
TOTAL DEPTH: 40'
CAVED DEPTH: 6'
DRY AUGER: Surface to 8'
WET ROTARY: 8' to 40'
BACKFILL: Cement-Bentonite Grout
LOGGER: E. Schulak

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LOG OF BORING NO. B-4

BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

LOCATION: See Plate 2
COORDINATES: 29°51'27.4" N
93°56'33.5" W

SURFACE EL.: Approximately +12'

DEPTH, FT	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		ASPHALT PAVEMENT - 1.5"	0.1						
	20	FILL: CLAYEY SAND, medium dense, brown			33	16	27	14	13
	17				32				
5	25	- brown and dark gray, 4' to 5' - with shell fragments and gravel below 4' - dark gray and greenish gray below 5' - with SILTY SAND layer below 6'			42				
		CLAY, firm to stiff, dark gray - with organic material to 8' - olive gray, 8' to 10' - tan and gray, with ferrous nodules below 10'	6.5	96	94	26	47	15	32
10						36			
		SANDY CLAY, stiff, tan and gray - with ferrous nodules to 18'	12.0	104		23	46	12	34
15									
		- gray and tan, 18' to 23'							
20				105		22			
		- brown, 23' to 28'							
25									
		- firm to stiff, 28' to 38' - brown and gray, with sand pockets below 28'							
30				100		25			
		- with sand seams, 33' to 38'							
35					64				
		- stiff below 38'							
40			40.0	106		22			

SHEAR STRENGTH

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

KIPS PER SQ FT
0.5 1.0 1.5 2.0 2.5

NOTES:
1. Free water not observed during drilling activities to a depth of 8 feet.
2. Terms and symbols defined on Plates A-6a and A-6b.
3. Boring coordinates obtained using a hand-held GPS device.

DATE: October 18, 2012
TOTAL DEPTH: 40'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 8'
WET ROTARY: 8' to 40'
BACKFILL: Cement-Bentonite Grout
LOGGER: E. Schulak

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LOG OF BORING NO. B-5
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

SOIL TYPES

Sand
Silt
Clay
Gravel
Silty Sand
Sandy Silt
Sandy Clay
Peat or Highly Organic
Clayey Sand
Clayey Silt
Silty Clay
Debris or Mixed Fill
Asphalt
Concrete

SAMPLER TYPES

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

SOIL GRAIN SIZE

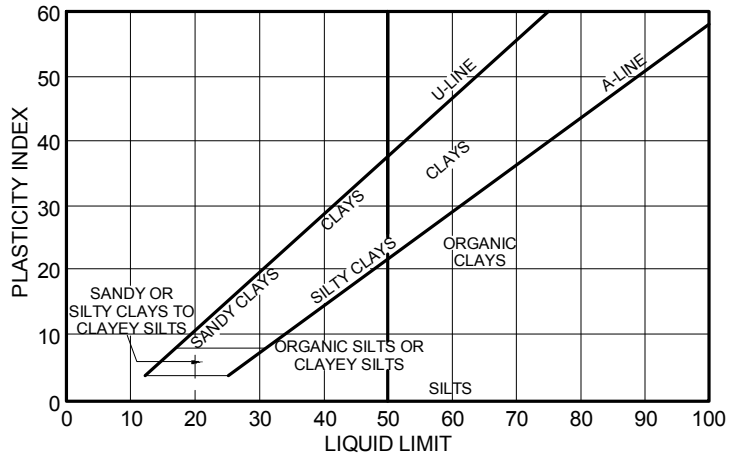
U.S. Standard Sieve

Boulders	Cobbles	Gravel		Sand			Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine		

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

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002 (mm)

PLASTICITY CHART



SOIL STRUCTURE

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

TERMS AND SYMBOLS USED ON BORING LOGS

SOIL CLASSIFICATION (1 of 2)

STANDARD PENETRATION TEST (SPT)

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

DENSITY OF GRANULAR SOILS

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

*Estimated from sampler driving record.
**Requires correction for depth, groundwater level, and grain size.

STRENGTH OF COHESIVE SOILS

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

SHEAR STRENGTH TEST METHOD

U - Unconfined Q = Unconsolidated - Undrained Triaxial

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

HAND PENETROMETER CORRECTION

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

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

TERMS AND SYMBOLS USED ON BORING LOGS

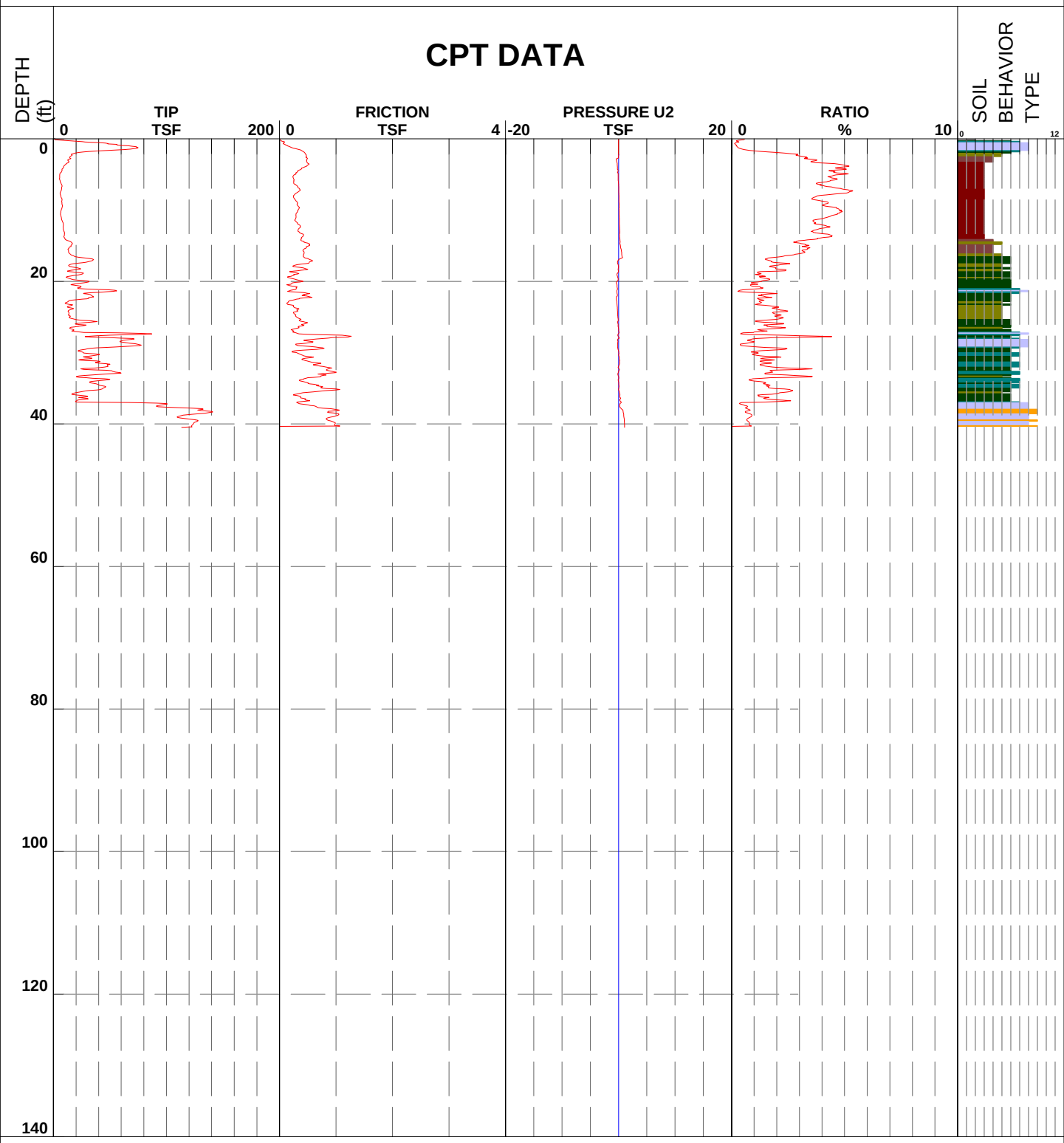
SOIL CLASSIFICATION (2 of 2)

APPENDIX B

CPT SOUNDING LOGS

Job Number	04.1012-0193	CPT Number	CPT-01	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	15-Oct-2012 14:24:25	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				

CPT DATA

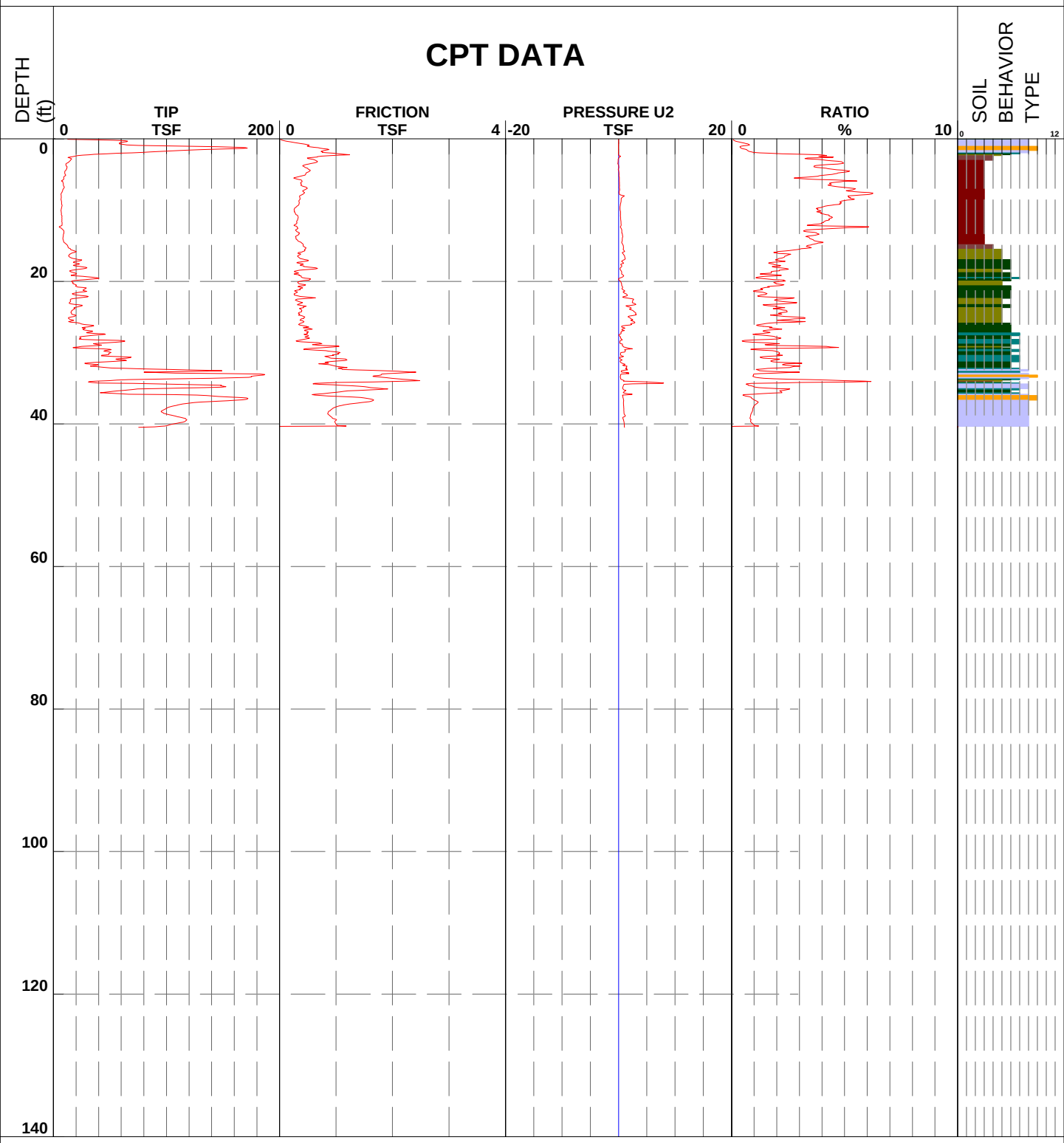


1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Robertson et al. 1986 * Overconsolidated or Cemented

Job Number	04.1012-0193	CPT Number	CPT-02	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	15-Oct-2012 15:38:39	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				

CPT DATA

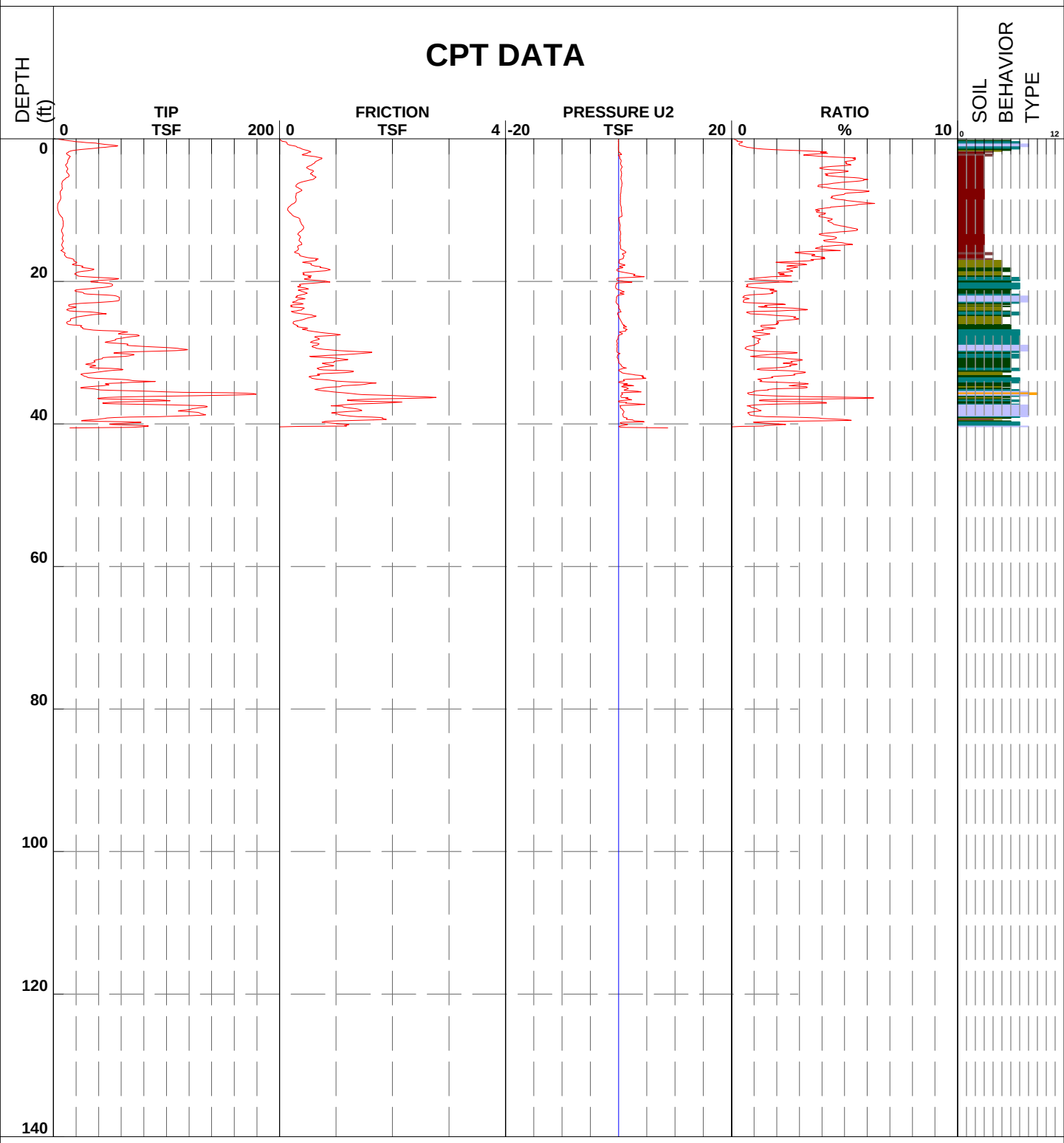


1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Robertson et al. 1986 * Overconsolidated or Cemented

Job Number	04.1012-0193	CPT Number	CPT-03	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	16-Oct-2012 07:35:12	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				

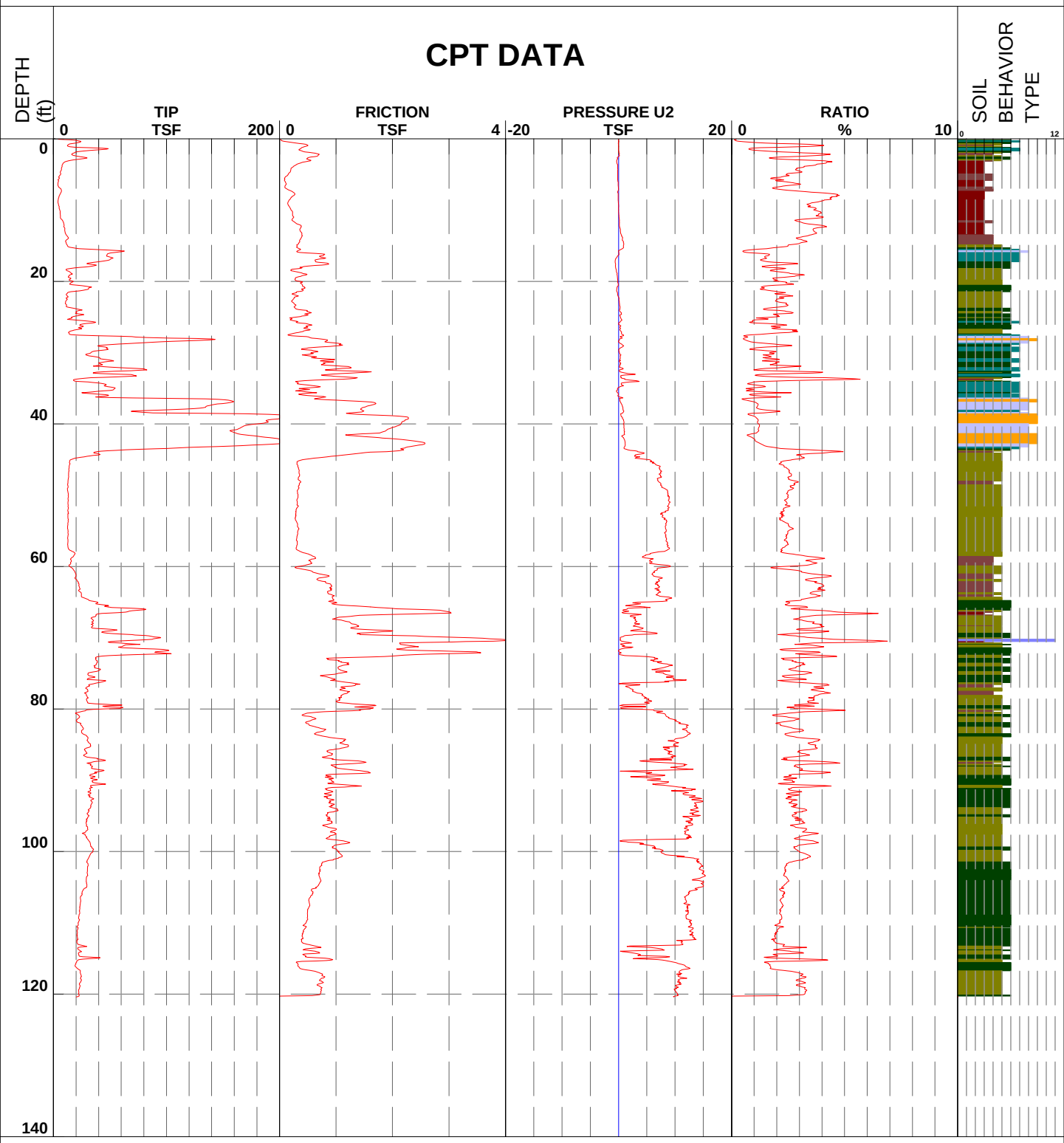
CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Robertson et al. 1986 * Overconsolidated or Cemented

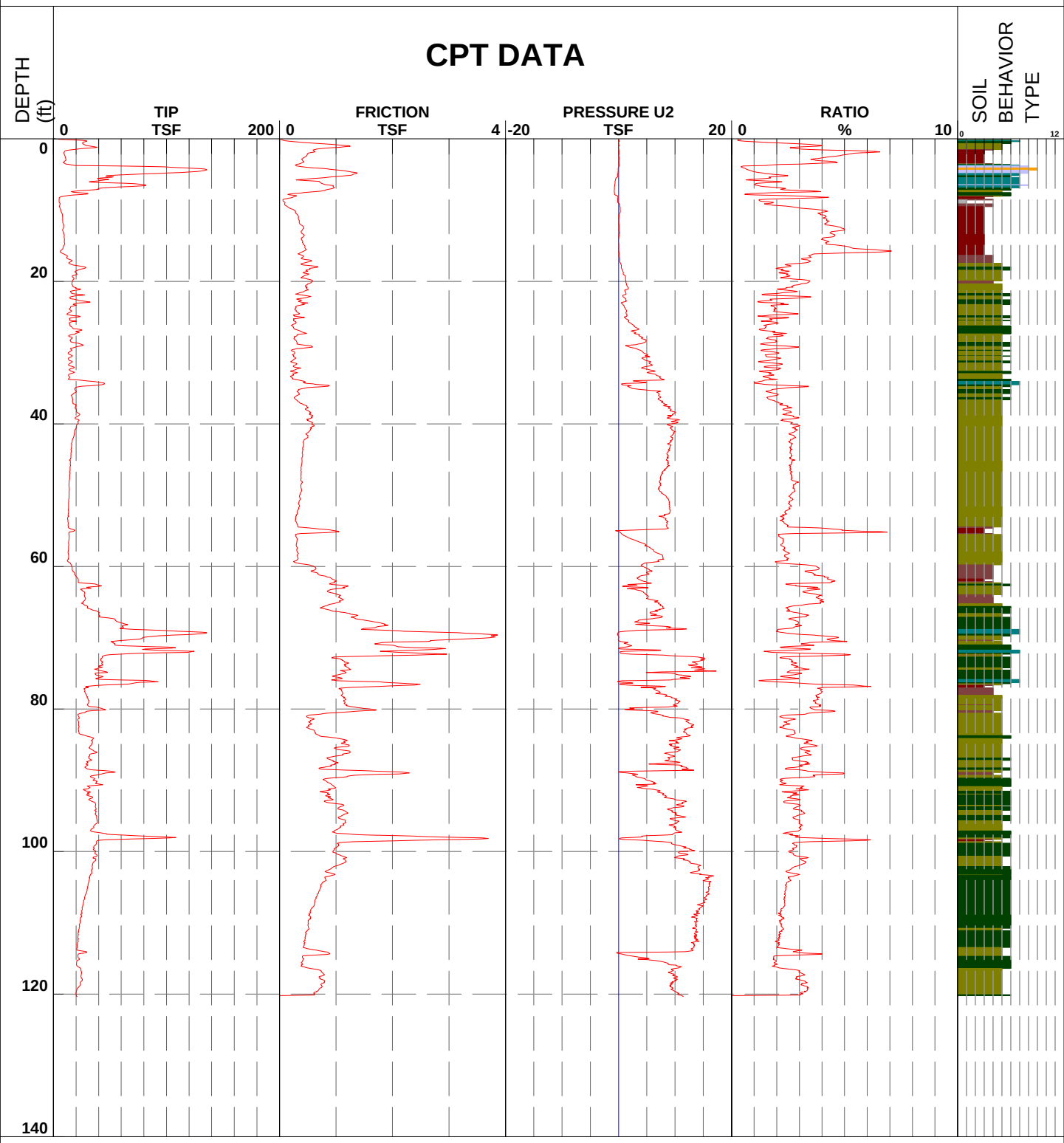
Job Number	04.1012-0193	CPT Number	CPT-04	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	16-Oct-2012 09:22:11	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				



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 (*)

Robertson et al. 1986 * Overconsolidated or Cemented

Job Number	04.1012-0193	CPT Number	CPT-05	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	16-Oct-2012 11:27:19	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				

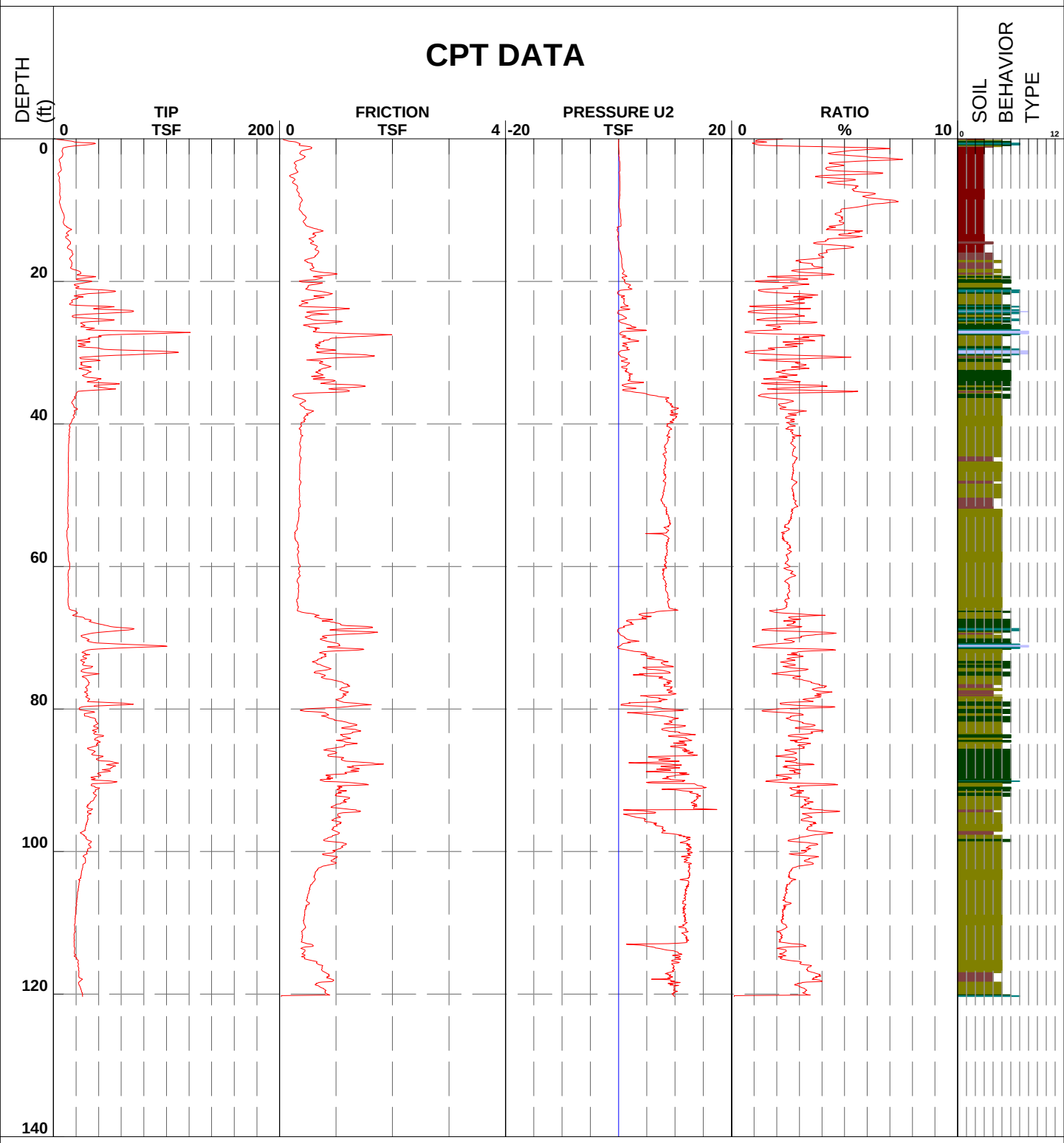


1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Robertson et al. 1986 * Overconsolidated or Cemented

Job Number	04.1012-0193	CPT Number	CPT-06	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	16-Oct-2012 13:06:08	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				

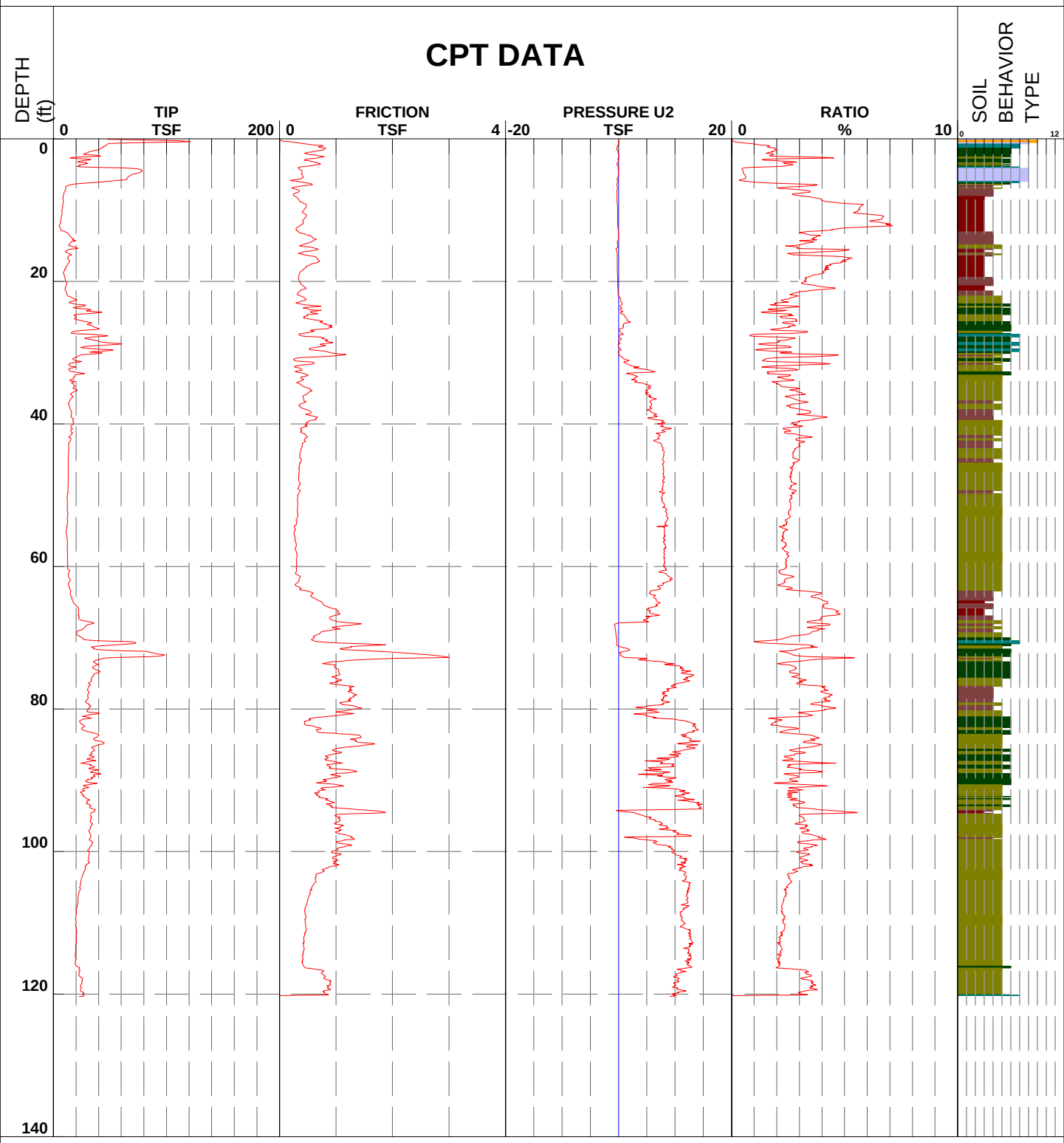
CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Robertson et al. 1986 * Overconsolidated or Cemented

Job Number	04.1012-0193	CPT Number	CPT-07	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	17-Oct-2012 08:39:59	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				

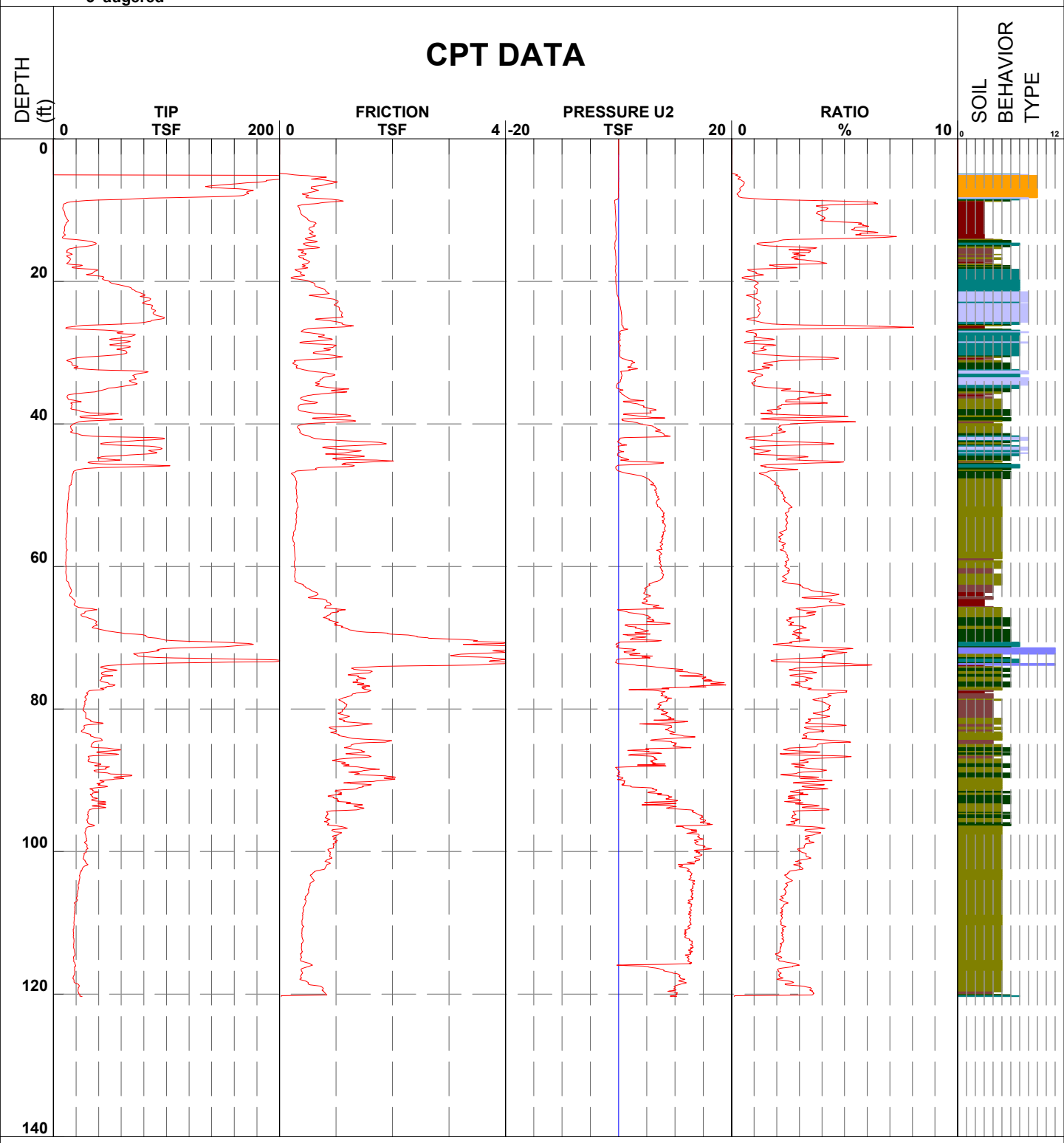


1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Robertson et al. 1986 * Overconsolidated or Cemented

Job Number	04.1012-0193	CPT Number	CPT-08	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	17-Oct-2012 12:33:55	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				

5' augered

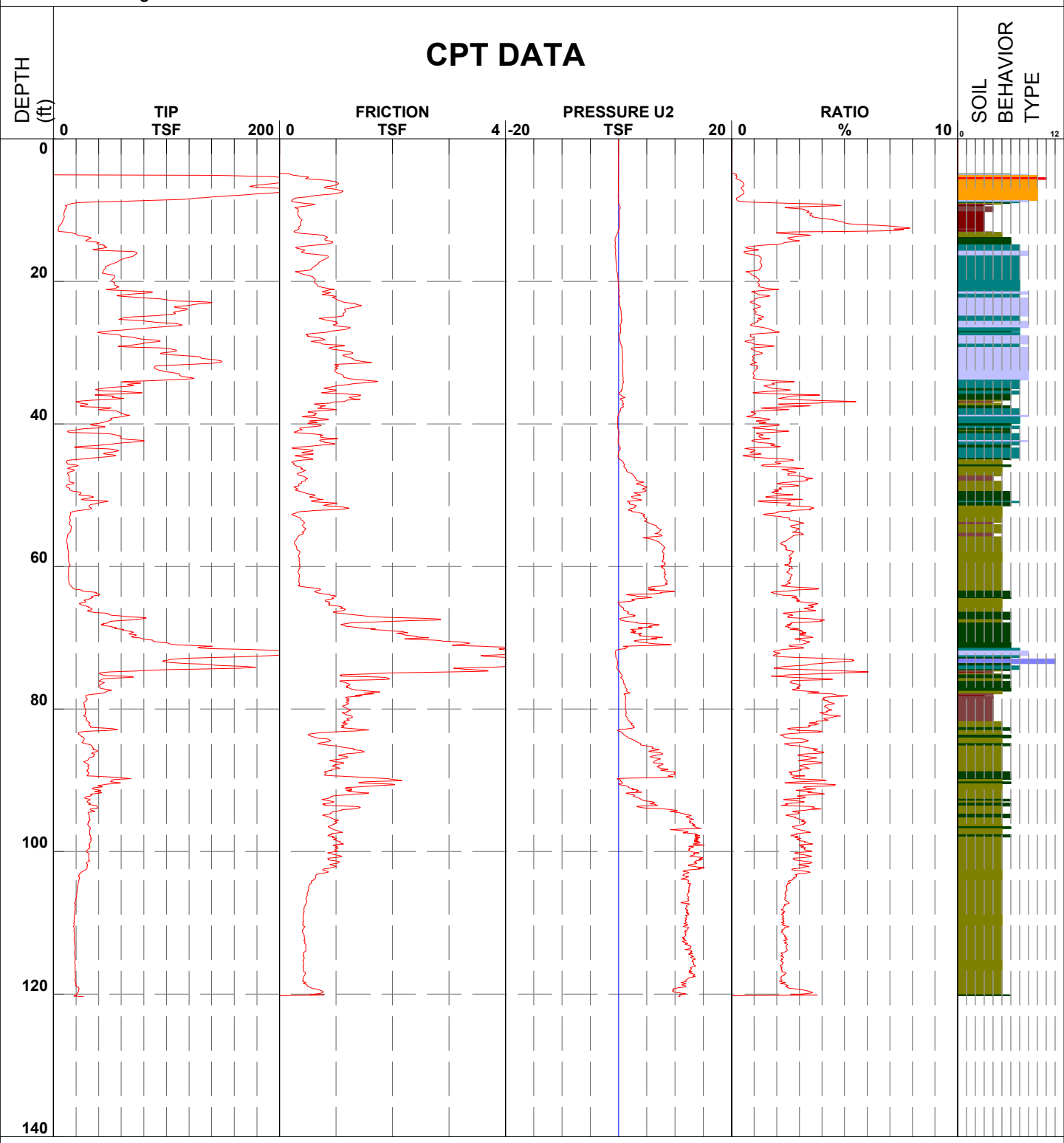


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Robertson et al. 1986 * Overconsolidated or Cemented

Job Number	04.1012-0193	CPT Number	CPT-09	Location	Port Arthur, Texas
Operator	Herbert Jackson	Date and T	17-Oct-2012 10:42:19	Cone Number	F7.5CKE2HAW21629
Client	Fugro Consultants, Inc.				

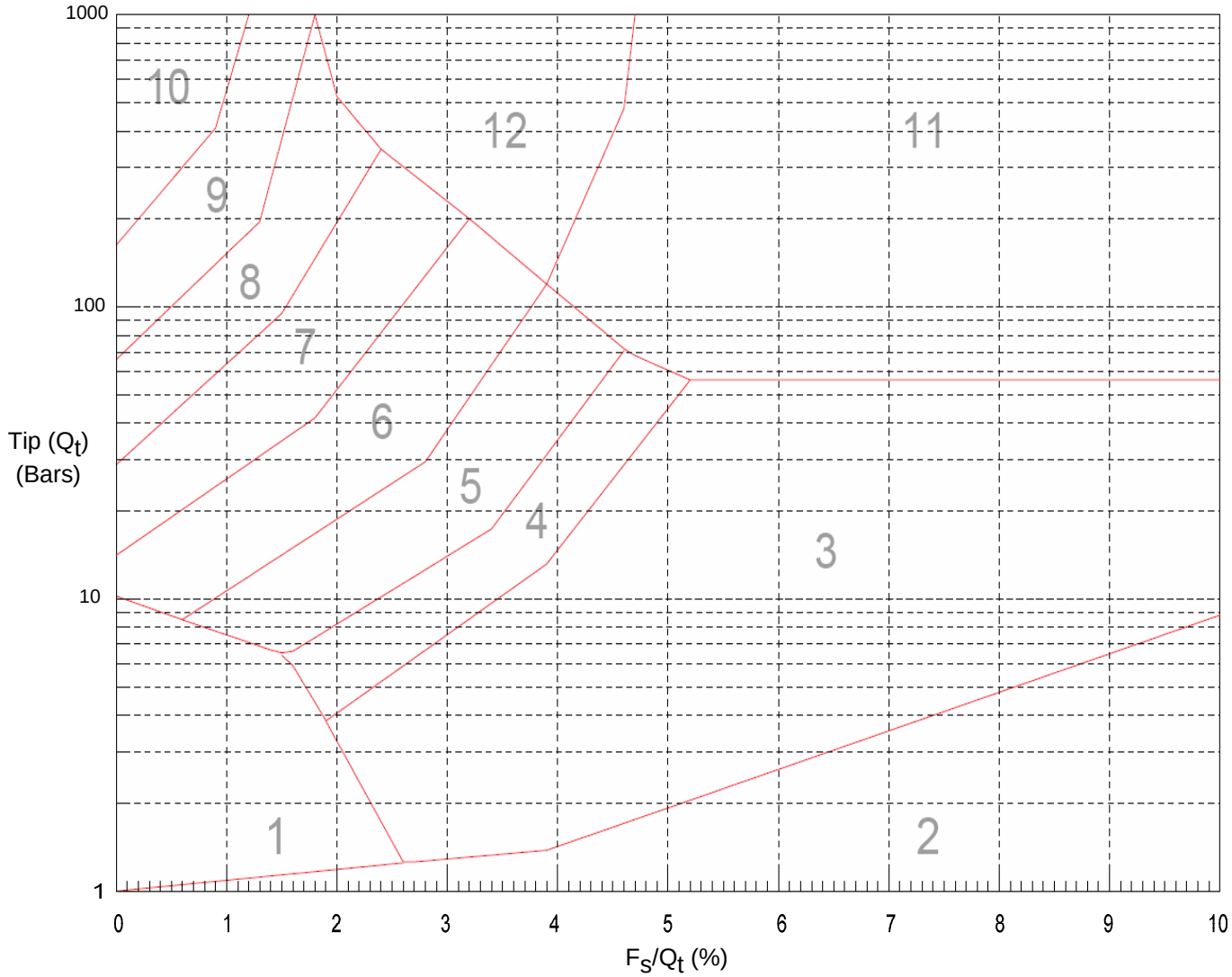
5' augered



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 (*)

Robertson et al. 1986 * Overconsolidated or Cemented

Classification Data:
Robertson and Campanella UBC-1986



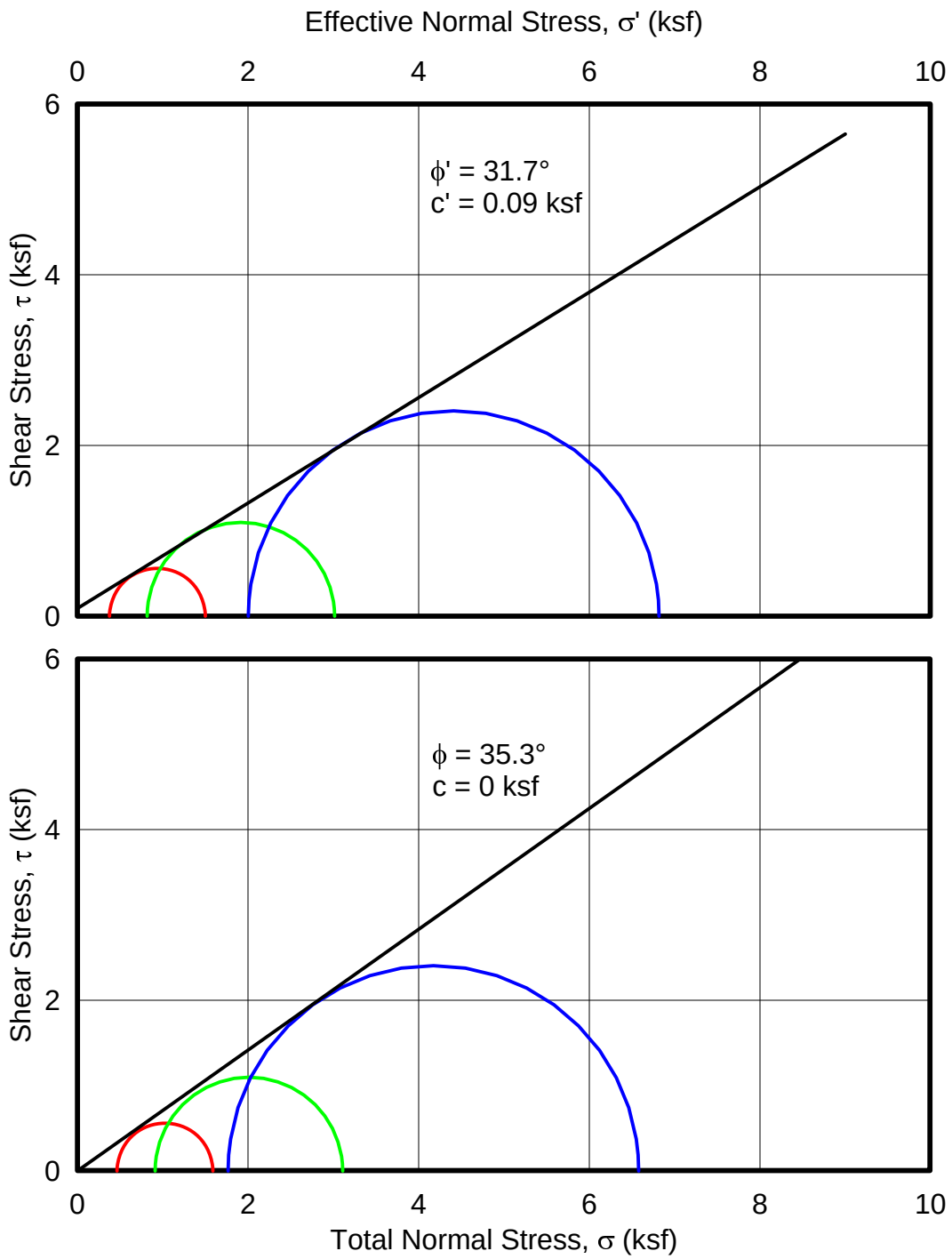
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

* overconsolidated or cemented

12 ZONE SOIL BEHAVIOR CHART
FOR CONE PENETROMETER TESTS

APPENDIX C

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS



MATERIAL: SANDY CLAY, brown and tan with sand pockets

INITIAL WATER CONTENT: 23%	LIQUID LIMIT: 28
INITIAL WET UNIT WEIGHT: 127 pcf	PLASTIC LIMIT: 18
INITIAL DRY UNIT WEIGHT: 104 pcf	PLASTICITY INDEX: 10
PERCENT PASSING NO. 200 SIEVE: 66	SPECIFIC GRAVITY: 2.70 (assumed)

MULTI STAGE CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

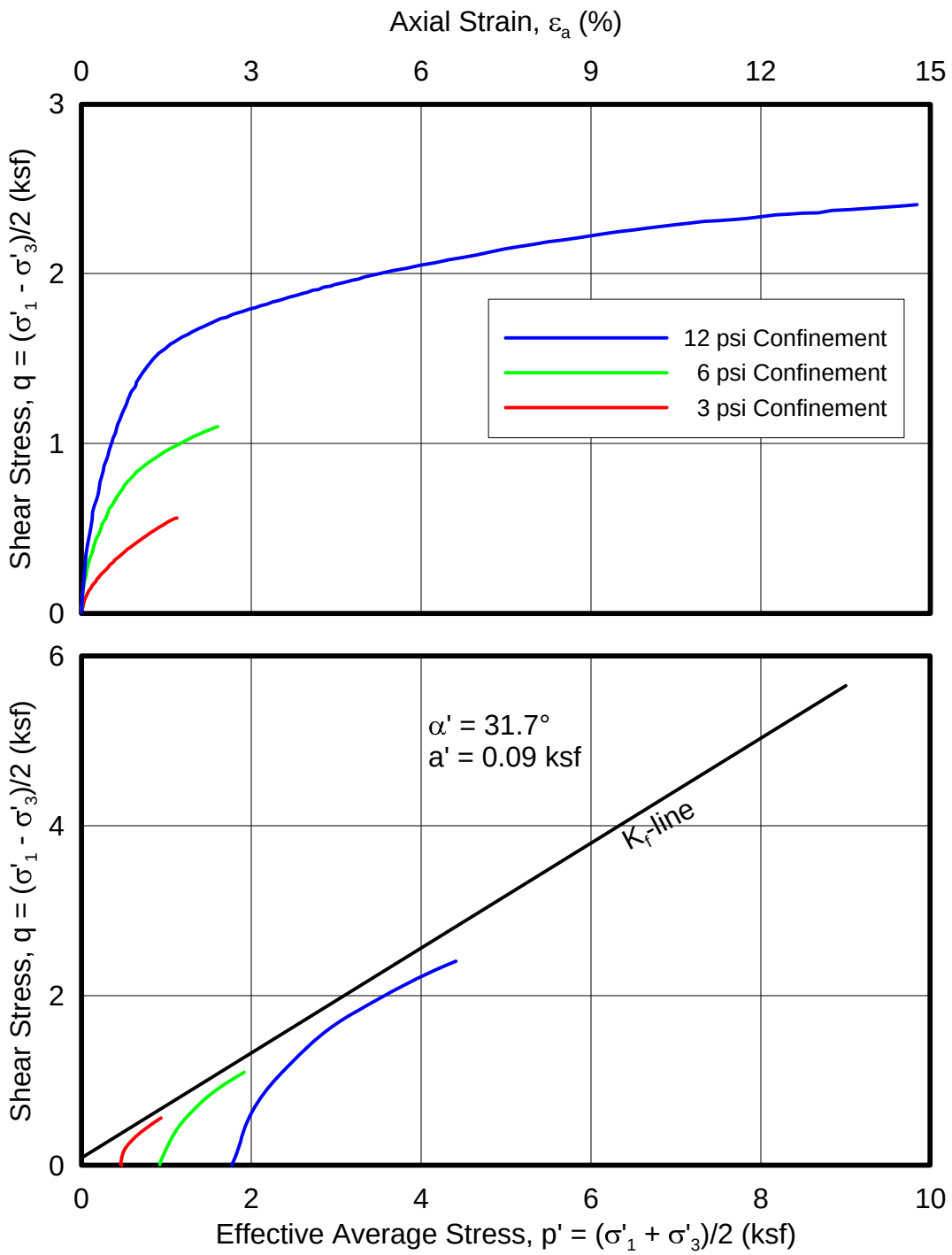
ISOTROPICALLY CONSOLIDATED - MOHR'S CIRCLES

BORING B-4, 20 FEET DEPTH

BERTH 6 AND BULKHEAD WALL EXPANSION

PORT OF PORT ARTHUR

PORT ARTHUR, TEXAS

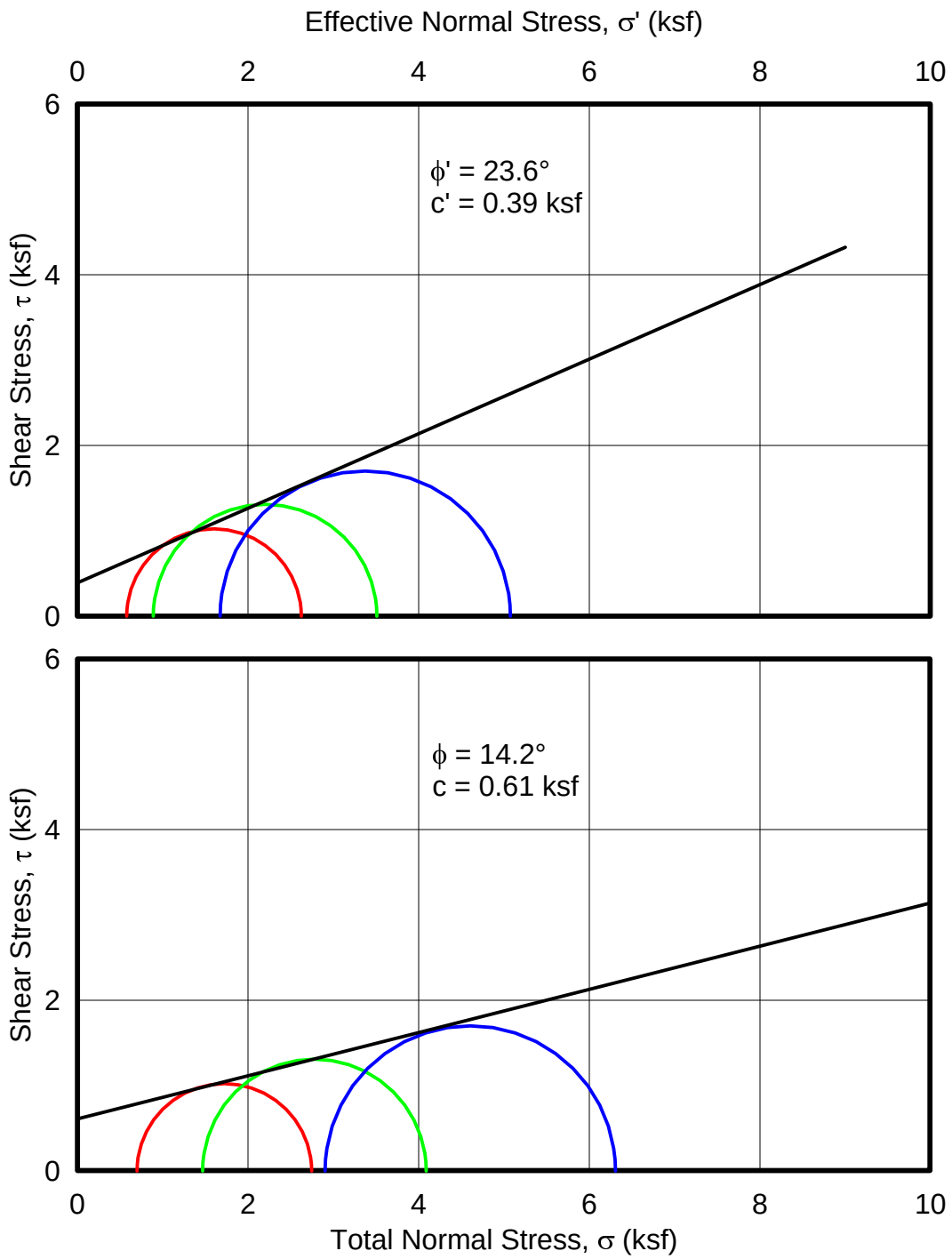


MATERIAL: SANDY CLAY, brown and tan, with sand pockets

INITIAL WATER CONTENT: 23%	LIQUID LIMIT: 28
INITIAL WET UNIT WEIGHT: 127 pcf	PLASTIC LIMIT: 18
INITIAL DRY UNIT WEIGHT: 104 pcf	PLASTICITY INDEX: 10
PERCENT PASSING NO. 200 SIEVE: 66	SPECIFIC GRAVITY: 2.70 (assumed)

MULTI STAGE CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS AND STRAIN CURVES
BORING B-4, 20 FEET DEPTH
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS



MATERIAL: CLAY, olive gray with silt seams

INITIAL WATER CONTENT: 37%	LIQUID LIMIT: 60
INITIAL WET UNIT WEIGHT: 115 pcf	PLASTIC LIMIT: 17
INITIAL DRY UNIT WEIGHT: 84 pcf	PLASTICITY INDEX: 43
PERCENT PASSING NO. 200 SIEVE: 100	SPECIFIC GRAVITY: 2.75 (assumed)

MULTI STAGE CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

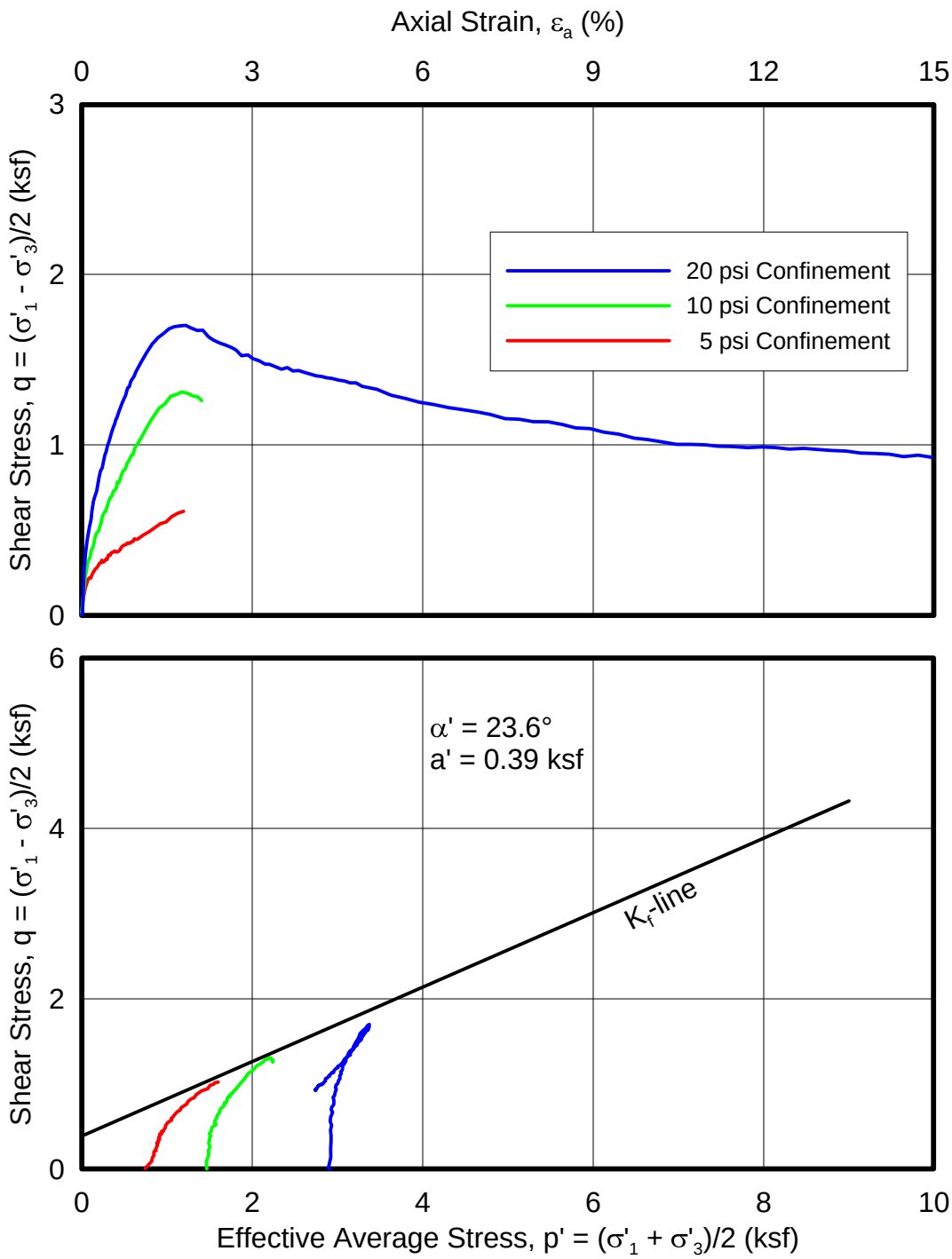
ISOTROPICALLY CONSOLIDATED - MOHR'S CIRCLES

BORING B-4, 40 FEET DEPTH

BERTH 6 AND BULKHEAD WALL EXPANSION

PORT OF PORT ARTHUR

PORT ARTHUR, TEXAS

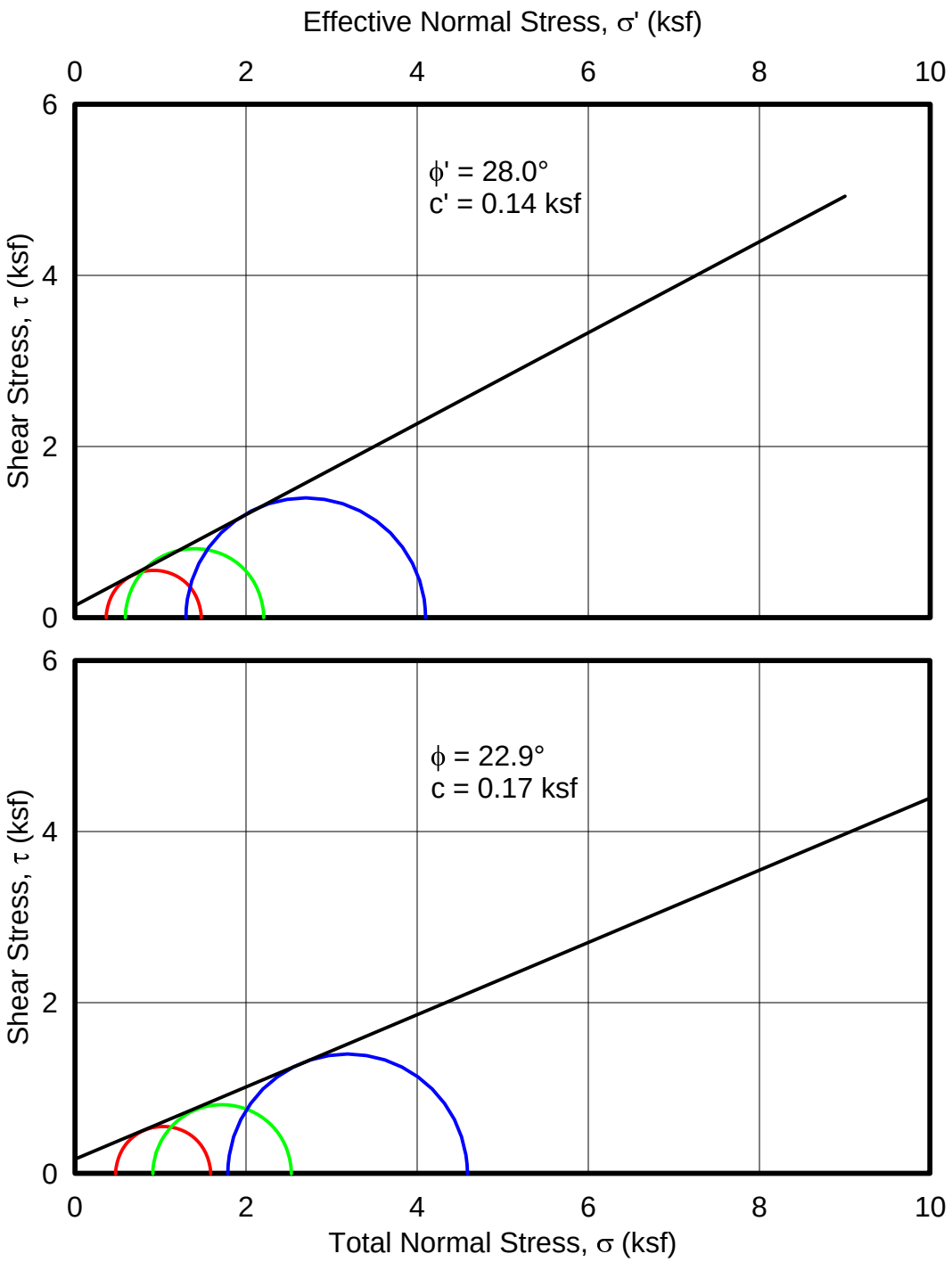


MATERIAL: CLAY, olive gray with silt seams

INITIAL WATER CONTENT: 37%	LIQUID LIMIT: 60
INITIAL WET UNIT WEIGHT: 115 pcf	PLASTIC LIMIT: 17
INITIAL DRY UNIT WEIGHT: 84 pcf	PLASTICITY INDEX: 43
PERCENT PASSING NO. 200 SIEVE: 100	SPECIFIC GRAVITY: 2.75 (assumed)

MULTI STAGE CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS AND STRAIN CURVES
BORING B-4, 40 FEET DEPTH
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS



MATERIAL: CLAY, dark gray, with organic material

INITIAL WATER CONTENT: 26%	LIQUID LIMIT: 47
INITIAL WET UNIT WEIGHT: 122 pcf	PLASTIC LIMIT: 15
INITIAL DRY UNIT WEIGHT: 96 pcf	PLASTICITY INDEX: 32
PERCENT PASSING NO. 200 SIEVE: 94	SPECIFIC GRAVITY: 2.75 (assumed)

MULTI STAGE CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

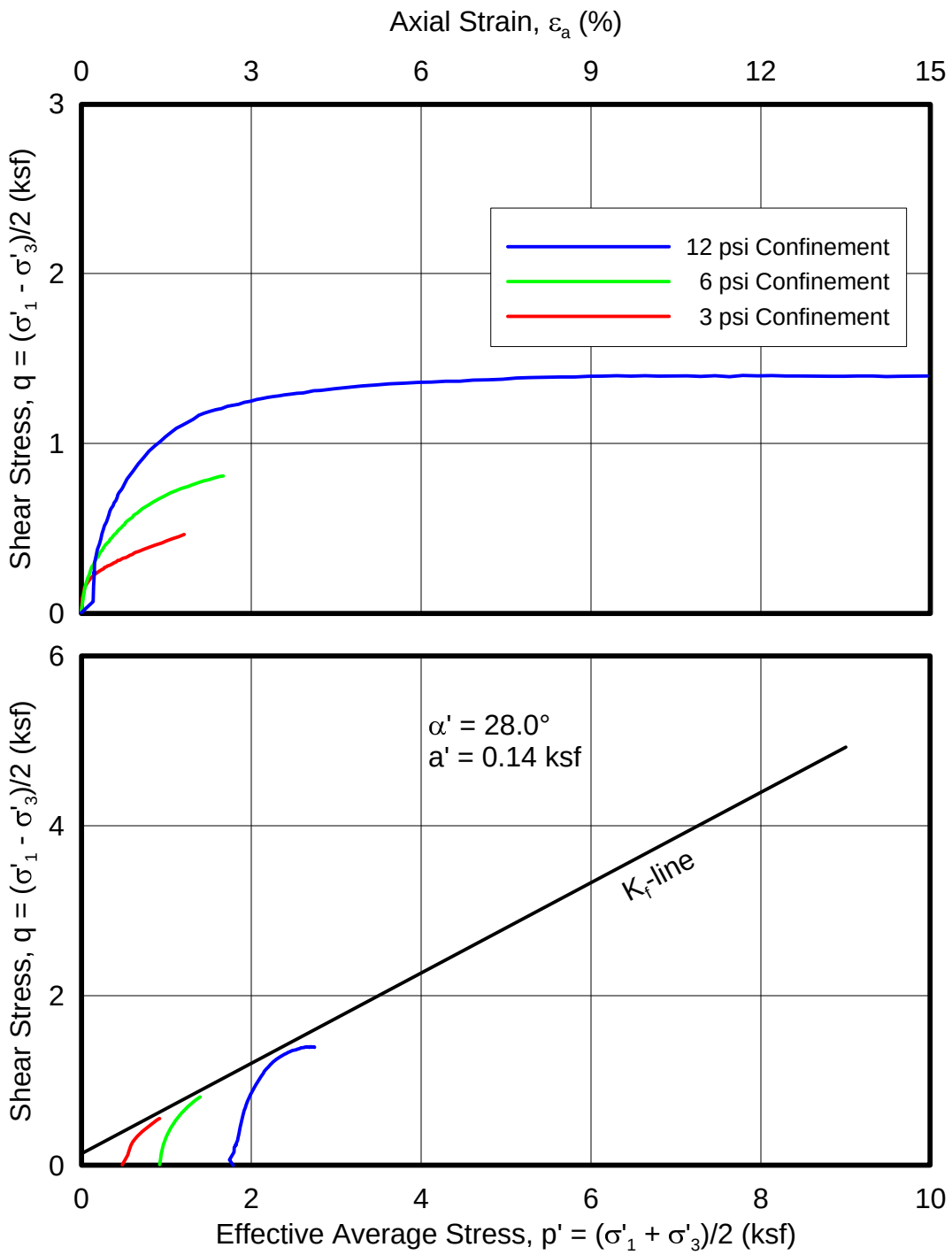
ISOTROPICALLY CONSOLIDATED - MOHR'S CIRCLES

BORING B-5, 8 FEET DEPTH

BERTH 6 AND BULKHEAD WALL EXPANSION

PORT OF PORT ARTHUR

PORT ARTHUR, TEXAS



MATERIAL: CLAY, dark gray, with organic material

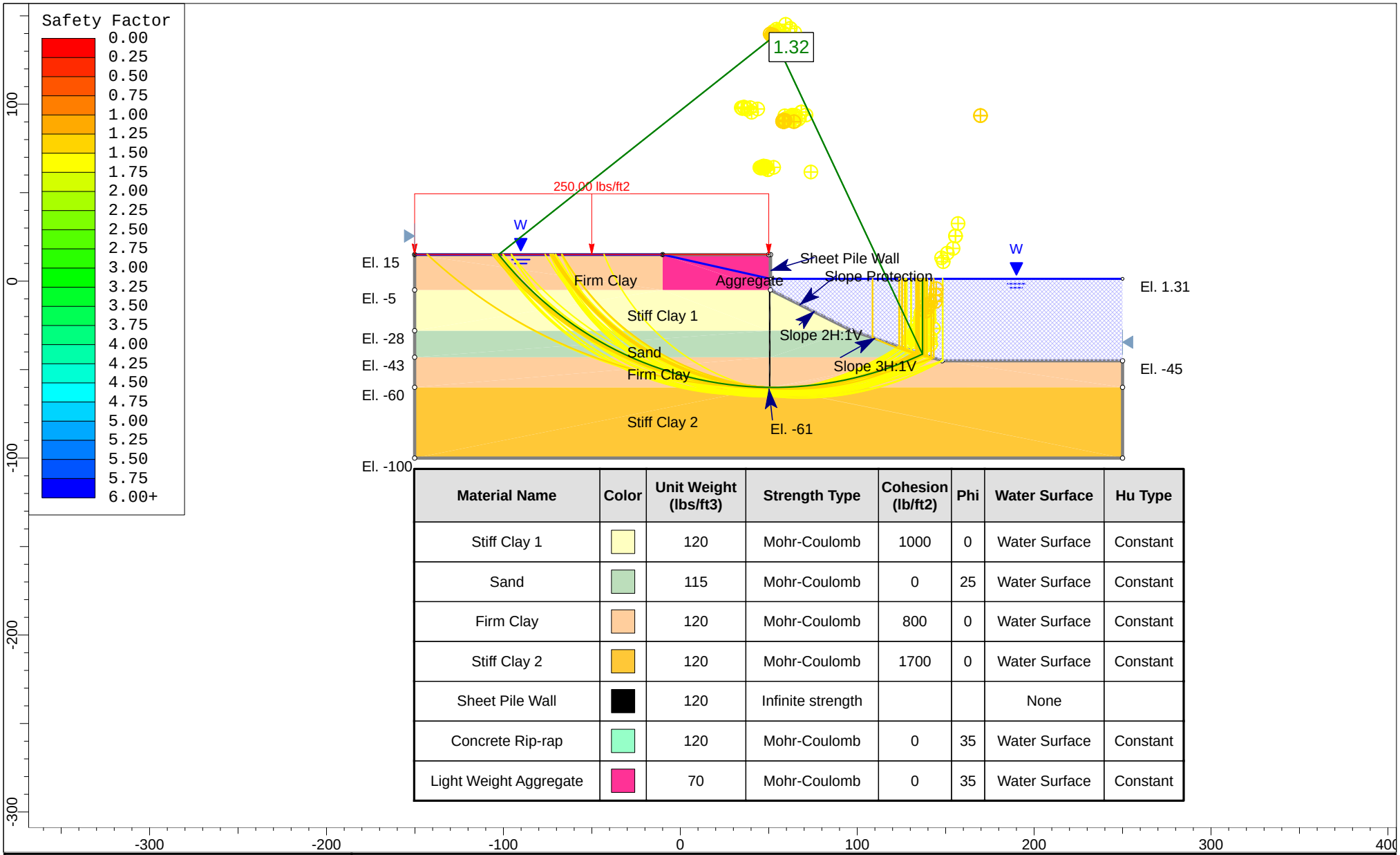
INITIAL WATER CONTENT: 26%	LIQUID LIMIT: 47
INITIAL WET UNIT WEIGHT: 122 pcf	PLASTIC LIMIT: 15
INITIAL DRY UNIT WEIGHT: 96 pcf	PLASTICITY INDEX: 32
PERCENT PASSING NO. 200 SIEVE: 94	SPECIFIC GRAVITY: 2.75 (assumed)

MULTI STAGE CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS AND STRAIN CURVES
BORING B-5, 8 FEET DEPTH
BERTH 6 AND BULKHEAD WALL EXPANSION
PORT OF PORT ARTHUR
PORT ARTHUR, TEXAS

APPENDIX D

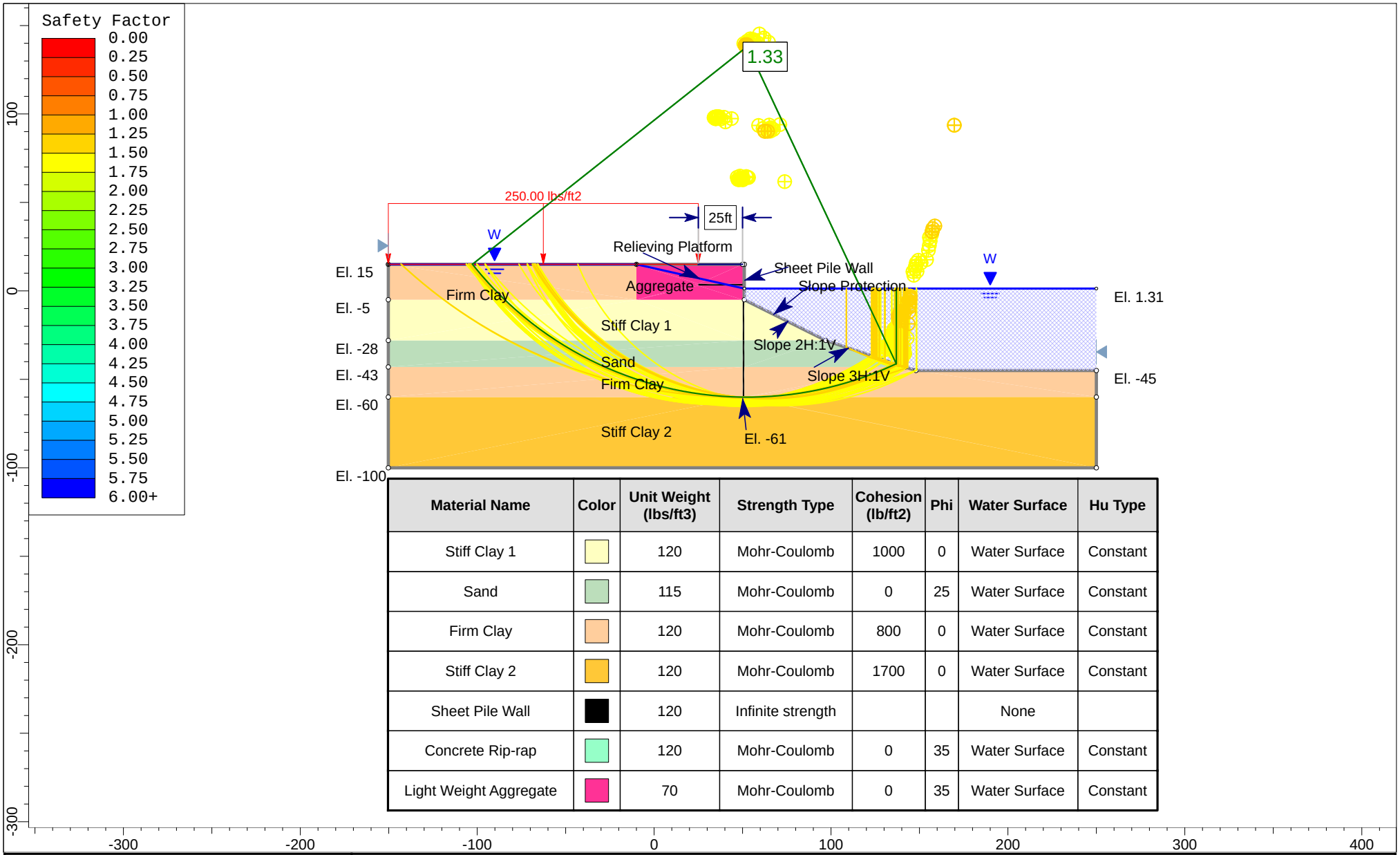
GLOBAL SLOPE STABILITY ANALYSIS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

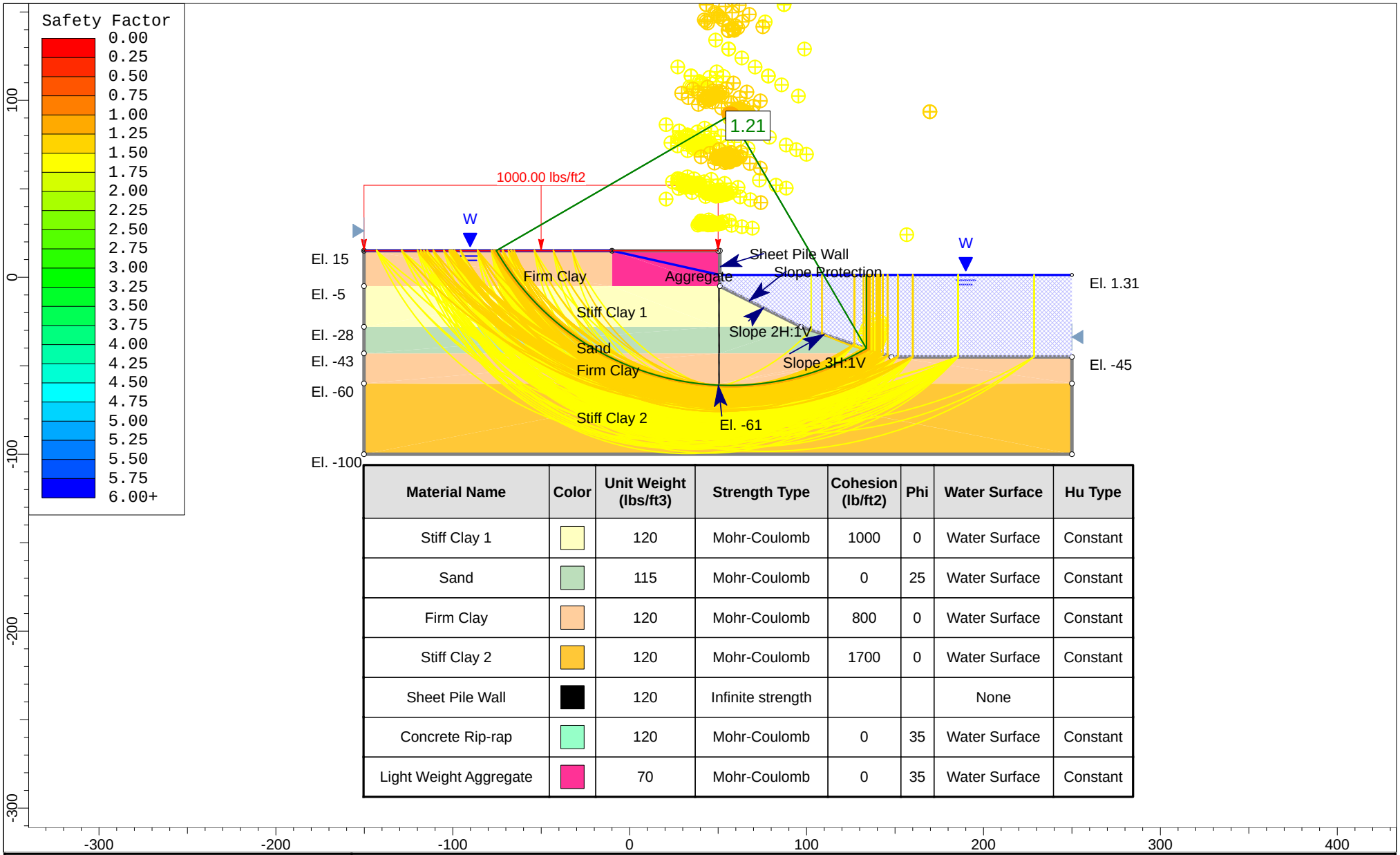
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Short Term - 250 psf Surcharge - No Platform (El. -61).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

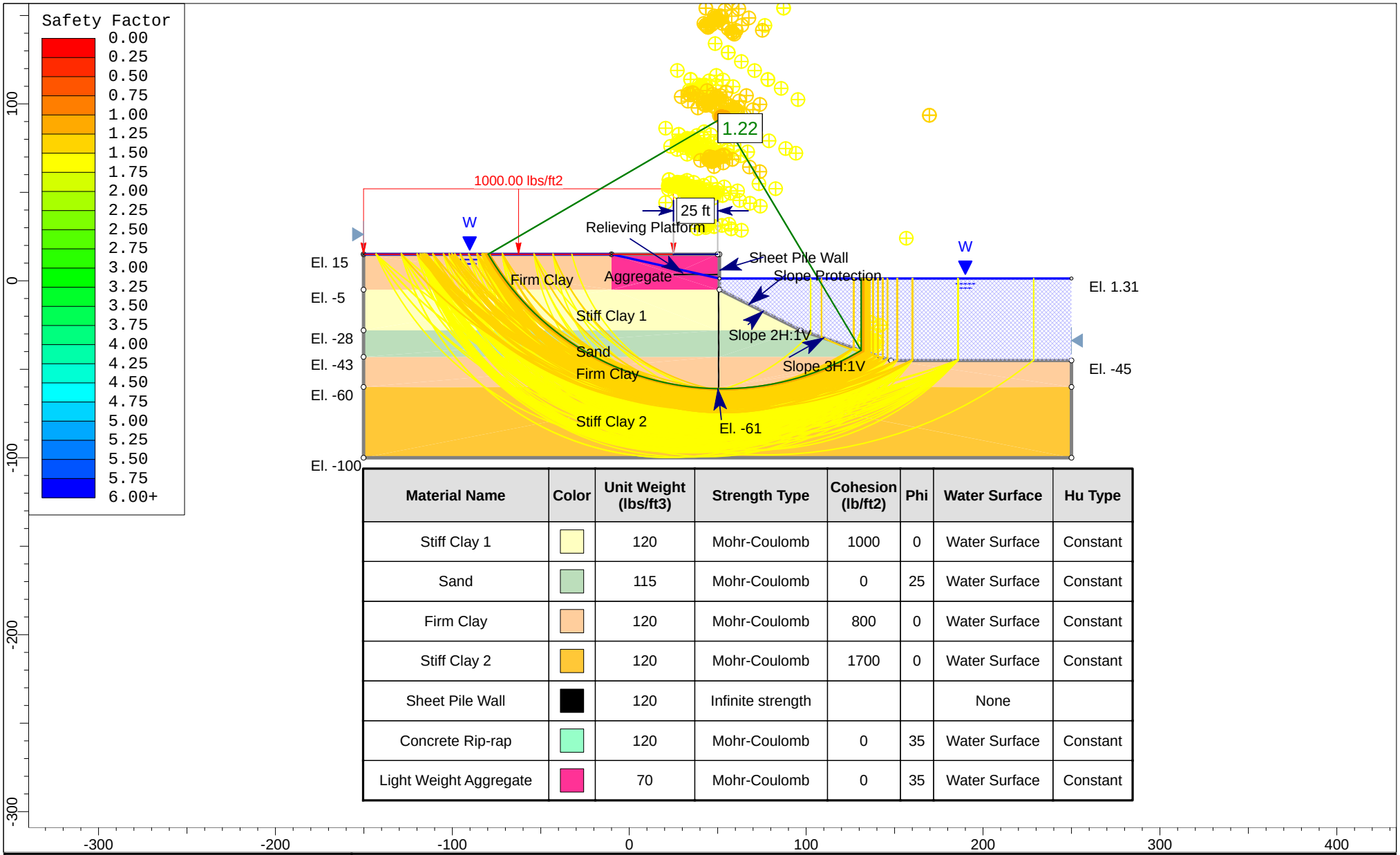
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Short Term - 250 psf Surcharge - With Platform (El. -61).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

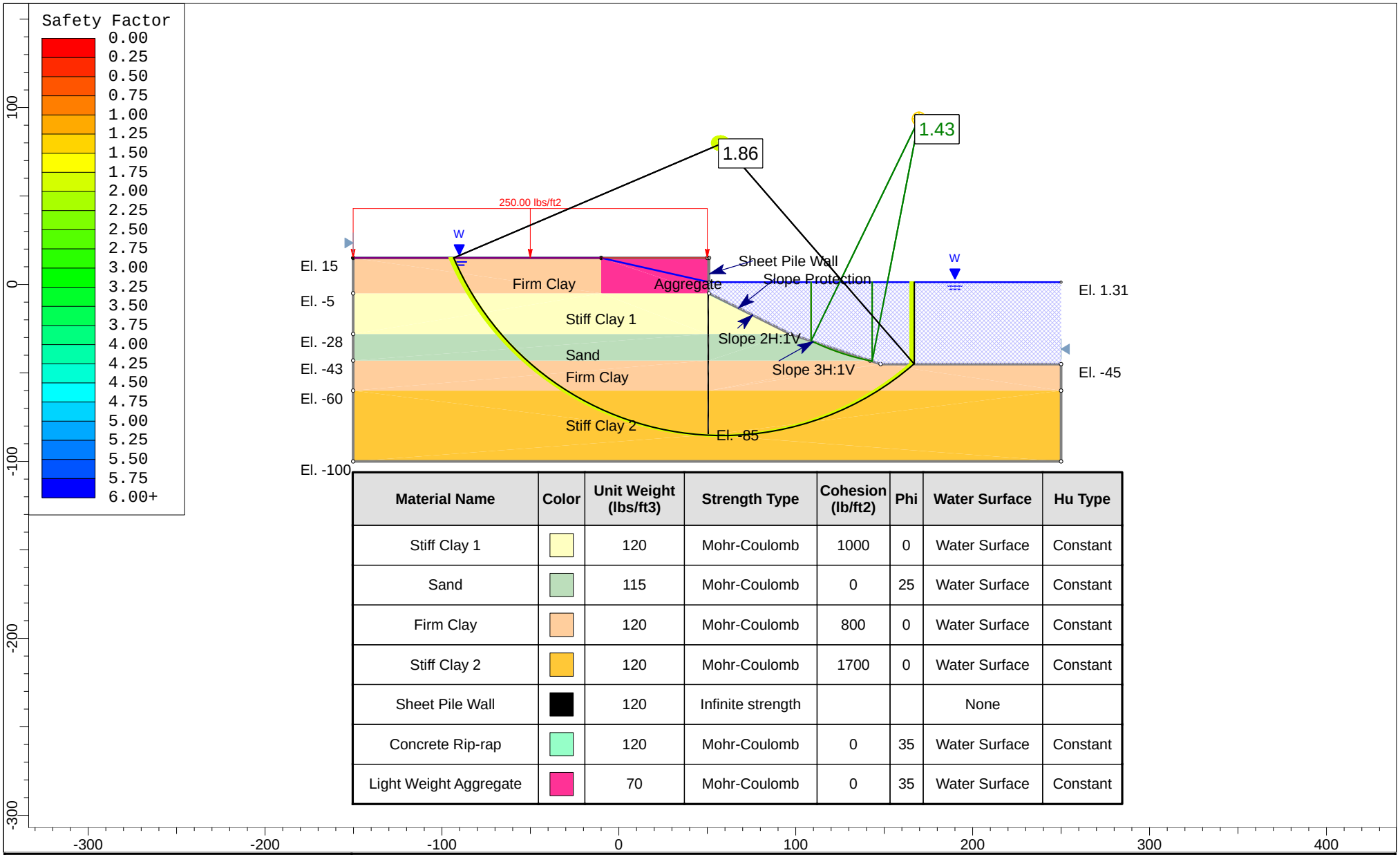
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Short Term - 1000 psf Surcharge - No Platform (El. -61).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

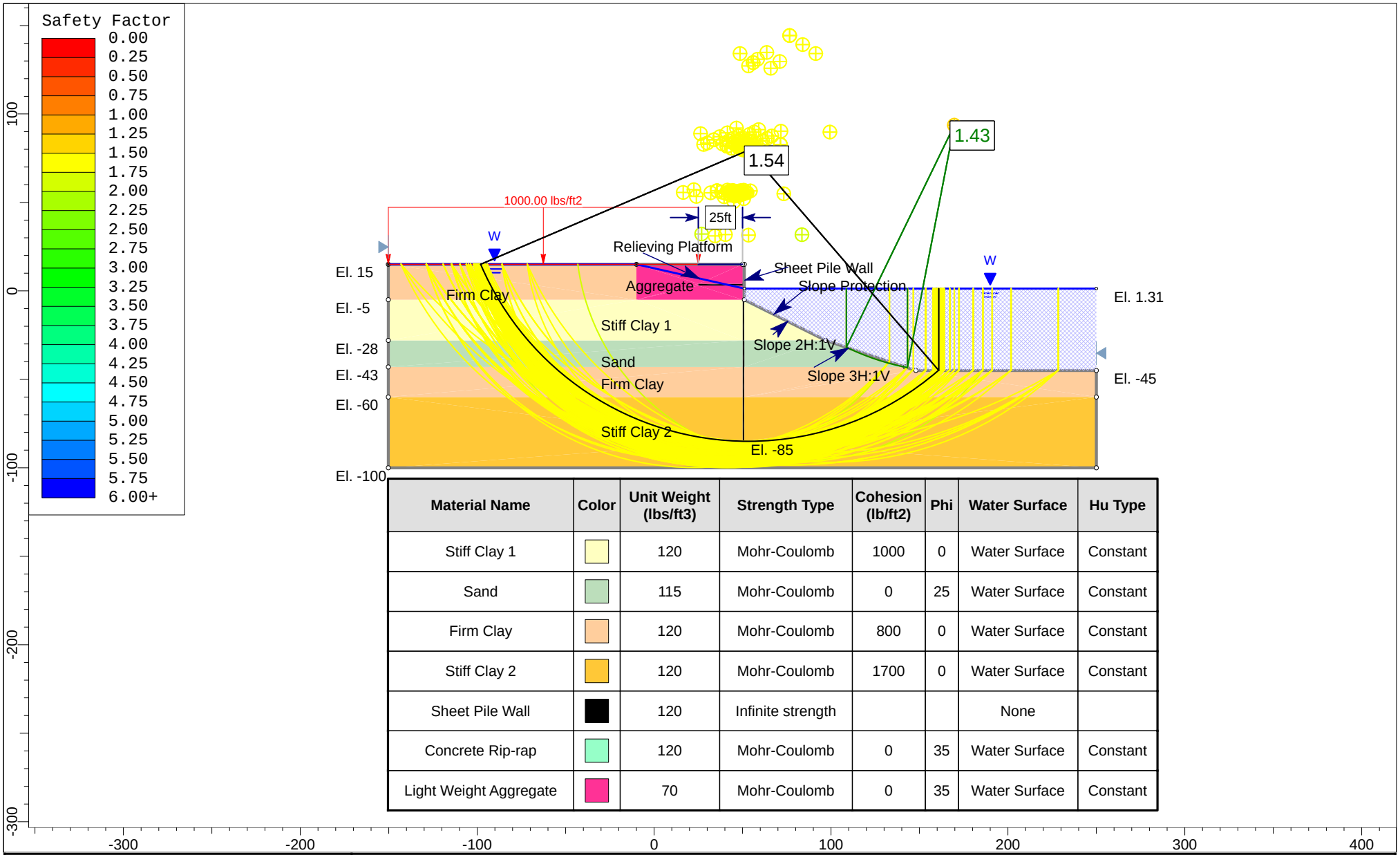
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Short Term - 1000 psf Surcharge - With Platform (El. -61).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

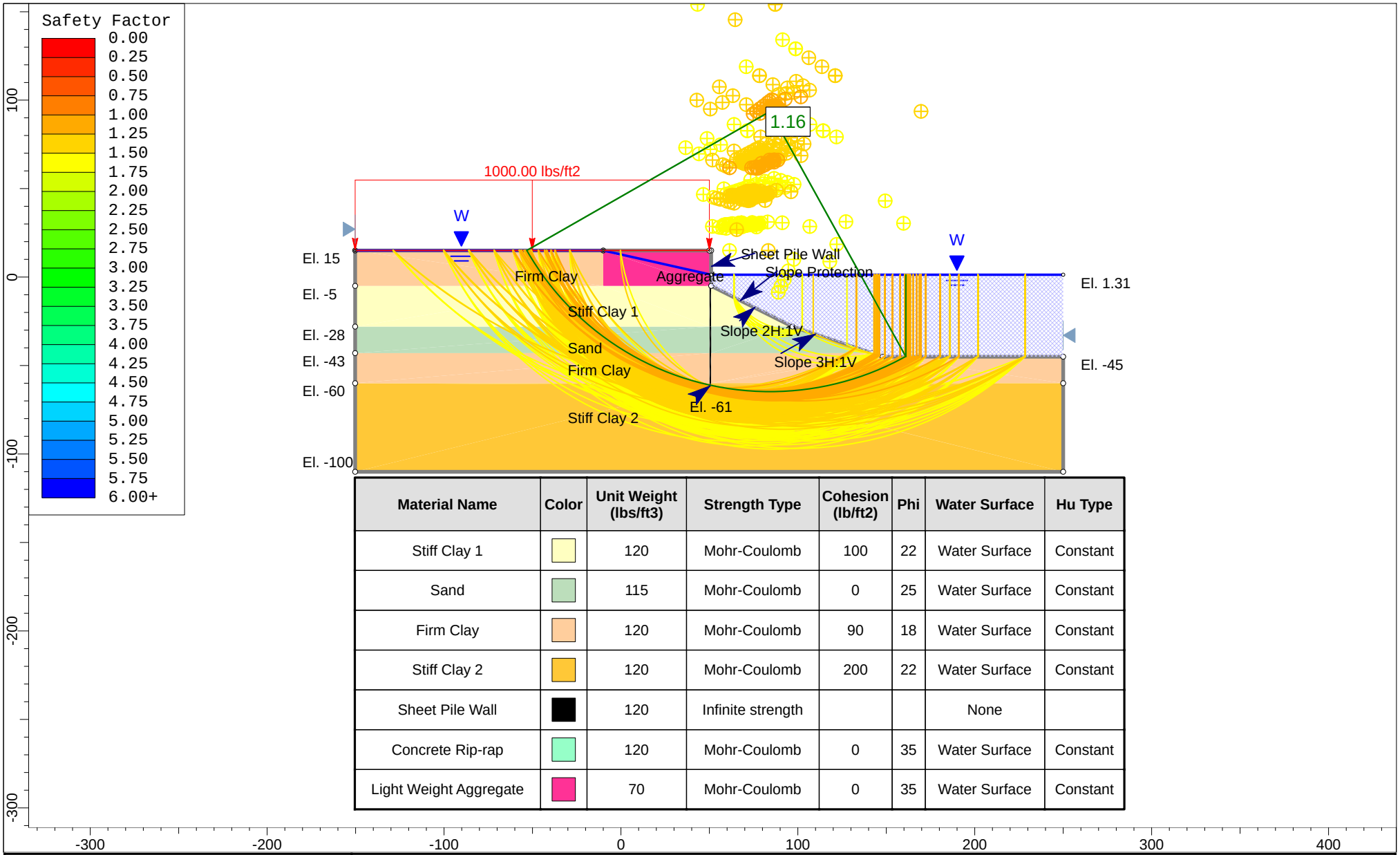
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Short Term - 250 psf Surcharge - No Platform (El. -85).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Short Term - 1000 psf Surcharge - With Platform (El. -85).slim

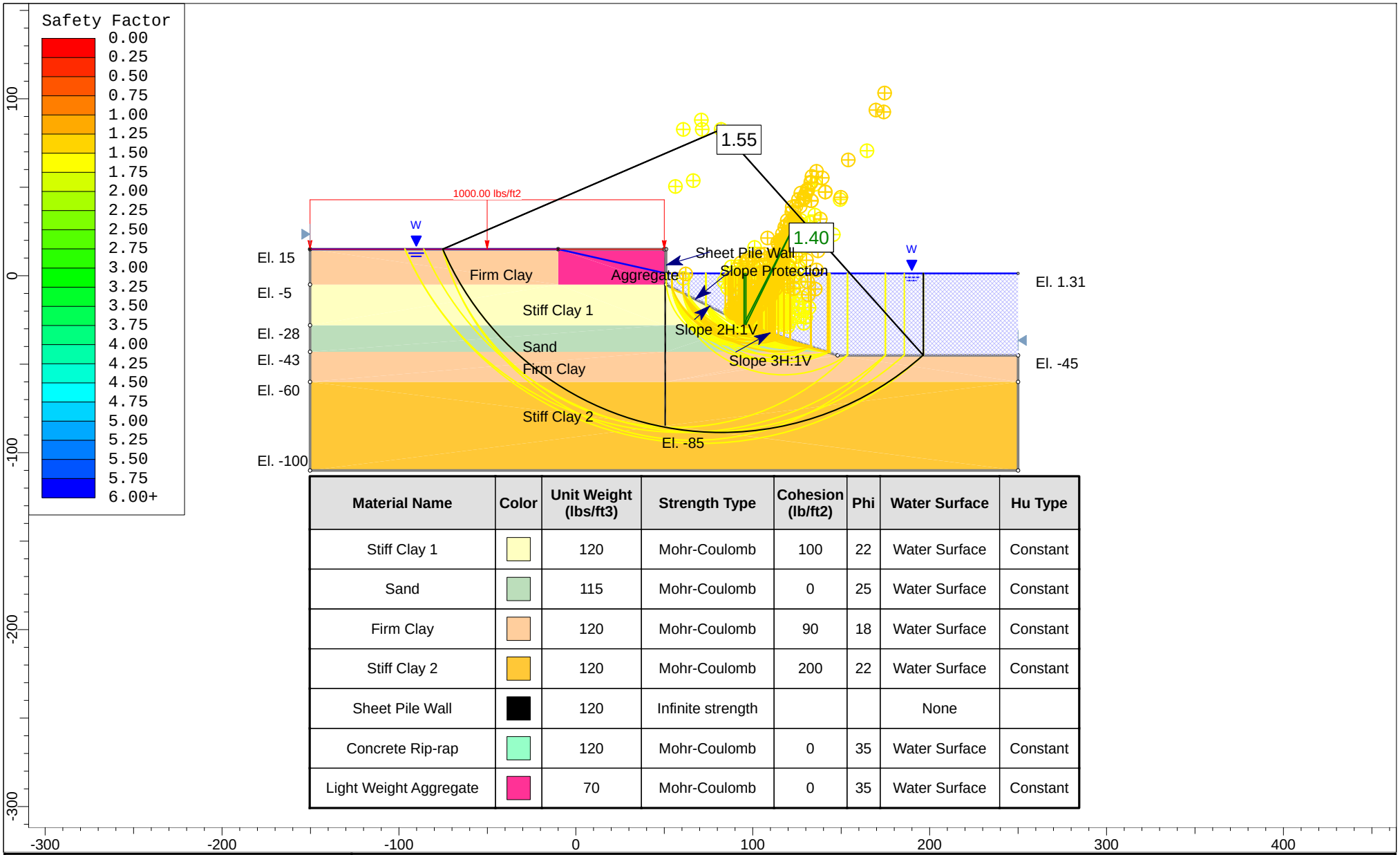


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	100	22	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	90	18	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	200	22	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

FUGRO

SLIDEINTERPRET 6.008

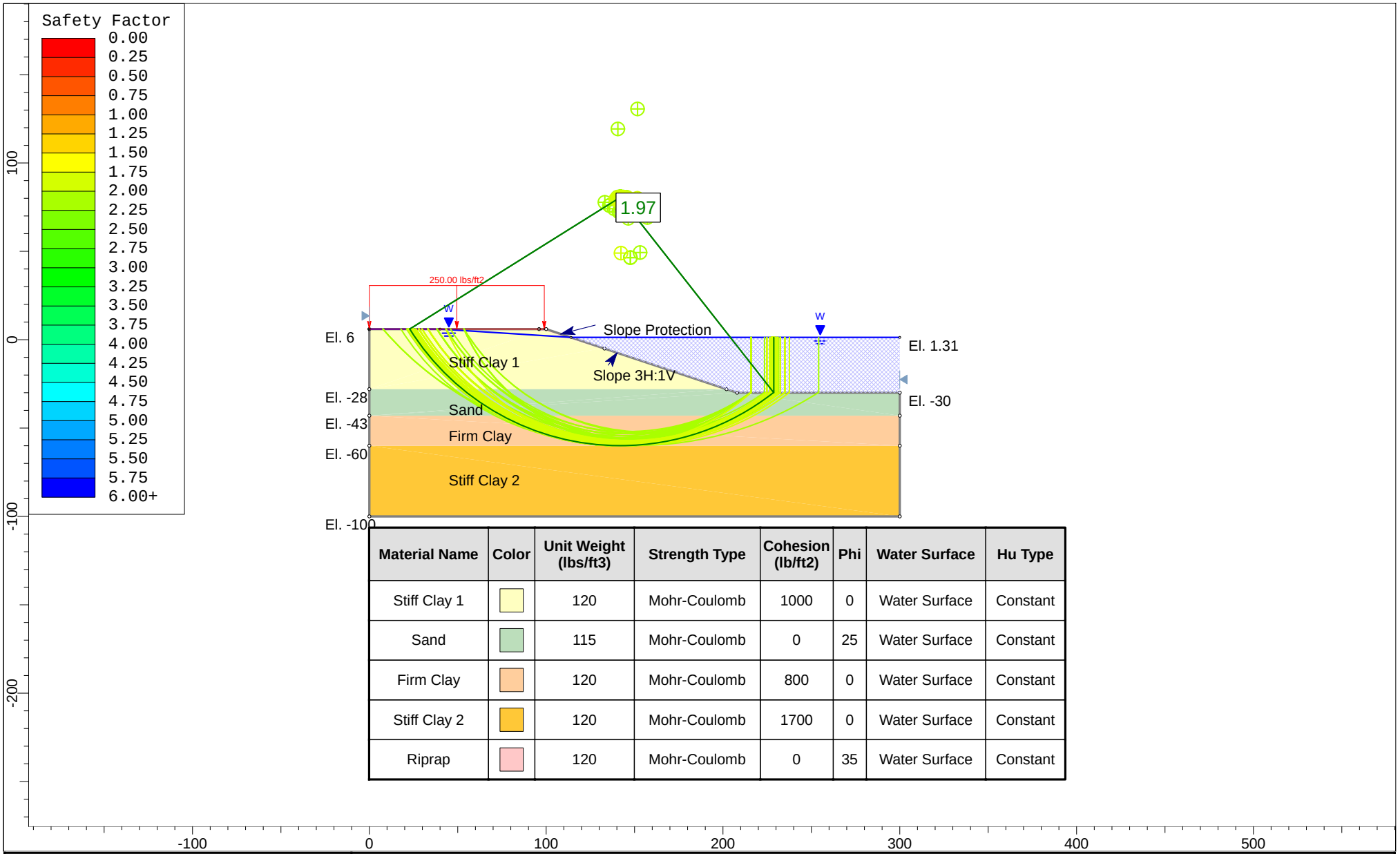
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Long Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Long Term - 1000 psf Surcharge - No Platform (El. -61).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	100	22	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	90	18	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	200	22	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Long Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Long Term - 1000 psf Surcharge - No Platform (El. -85).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Riprap		120	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

Project	Berth 6 and Bulkhead Wall Extension
Analysis Description	Global Stability - Short Term - Cross Section 3
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/23/2013, 4:46:16 PM
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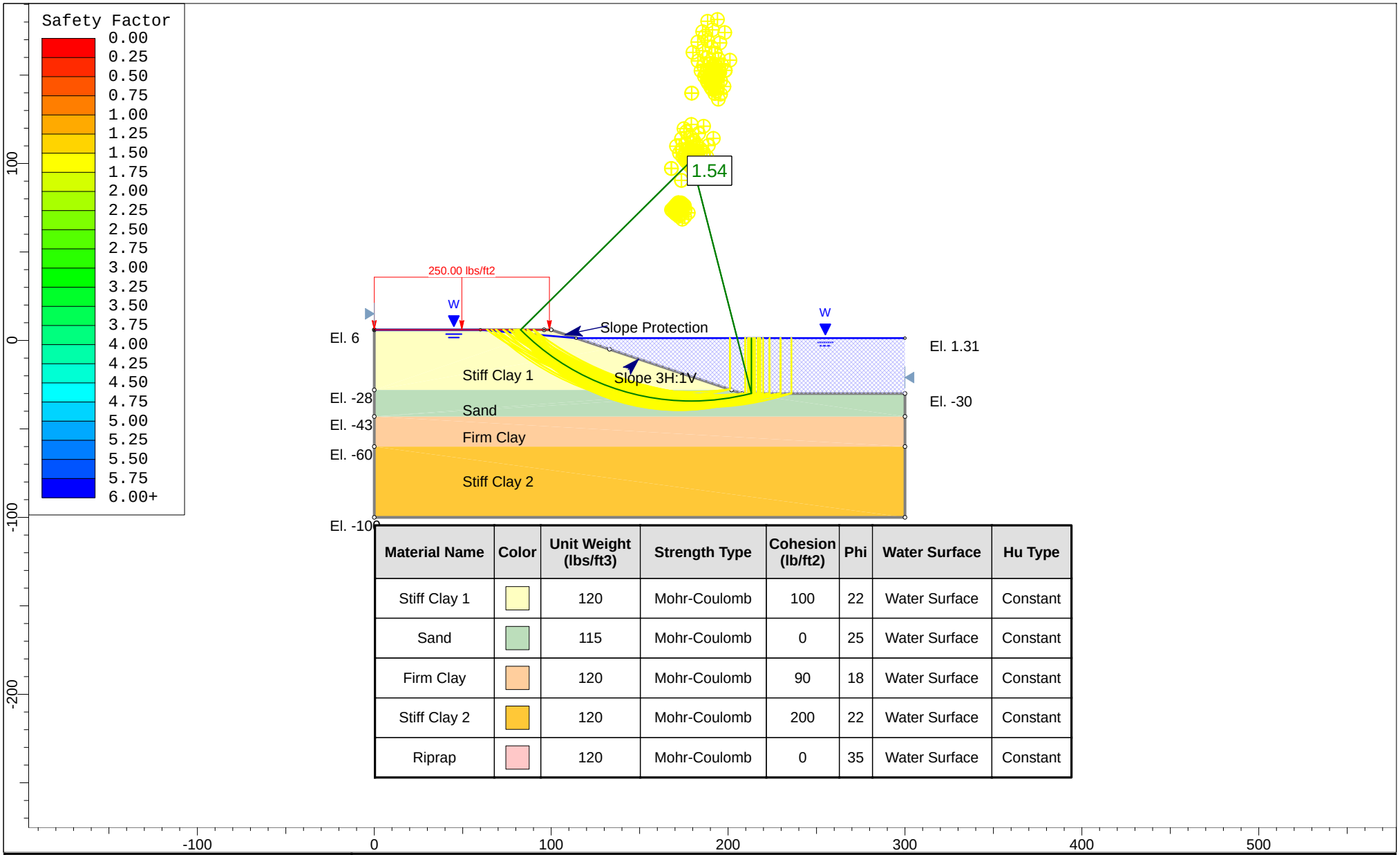
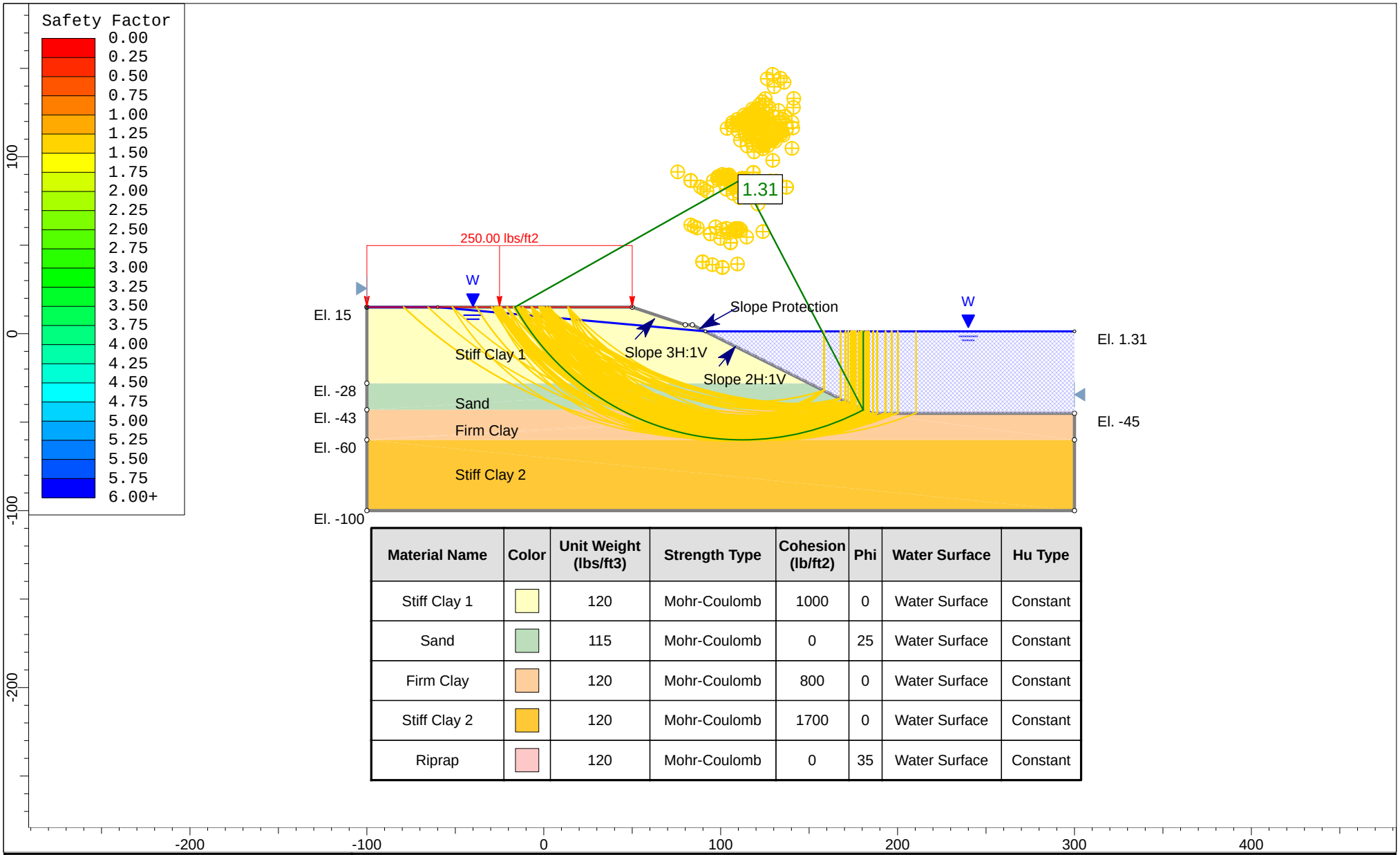
File Name	Short Term - 250 psf Surcharge.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	100	22	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	90	18	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	200	22	Water Surface	Constant
Riprap		120	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

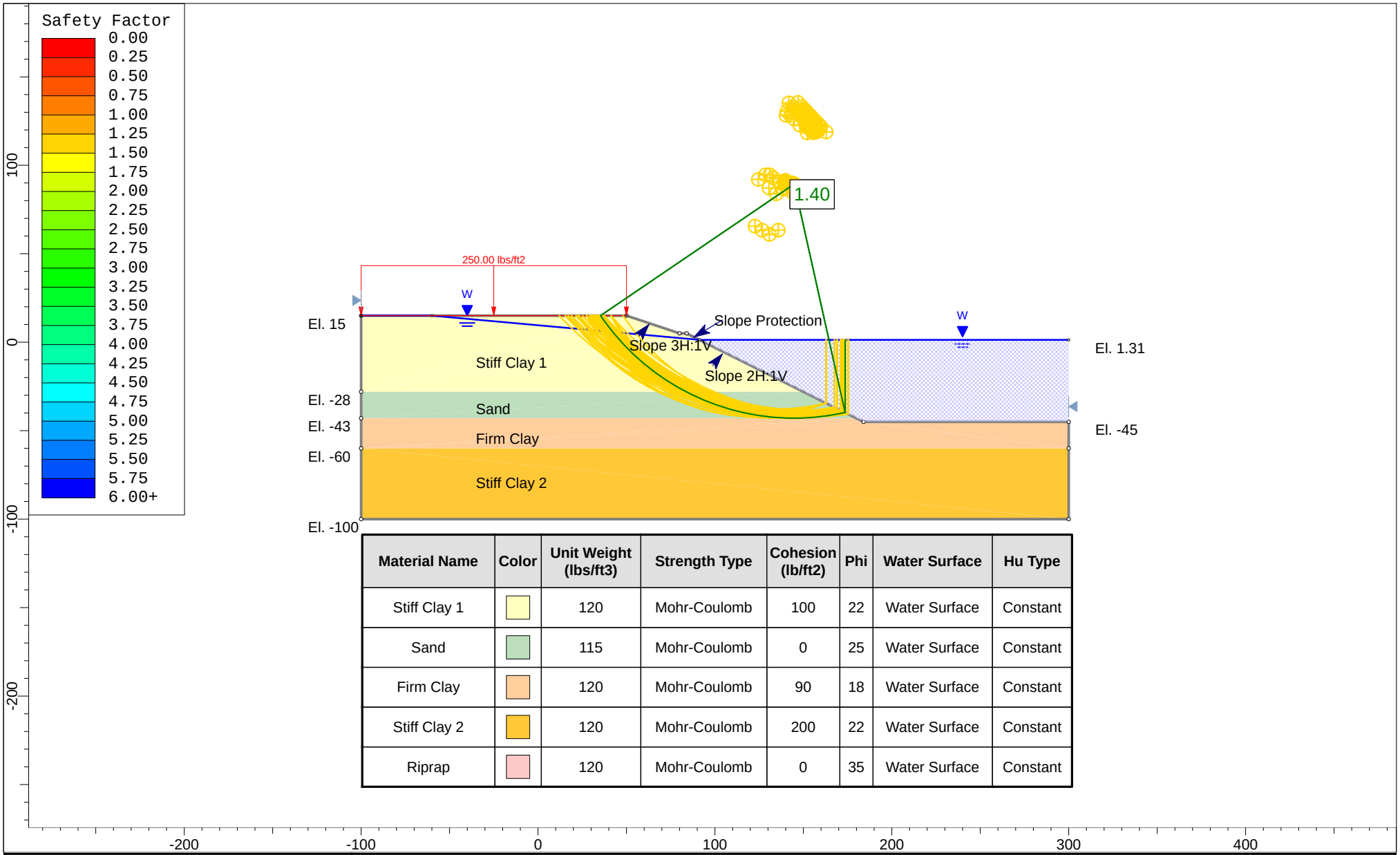
Project	Berth 6 and Bulkhead Wall Extension		
Analysis Description	Global Stability - Long Term - Cross Section 3		
Drawn By	S. Vedantam	*Company*	Fugro Consultants, Inc.
Date 1/23/2013, 4:46:16 PM	*Scale* 1:900	*File Name*	Long Term - 250 psf Surcharge.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Riprap		120	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

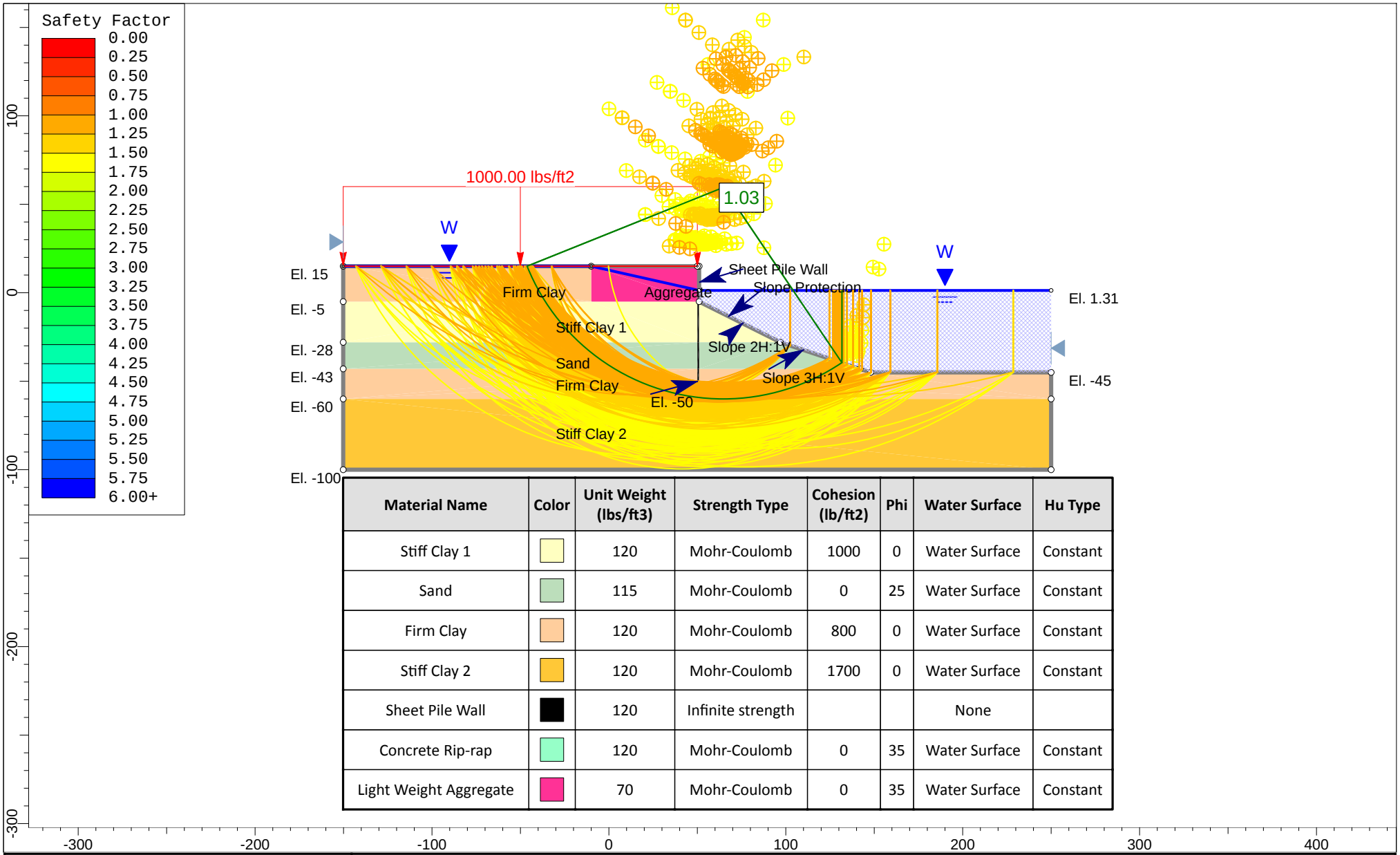
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Cross Section 4
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/23/2013, 3:34:05 PM
Scale	1:900
File Name	Short Term - 250 psf Surcharge.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	100	22	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	90	18	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	200	22	Water Surface	Constant
Riprap		120	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

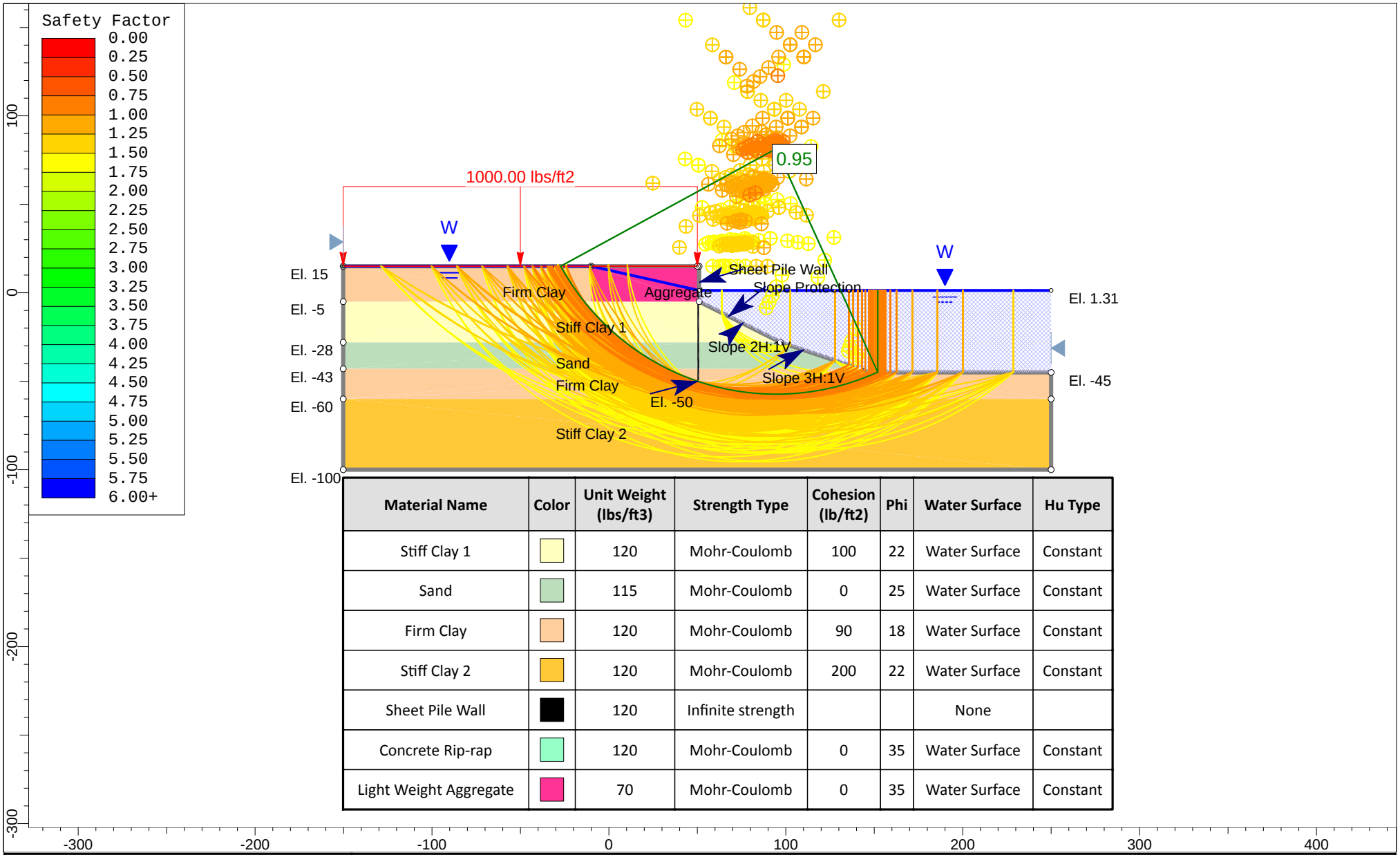
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Long Term - Cross Section 4
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/23/2013, 3:34:05 PM
Scale	1:900
File Name	Long Term - 250 psf Surcharge.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Short Term - 1000 psf Surcharge - No Platform (El. -50).slim

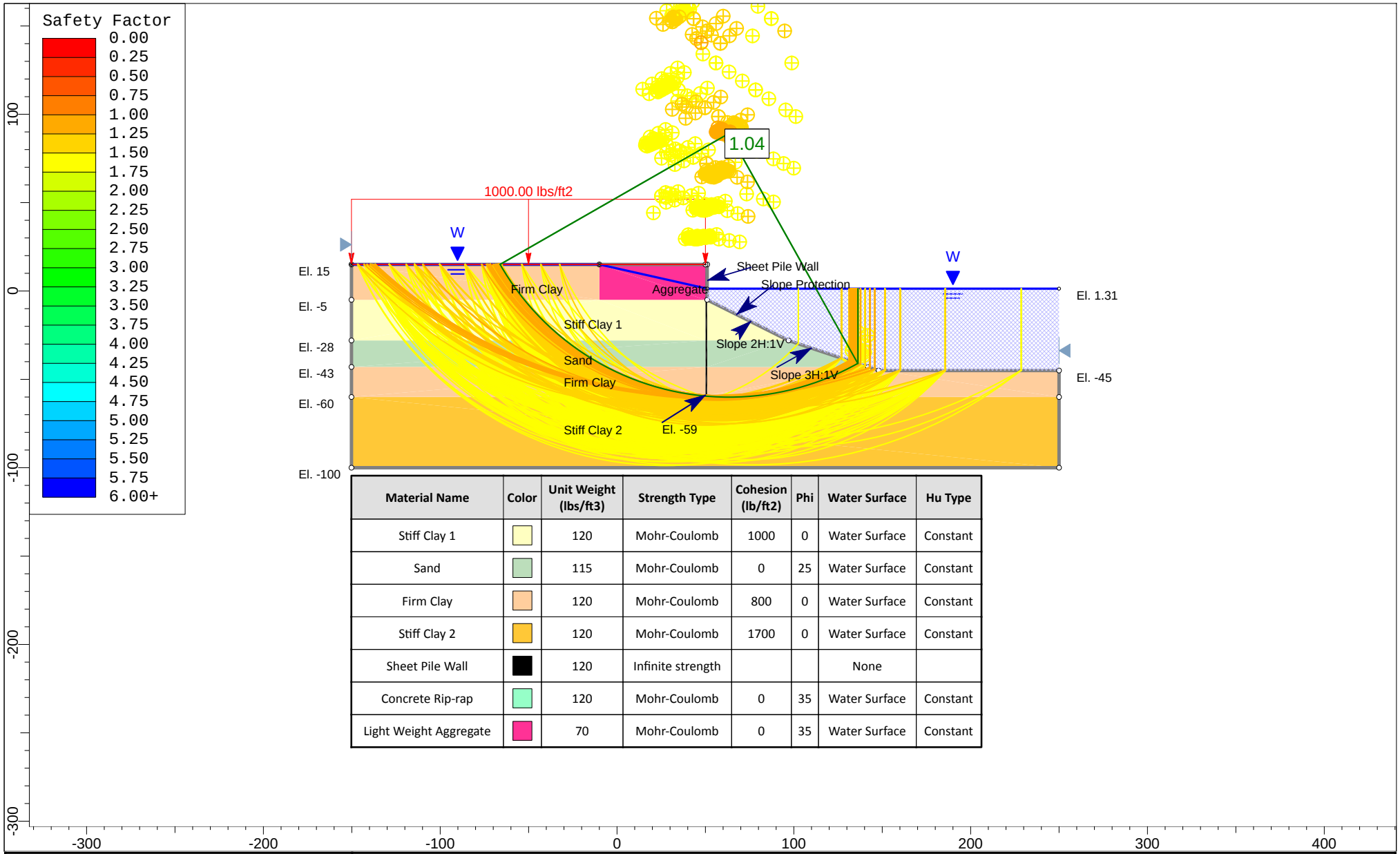


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	100	22	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	90	18	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	200	22	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

FUGRO

SLIDEINTERPRET 6.008

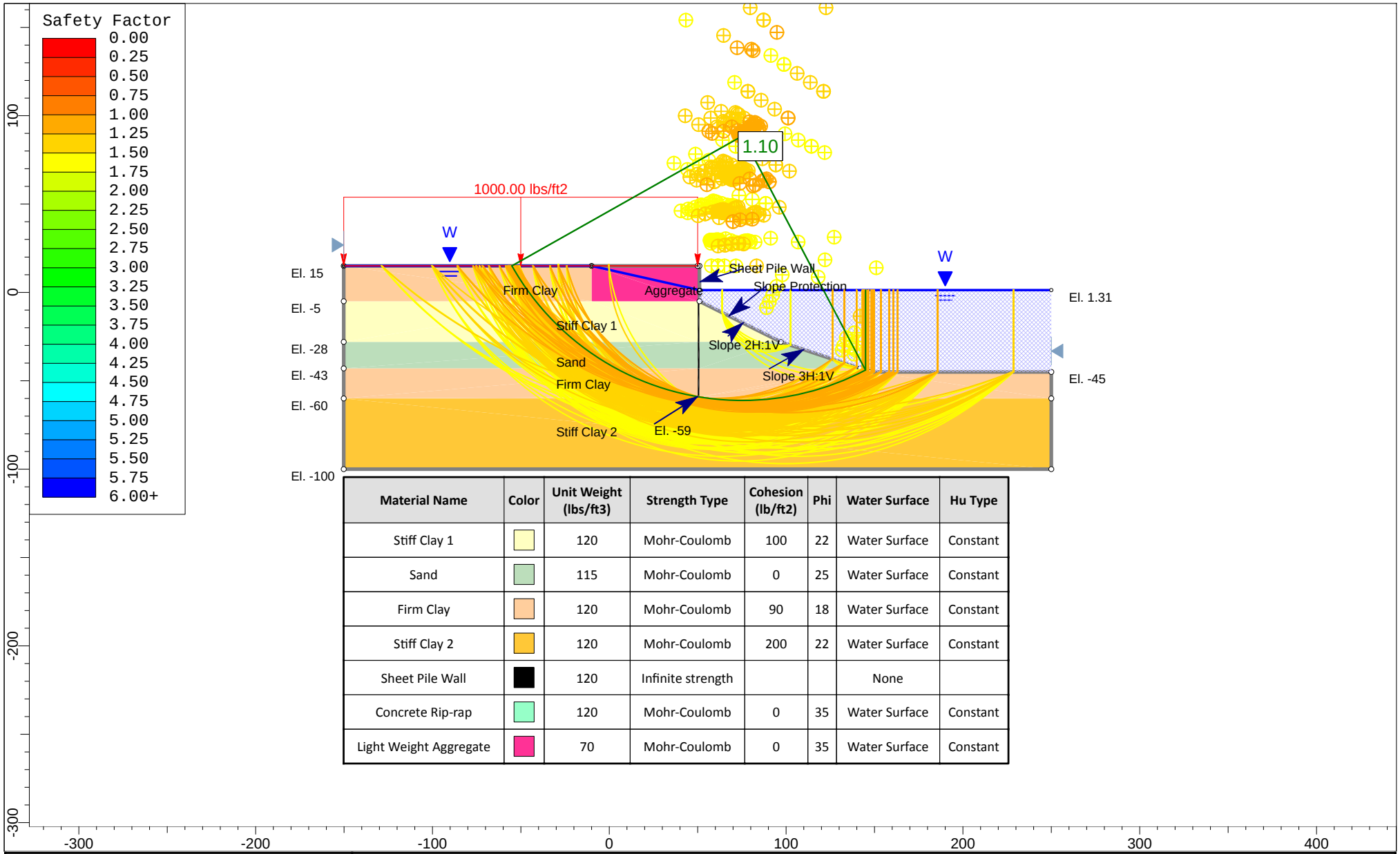
Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Long Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Long Term - 1000 psf Surcharge - No Platform (El. -50).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	1000	0	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	800	0	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	1700	0	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

SLIDEINTERPRET 6.008

Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Short Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
Scale	1:900
File Name	Short Term - 1000 psf Surcharge - No Platform (El. -59).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Stiff Clay 1		120	Mohr-Coulomb	100	22	Water Surface	Constant
Sand		115	Mohr-Coulomb	0	25	Water Surface	Constant
Firm Clay		120	Mohr-Coulomb	90	18	Water Surface	Constant
Stiff Clay 2		120	Mohr-Coulomb	200	22	Water Surface	Constant
Sheet Pile Wall		120	Infinite strength			None	
Concrete Rip-rap		120	Mohr-Coulomb	0	35	Water Surface	Constant
Light Weight Aggregate		70	Mohr-Coulomb	0	35	Water Surface	Constant

FUGRO

SLIDEINTERPRET 6.008

Project	Berth 6 and Bulkhead Wall Expansion
Analysis Description	Global Stability - Long Term - Bulkhead (Revised Slope) - Cross Section 7
Drawn By	S. Vedantam
Company	Fugro Consultants, Inc.
Date	1/24/2013, 3:49:04 PM
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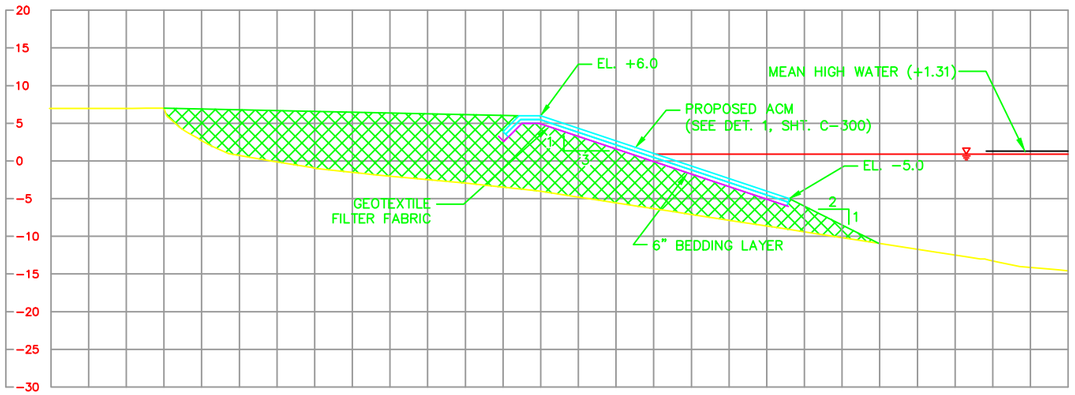
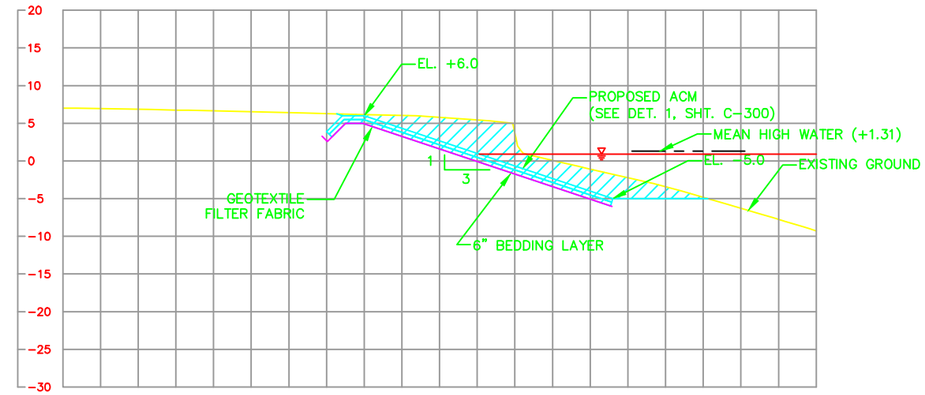
File Name	Long Term - 1000 psf Surcharge - No Platform (El. -59).slim

APPENDIX E

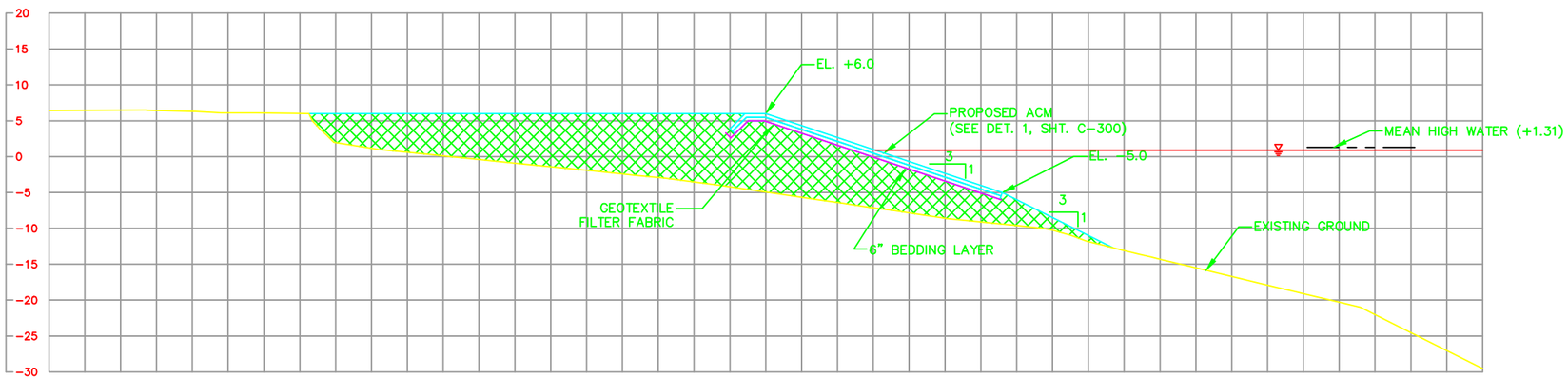
DESIGN DRAWINGS – SLOPE STABILITY ANALYSIS



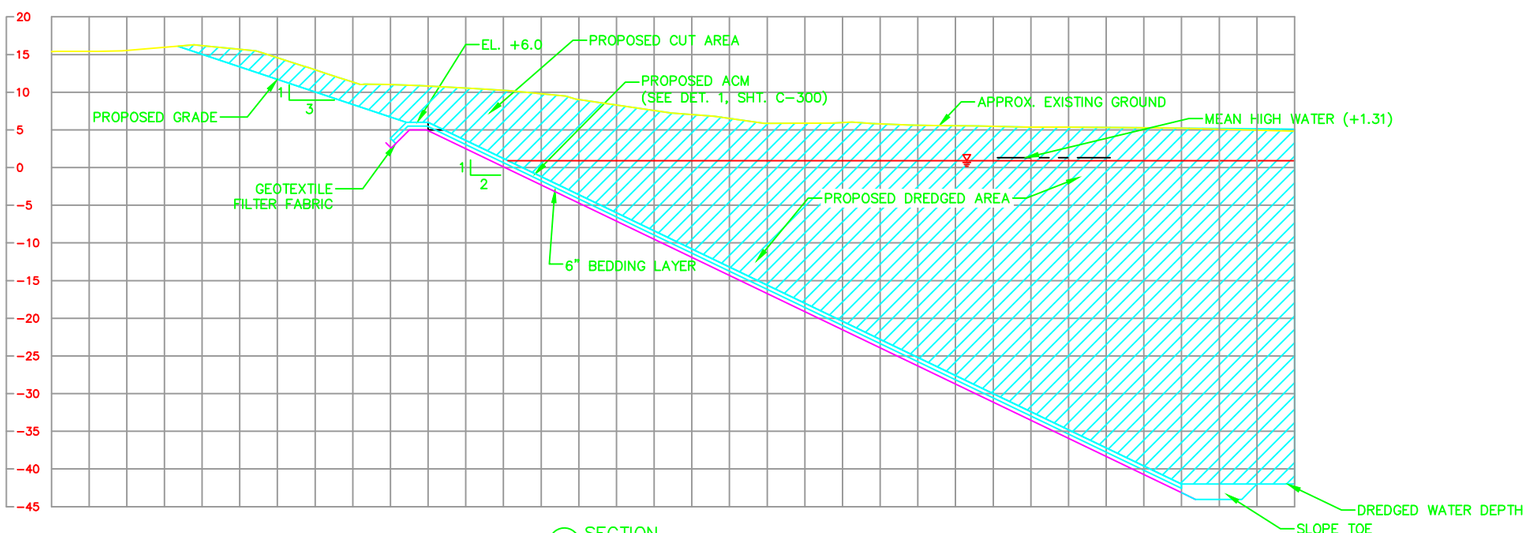
1 SECTION
SCALE 1"=10'
SEE PLAN VIEW SHEET C-104



2 SECTION
SCALE 1"=10'
SEE PLAN VIEW SHEET C-103



3 SECTION
SCALE 1"=10'
SEE PLAN VIEW SHEET C-103



4 SECTION
SCALE 1"=10'
SEE PLAN VIEW SHEET C-103

LEGEND

PROPOSED FILL

PROPOSED CUT

SHORELINE STABILIZATION & WHARF DESIGN

PORT OF PORT ARTHUR

Port Arthur, Texas

Lockwood, Andrews & Newnam, Inc.
A LEO A DALY COMPANY
PLANNING
ENGINEERING
PROGRAM MANAGEMENT

2925 Briarpark Drive
Houston, Texas 77042-3720
Tel 713-266-6900 Fax
713-266-7596

PORT ARTHUR
INTERNATIONAL
PUBLIC PORT
PORT OF PORT ARTHUR
NAVIGATION DISTRICT OF
JEFFERSON COUNTY, TEXAS

REVISIONS

NO.	DESCRIPTION	DATE

FILE LOG

Manager	ETT
Design	
Draw	
Check	

STAMP

Project No. 120-10849-001
08/06/2012
NOT TO SCALE

ACM CROSS SECTIONS

C-200

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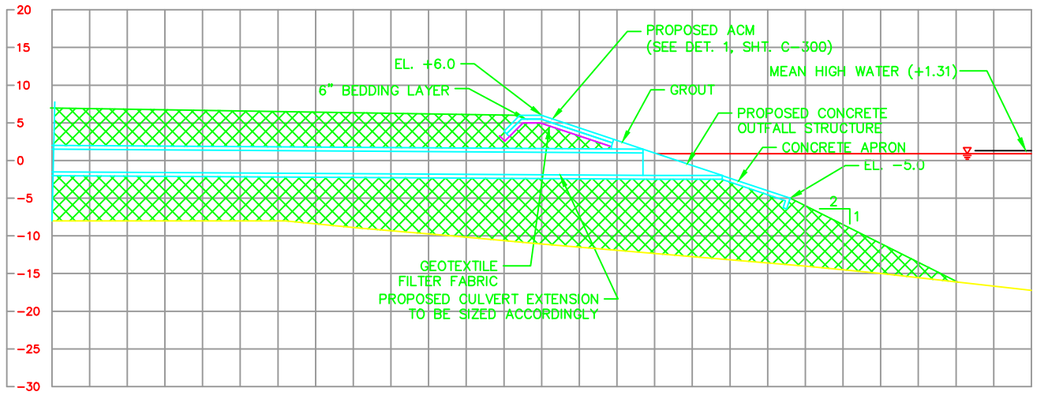
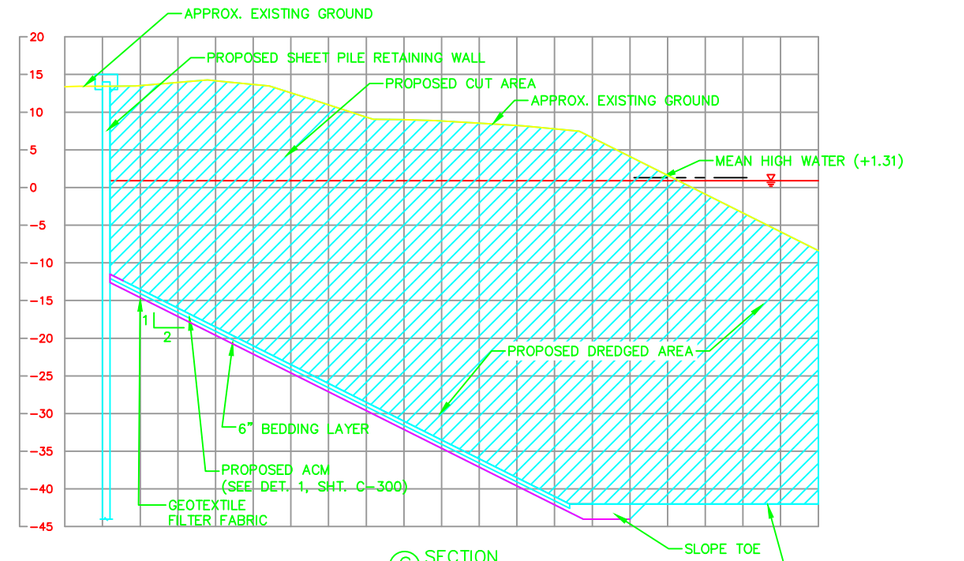
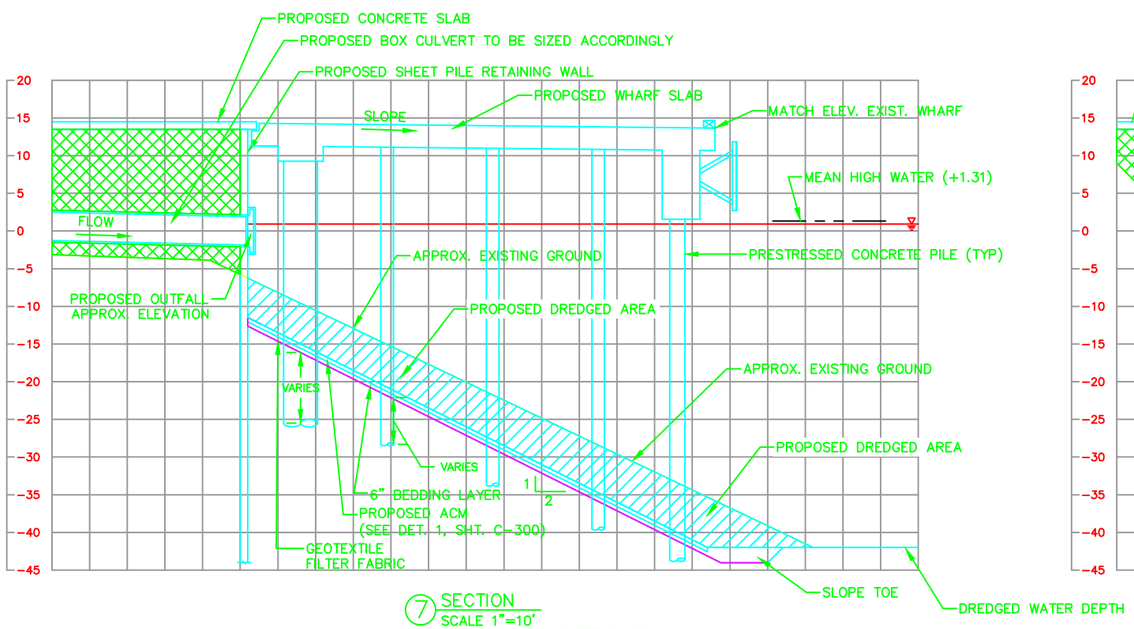


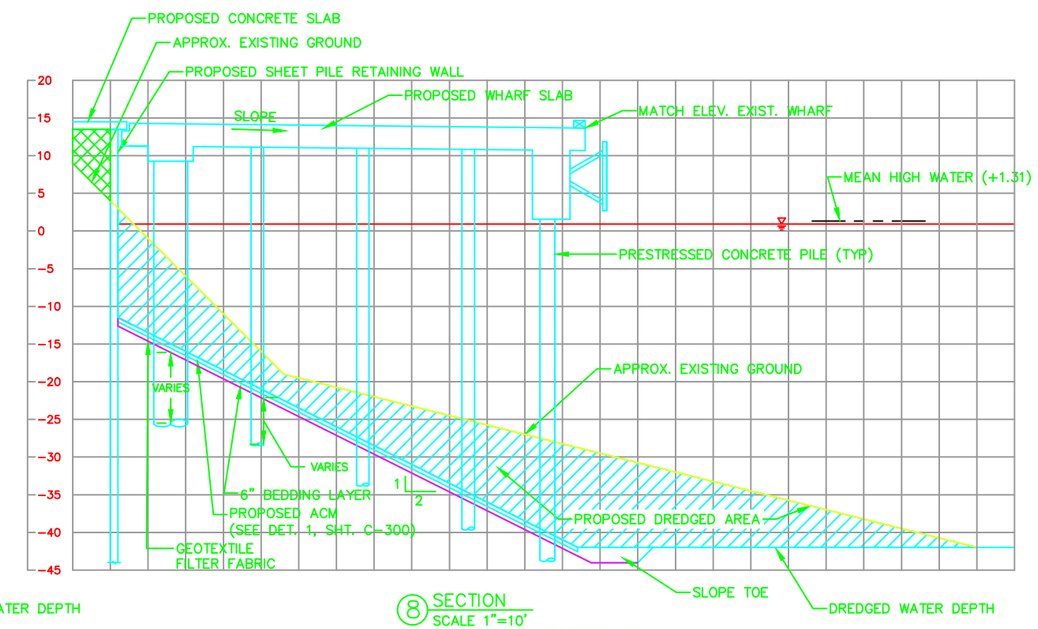
5 SECTION
SCALE 1"=10'
SEE PLAN VIEW SHEET C-104



6 SECTION
SCALE 1"=10'
SEE PLAN VIEW SHEET C-102



7 SECTION
SCALE 1"=10'
SEE PLAN VIEW SHEET C-102



8 SECTION
SCALE 1"=10'
SEE PLAN VIEW SHEET C-101

NOTES:
1. CONTRACTOR TO COORDINATE WITH PORT OF PORT ARTHUR OPERATIONS WHEN WORKING NEAR RAIL BRIDGE OR OTHER POPA FACILITIES SO AS NOT TO INTERRUPT ANY ONGOING OPERATIONS.
2. CONTRACTOR TO RESTORE BRIDGE TO ORIGINAL CONDITION OR BETTER IF DAMAGED.

LEGEND
PROPOSED FILL
PROPOSED CUT

SHORELINE STABILIZATION & WHARF DESIGN

PORT OF PORT ARTHUR

Port Arthur, Texas

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713-266-7596

PORT ARTHUR INTERNATIONAL PUBLIC PORT
PORT OF PORT ARTHUR NAVIGATION DISTRICT OF JEFFERSON COUNTY, TEXAS

REVISIONS

NO.	DESCRIPTION	DATE

FILE LOG

Manager	ETT
Design	
Draw	
Check	

STAMP

Project No. 120-10849-001
08/06/2012
NOT TO SCALE

ACM CROSS SECTIONS

C-201

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