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December 22, 2010

Baker & Lawson, Inc.
300 E. Cedar Street
Angleton, Texas 77515

Attn: Mr. Doug Roesler
Ph: (979) 849-6681

Re: Geotechnical Exploration Report
Proposed Shore Stabilization and Pedestrian Bridge
Surfside, Texas
PSI Report No. 286-373

Dear Mr. Roesler,

Professional Service Industries, Inc. is pleased to transmit our Geotechnical Exploration Report for the referenced project. This report includes the results of field and laboratory testing, and recommendations for the proposed Shore Stabilization and Pedestrian Bridge project in Surfside, Texas.

We appreciate the opportunity to perform this Geotechnical Study and look forward for continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office.

Respectfully submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

Emily Chemadurov, P.E.
Project Manager

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(3) Copies to Client

GEOTECHNICAL EXPLORATION REPORT

PROPOSED SHORE STABILIZATION AND PEDESTRIAN BRIDGE
SURFSIDE, TEXAS
PSI REPORT NO. 286-373

PREPARED FOR

Baker & Lawson, Inc.
300 E. Cedar Street
Angleton, Texas 77515

DECEMBER 22, 2010

BY

PROFESSIONAL SERVICE INDUSTRIES, INC.
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PROJECT INFORMATION

PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI) has completed a geotechnical exploration for the proposed Shore Stabilization and Pedestrian Bridge project in Matagorda, Texas. Mr. Herb Smith, President, of Baker and Lawson, Inc. authorized the services on by signing PSI Proposal 286-29209, dated September 8, 2010.

PROJECT DESCRIPTION

Mr. Doug Roesler of Baker and Lawson provided the project information to PSI. It is understood that the proposed project consists of two projects:

1. Bank Stabilization/Re-establishment, and,
2. Construction of Pedestrian Footbridge

The projects are located near Jetty Park, within the southeastern quadrant of Fort Velasco Drive and W 14th Street, in Surfside, Texas. Plate 1 in the Appendix shows the site location. The area currently consists of a peninsula that is gradually being eroded, in part due to its location adjacent to the entrance to the Intracoastal Canal and Gulf of Mexico. The depth of water surrounding the peninsula is understood to be about 4 feet or less.

In general, the peninsula is 150 feet wide by 170 feet long. A culvert outlet is located on the northeastern side of the peninsula. A topographic drawing of the site was provided by Baker and Lawson, as shown on Plate 2 in the Appendix.

Based on the topographic drawing the existing grade at the peninsula and east end of the pedestrian bridge is at about elevation +10 feet. The existing grade at the west end of the pedestrian bridge is at about elevation +7 feet. The bank at both sides consists of a steep drop to about El. +0 ft. The grade then gradually slopes to an elevation of about -2 feet.

The locations of the pedestrian bridge and limits of the proposed bank stabilization are shown on Plate 2. It is understood that along with bank stabilization, land will be reclaimed with the use of sheet piling. The top of the sheet pile will be at El. +5 feet and located about 12 to 30 feet in front of the existing bank. This area will be backfilled and sloped 6H:1V from El. +5 feet to the elevation of the existing bank (El. +7 feet to +10 feet).

The pedestrian footbridge is planned to be located on the southwestern side of the peninsula, crossing towards an existing road, generally parallel with W. 14th Street. The bridge will be approximately 150 feet long consisting of driven 6 inch timber piles.

The geotechnical recommendations presented in this report are based on the available project information, site location, laboratory testing, and the subsurface materials described in this report. If any of the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

PURPOSE AND SCOPE OF SERVICES

The purpose of the geotechnical study was to explore the subsurface conditions at the site to enable an evaluation for a new sheet pile bulkhead and pedestrian bridge. The geotechnical exploration for this project involved the collection of subsurface data, laboratory testing, and geotechnical analyses. The scope of services includes drilling a total of four (4) soil borings. The scope of services also included laboratory testing, and preparation of this geotechnical report. This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents recommendations regarding the following:

- Sheet pile analysis
- Slope stability analyses
- Deep foundation recommendations
- Comments regarding factors that will impact construction and performance of the proposed construction.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, surface water, ground water, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes. A geologic fault study to evaluate the possibility of surface faulting at this site was beyond the scope of this investigation. Should you desire a detailed fault study, please contact us.

SITE AND SUBSURFACE CONDITIONS

SITE LOCATION AND DESCRIPTION

The project site is located near Jetty Park, within the southeastern quadrant of Fort Velasco Drive and W 14th Street, in Surfside, Texas. At the time of the field study, the ground surface generally consisted of grass within the eastern project site (peninsula) and tall grass/weeds within the western project site. Both sites were generally being eroded with steep slopes from soil falling away. Sloughing was observed within the slope on the southern side of the peninsula.

SUBSURFACE CONDITIONS

The subsurface conditions were explored by drilling a total of four (4) soil borings at the site. Borings B-1 to B-4 were drilled to a depth of 60 feet below the existing ground surface. Plate 1 in the Appendix shows the approximate boring locations in plan.

The borings were located in the field by a PSI representative. The borings were drilled using a track mounted drilling equipment and wet rotary drilling methods. Soil samples were routinely obtained during the drilling process. Drilling and sampling techniques were accomplished generally in accordance with ASTM procedures (ASTM D 1586 and D 1587).

The soil samples obtained during the field exploration were transported to the laboratory and selected soil samples were tested in the laboratory to determine material properties for our evaluation. Laboratory testing was accomplished generally in accordance with ASTM procedures. Laboratory testing was performed on selected samples to evaluate the classification, strength and other engineering characteristics of the subsurface materials. Laboratory testing on selected samples included Moisture Content (ASTM D 2216), Unit Weight, Atterberg Limits (ASTM D 4318), Percent Passing No. 200 Sieve (ASTM D 1140), Unconfined Compression (ASTM D 2166), and Unconsolidated Undrained triaxial (ASTM D 2850).

The soil samples obtained from the drilling operation were classified in general accordance with ASTM D 2487 or D 2488. Laboratory test data along with detailed descriptions of the soils can be found on the logs of borings. Plates 3 through 6 located in the Appendix show the logs of borings. A key to terms and symbols used on the logs is presented on Plate 7 located in the Appendix.

The subsurface soil strata identified by the borings generally consists of fill soils that consist of silty sand and clayey sand, extending from the ground surface (approx. El. +10 feet) to about El. +2 feet. Below the fill soils are medium dense to dense silty sand (SM) soils that extend from about El. +2 feet to El. -12 feet. Extending from El. -12 feet to the explored depth at El. -50 feet are stiff to very stiff fat clay (CH) soils. Detailed descriptions of the soils encountered are presented on the attached boring logs. Based on the soil borings, a generalized soil profile was developed for this site and the profile is shown on Plate 8 in the Appendix.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the Appendices should be reviewed for specific information at individual boring locations. These records include soil descriptions, stratification, locations of the samples, and laboratory test data. The stratification shown on the boring logs represent the conditions only at the actual boring locations. Variations may occur and should be expected across the site. The stratification represents the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on the boring logs. The samples, which were not altered by laboratory testing, will be retained for 60 days from the date of this report and then will be discarded without further notice.

GROUNDWATER INFORMATION

Groundwater was found at a depth of about 8 feet below the existing ground surface (approx. El. +2 feet) during drilling. Groundwater was found at a depth of about 5 feet below the existing ground surface (approx. El. +5 feet) 24 hours after drilling.

It is possible that seasonal variations (rainfall, tidal, hurricane, etc) will cause fluctuations in the groundwater level. Additionally, perched water may be encountered in discontinuous zones within the overburden. The groundwater levels presented in this report are the levels that were measured at the time of our field activities. We recommend that the contractor determine the actual groundwater levels at the site at the time of the construction activities to determine the impact, if any, on the construction procedures.

EVALUATION AND RECOMMENDATIONS

It is understood that the proposed project consists of two projects:

1. Bank Stabilization/Re-establishment, and,
2. Construction of Pedestrian Footbridge

The locations of the pedestrian bridge and limits of the proposed bank stabilization are shown on Plate 2 in the Appendix. Based on the topographic drawing provided, the existing grade at the peninsula and east end of the pedestrian bridge is at about elevation +10 feet. The existing grade at the west end of the pedestrian bridge is at about elevation +7 feet. The bank at both sides consists of a steep drop to about El. +0 ft. The grade then gradually slopes to an elevation of about -2 feet.

It is understood that along with bank stabilization, land will be reclaimed with the use of sheet piling. The top of the sheet pile will be at El. +5 feet and located about 12 to 30 feet in front of the existing bank. This area will be backfilled and sloped 6H:1V from El. +5 feet to the elevation of the existing bank (El. +7 feet to +10 feet). It is anticipated that the shore will be stabilized using a cantilevered sheet pile system.

Evaluation of the sheet pile wall will consider the potential for a maximum of five (5) feet of scour in front of the sheet pile wall. No surcharge loads behind the wall are anticipated.

Additionally, due to the potential for erosion behind the sheet pile wall from rain and large waves during strong storms, it is recommended that the top two (2) feet of backfill placed behind the sheet pile wall be clay soils. Below the top two (2) feet, the backfill material may consist of either clay or sand. Plate 9 shows a schematic of the sheet pile wall and the area to be backfilled.

The pedestrian footbridge is planned to be located on the southwestern side of the peninsula, crossing towards an existing road, generally parallel with W. 14th Street. The bridge will be approximately 150 feet long consisting of driven 6 inch timber piles.

SHEET PILE WALL ANALYSES

Cantilever sheet piles derive their support to resist the lateral earth pressures by the passive pressures on the front of the embedded portion of the sheet pile. A computer program "CWALSHT" was used to perform the analyses of anchored sheet pile walls. The program is developed by Information Technology Laboratory, Department of Army, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi and has been approved for public release in October 1991.

The program utilizes classical soil mechanics procedures to determine the minimum wall penetration depths, moment and shear along the length of the pile. Classical soil mechanics procedures include determining the earth pressure coefficients (both active and passive) based on Coulomb/Rankine theory. The analyses were performed based upon the ultimate earth pressure values; i.e., with a factor of safety 1.0 and utilizing Free-Earth support method.

Analysis was performed for a cantilevered sheet pile wall. Design assumptions for analysis of the sheet pile are:

- top of sheet pile: El. +5 feet
- slope behind sheet pile wall: 6H:1V

- existing grade beyond slope/behind the sheet pile: El. +10 feet
- mudline in front of the sheet pile: El. -7 feet (neglects 5 feet for scour)
- no surcharge load
- water elevation: El. +0 feet (both sides)

The analyses were performed using short term and long term soil parameters. The analyses included the determination of 1) minimum penetration required, 2) shear force along the length of the pile, 3) maximum moment along the length of the pile, and 4) anchor force. The results for are summarized in Table 1 and are included on Plates 10 and 11 in the Appendix. The results indicate that long term soil conditions govern the pile penetration for a cantilevered sheet pile wall.

Table 1: Summary of Sheet Pile Analysis

Sheet Pile Type	Soil Condition	Minimum Tip Elevation (feet)	Minimum Penetration length (feet)	Maximum Moment (k-ft)
Cantilevered	Short Term (Plate 10)	-27.5	20.5	82.0
	Long Term (Plate 11)	-33.7	26.7	91.0

The results shown correspond to a factor of safety of 1.0. Therefore, it is recommended to increase the embedment length of the sheet pile by 30 percent. The structural engineer in determining the final sectional modulus of the sheet pile should assume an adequate factor of safety.

Global or overall slope stability of the sheet pile system can be performed after the depth of the sheet pile is determined. At this time, global slope stability analysis was performed to determine the minimum pile length required for slope stability, which resulted in a sheet pile with a tip El. -25 ft. Based on the analyses, the length of the wall will be governed by the sheet pile wall analysis. A sheet pile with a deeper tip elevation will have the same or increased factor of safety. For the overall slope stability of the system, a factor of safety of at least 1.5 is recommended.

BACKFILL BEHIND SHEET PILE WALL

As previously mentioned, due to the potential for erosion behind the sheet pile wall from rain and large waves during strong storms, it is recommended that the top two (2) feet of backfill placed behind the sheet pile wall be clay soils. Below the top two (2) feet, the backfill material may consist of either clay or sand. Plate 9 shows a schematic of the sheet pile wall and the area to be backfilled.

The fill soils should be placed and compacted at the sheet pile wall first and then compacting away from the wall. The fill soils should be compacted to at least 95 percent of standard Proctor maximum dry density as determined by ASTM D 698.

SLOPE STABILITY ANALYSES

Generally, slope failure occurs when the weight of the sliding soil exceeds the resistance derived from the shear strength or frictional resistance of the soil along the sliding surface. Slope stability analysis involves the determination of the most likely sliding surface by comparing the developed shearing resistance along a sliding surface with the weight forces associated with the sliding soil. The method of comparison involves the determination of the factor of safety i.e., a ratio of shear resistance along a sliding surface to the weight of the sliding soil. Slope stability analysis is a method to check the stability of the soil slope. The check for stability involves determination of various values factor of safety along various assumed likely sliding surfaces. The least value of factor of safety corresponding to a sliding surface is considered the stability of the slope. For the slope stability analysis, the failing surface is typically assumed to be circular.

For the present study, to predict the factor of safety against sliding, a computer program SLOPE-W was utilized. The analysis included the determination of a particular sliding surface that has a least factor of safety against sliding. For the slope stability analyses, the following three different soil conditions were considered:

1) Short Term or Undrained Condition: This condition occurs when the pore pressures within the soil mass are not dissipated. Typically, this condition corresponds to the state of the soils that exists immediately after performing any cut/fill or during the construction of any slope. For this condition, slopes are analyzed using undrained soil parameters obtained from laboratory tests such as unconfined compression tests and unconsolidated-undrained tests.

2) Long Term or Drained Condition: This condition occurs when the pore pressures within the soil mass are dissipated. Typically, this condition corresponds to the state of the soils a few months or years after the construction is complete. For this condition, slopes are analyzed using drained or effective stress parameters.

3) Sudden Drawdown Condition: This condition occurs when the water level rises during a flood saturating the side slopes, and then drains rapidly as the flood waters recede. The state of stress within the soils of the slope after a flood event depends largely on the permeability and drainage characteristics of the slope as well as the state of the soils prior to drawdown (i.e., flood event, hurricane). This analysis was performed using effective stress parameters with a water elevation at El. -2 feet and the phreatic surface to coincide with the slope surface.

The soil strength parameters selected for the slope stability calculations are based on: 1) the results of our field and laboratory test data, 2) engineering judgment based on experience with similar soils and 3) published correlations[1]. The soil strength parameters used in the stability analysis for various soil layers identified are given in Table 2.

[1] A.A Saleh and Stephen G. Wright; “Shear Strength Correlations and Remedial Measure Guidelines for Long Term Stability of Slopes Constructed of Highly Plastic Clay Soils”, Research Report 1435-2F, Center of Transportation Research Bureau of Engineering Research, UT Austin, Oct. 1997.

Table 2: Soil Parameters for Slope Stability Analyses

Layer or Stratum	Soil Parameter(s)	
	Undrained	Drained
Silty Sand (SM) or Clayey Sand (SC) El. +10 to +0 feet	N/A	$\phi = 20^{\circ}$
Silty Sand (SM) El. +0 to -12 feet	N/A	$\phi = 30^{\circ}$
Fat Clay (CH) or Lean Clay (CL) El. -12 to -17 feet	Su = 50 psf	c' = 0 psf, $\phi' = 10^{\circ}$
Fat Clay (CH) Below El. -17 feet	Su = 2,000 psf	c' = 100 psf, $\phi' = 25^{\circ}$

Notes: Su - Undrained Shear Strength, c' - drained cohesion, ϕ' - drained angle of internal friction

Using the selected soil parameters, slope stability analyses were performed to determine the minimum sheet pile wall length required for slope stability. A surcharge pressure was not considered in the analysis. It should be realized that surcharge pressures would result in additional driving forces on the soil mass that will increase the potential for the soil mass to slide (i.e., resulting in a lower factor of safety against sliding). Therefore, it is recommended that during the service period of the slope, adequate measures should be adopted not to impose any long-term surcharge pressures on top of the slope.

Critical circles having minimum factors of safety were determined for the slope cross sections for the various soil conditions. Typically, a factor of safety of 1.5 should be considered as the minimum acceptable safety factor for slopes to be considered stable with low risk of failure in extreme loading conditions for short term and long term soil conditions. For sudden draw down conditions, a factor of safety of 1.1 to 1.3 is typically acceptable. Results of the slope stability analyses are shown in Table 3 and are shown on Plates 12 to 14 in the Appendix.

Table 3: Results of Slope Stability Analysis

Soil Conditions	Factor of Safety
Short Term Soil Conditions	FS = 3.40 (Plate 12)
Long Term Soil Conditions	FS = 1.63 (Plate 13)
Sudden Drawdown: Effective Parameters	FS = 1.22 (Plate 14)

Based on the slope stability analyses and subsurface layers, a minimum sheet pile wall length of 30 feet, or tip elevation of -25 feet, is recommended for slope stability. However, for this project, the length of the sheet pile wall will be governed by the sheet pile wall analysis.

DRIVEN PILE FOUNDATION RECOMMENDATIONS

It is understood that the pedestrian bridge will be supported using 6 inch square driven timber piles. At this time, it is unclear if bridge bents will be required. Therefore, two pile capacities are provided: 1) bridge abutments and 2) bridge bents.

The bridge abutments are assumed to be located behind the sheet pile wall at El. +5 feet and therefore will not be subjected to scour. The bridge bents are assumed to be located in front of the sheet pile wall at El. -2 feet and therefore will be subjected to scour. An additional 5 feet will be neglected for the bridge bents to account for the potential scour.

Design Criteria: The load carrying capacity of driven piles can be computed using the static method of analysis. According to this method, the axial capacity, Q, at a given penetration is taken as the sum of the skin friction on the side of the pile, Q_s, and the end or point bearing at the pile tip, Q_p, so that:

$$Q = Q_s + Q_p = fA_s + qA_p$$

Where, A_s and A_p represent, respectively, the embedded surface area and the end area of the pile; f and q represent, respectively, the allowable unit skin friction and the allowable unit end or point bearing.

The total axial capacity in compression will be the summation of the frictional capacity and the end bearing capacity. The total axial capacity in tension will be the frictional capacity alone neglecting the end-bearing component.

Axial Capacity of Driven Piles: For this site, based on the evaluation of the soil conditions, field and laboratory test results, the recommended allowable unit skin friction values are shown in Table 4. A factor of safety of at least 2.0 is included for the unit skin friction to arrive at the allowable values.

Table 4. Recommended Allowable Unit Skin Friction Values

Bridge Support	Elevation (feet)	Skin Friction (ksf)
Bridge Abutment (located behind sheet pile wall, El. +5 feet)	+5 to +0	N/A
	+0 to -12	0.07
	-12 to -17	0.015
	-17 to - 45	0.6
Bridge Bent (located in front of sheet pile wall, El. -2 feet)	-2 to -7	N/A
	-7 to -12	0.07
	-12 to -17	0.015
	-17 to - 45	0.6

Lateral Capacity: For drilled piles, the soil as well as the rigidity of the pile resists the lateral loads on the pile. Once the locations, loads and other pertinent information are provided, PSI can assist in performing lateral load analyses based on methods ranging from chart solutions to the 'p-y' approach utilizing computer programs such as LPILE or COM 624.

The lateral design information regarding the 'p-y' data is provided in Tables 5 and 6. The relationship between the soil resistance (p) and pile deflection (y) is commonly referred to as 'p-y'. Along the depth of the pile, soil resistance (p) is expressed as a non-linear function of lateral pile deflection (y). Various researchers developed 'p-y' criteria for different kinds of soils. The 'p-y' curves can be automatically generated utilizing the computer program LPILE. The program LPILE was developed by Lymon Reese and Shin-Tower Wang, Ensoft, Inc. 'p-y' parameters for LPILE analyses are provided for the analyses of individual piles.

Table 5: Soil Parameters to be used in the Lateral Load Analyses: Bridge Abutments

Elevation (feet)	'p-y' Criteria	Effective Unit Weight, γ (pcf)	Su (ksf) , or φ degrees	Ks (pci) or Kc (pci)	ε_{50}
+5 to +0	N/A	120	N/A	N/A	N/A
+0 to -12	Submerged Sand Criteria	60	$\varphi = 30^0$	K = 60 pci	-
-12 to -17	Soft Clay Criteria	60	Su = 0.05	Ks = 30 pci	0.02
-17 to - 45	Stiff Clay Criteria	60	Su = 2.0	Ks = 500 pci Kc = 200 pci	0.007

Note: Su-Undrained Shear Strength (tsf); φ, Angle of Internal friction; ks-modulus of subgrade reaction (pci) for static loading condition; kc-modulus of subgrade reaction (pci) for cyclic loading condition; ε_{50} – strain corresponding to one-half the principle stress.

Table 6: Soil Parameters to be used in the Lateral Load Analyses: Bridge Bents

Elevation (feet)	'p-y' Criteria	Effective Unit Weight, γ (pcf)	Su (ksf) , or φ degrees	Ks (pci) or Kc (pci)	ε_{50}
-2 to -7	N/A	120	N/A	N/A	N/A
-7 to -12	Submerged Sand Criteria	60	$\varphi = 30^0$	K = 60 pci	-
-12 to -17	Soft Clay Criteria	60	Su = 0.05	Ks = 30 pci	0.02
-17 to - 45	Stiff Clay Criteria	60	Su = 2.0	Ks = 500 pci Kc = 200 pci	0.007

Note: Su-Undrained Shear Strength (tsf); φ, Angle of Internal friction; ks-modulus of subgrade reaction (pci) for static loading condition; kc-modulus of subgrade reaction (pci) for cyclic loading condition; ε_{50} – strain corresponding to one-half the principle stress.

Settlement: A detailed analysis of axial load versus settlement was beyond the scope of this study. However, for a single isolated pile designed in accordance with the computed ultimate capacities and recommended factor of safety, the settlement should be less than ½ inch. A detailed settlement analysis for piles in a group was beyond the scope of this study. If desired PSI can assist in performing such a study.

Piles in Group: A group of piles subjected to vertical loads may not necessarily have the same capacity as the sum of the capacities of the individual piles. For axially loaded piles, published results indicate that the ratio of capacity per pile in a group to that of a single isolated pile typically ranges from 0.5 to 1.0. This efficiency factor depends on the spacing or distance between each pile. In planning groups of drilled piles, a minimum center-to-center spacing of 3D (where D is the diameter or the width) is recommended to avoid the reduction in capacity. Group action and settlement should be checked after the actual pile spacing is determined.

A group of driven piles subjected to lateral loads may not have the same capacity as the sum of the capacity of the individual piles. PSI should be contacted, once the pile group orientation, spacing and loading direction is determined.

Driven Pile Installation: Piles can be driven in accordance with Item 404 of Texas Department of Transportation, Standard specification for construction of highways, streets and bridges (TxDOT Specification). Timber piles can be in accordance with Item 406 of TxDOT Specification.

The piles should be driven to the desired penetration depth by driving methods alone. Any supplemental techniques such as use of vibratory hammers, pilot holes or jetting should be avoided whenever possible. These supplemental techniques, if not used properly, can reduce the pile capacity. For piles driven in stiff or dense soils, some researchers suggest that the use of vibratory hammers can reduce the pile capacity. If the use of pilot holes or jetting becomes necessary, PSI should be contacted to provide further recommendations. If techniques other than driving such as pilot hole, vibratory technique or jetting are used to aid pile installation, conditions assumed in computations based on driving alone may not be met. Driving should be performed such that the hammer speeds are adjusted appropriately and that the piles are not over-stressed and crack.

It is recommended that the pile driving be monitored by a geotechnical engineer or qualified technician. Sometimes, premature refusal occurs due to poor hammer performance rather than from soil resistance. Any changes in hammer blow counts should be carefully examined before making any decisions about the pile penetration.

Piles could heave during the driving process as a result of being driven adjacent to one another. It is suggested that if piles heave more than about 0.25 inch during the driving of an adjacent pile, the heaved piles should be re-driven to their initial depth. The ground surface surrounding the piles could also heave as a result of driving these displacement piles. When driving piles near existing structures, it is recommended that the driving sequence start with the piles nearest the existing structures and progress in a direction away from the structures.

It should be realized the pile capacity is dependent on the prevention of damage to the pile during construction. A damaged pile will have reduced performance under sustained loading conditions. Proper pile handling and proper driving govern the pile construction during construction. In addition, pile material strength, especially for precast piles can also dictate pile capacity. Good pile driving practice should be adopted to ensure proper driving. Among other things, proper driving includes the use of a proper pile cushion, reducing or increasing the hammer speed anticipating soft or hard driving situations. A proper pile driving record should be utilized to ensure proper pile driving. In maintaining pile driving records, among other things the primary things include, recording the blow counts for every foot of pile penetration, and, and recording actual time (excluding stoppage) that was required to drive the pile.

Selection of an appropriate hammer depends on several factors such as hammer performance, cushion type and size, pile type, pile size and length, pile weight, predicted or required pile capacity, soil resistance, etc. The selected hammer must be able to drive the pile to the required capacity or length without damaging the pile. Generally, experience of local contractor is often the primary source for the selection of the hammer. Wave equation analysis of piles may be used to aid in hammer selection.

CONSTRUCTION CONSIDERATIONS

It is recommended that PSI be retained to provide observation and testing of construction activities involved in the foundations, earthwork, and related activities of this project. PSI cannot accept any responsibility for any conditions that deviated from those described in this report, nor for the performance of the foundations if not engaged to also provide construction observation and testing for this project.

MOISTURE SENSITIVE SOILS/WEATHER RELATED

During wet weather periods and/or poor site drainage, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. Soils that become wet might be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform any earthwork and foundation construction activities during dry weather.

EXCAVATIONS

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document was issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "competent", as defined in 29 CFR Part 1926.650 to 652 should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's or other party's compliance with local, state, and federal safety or other regulations.

REPORT LIMITATIONS

The recommendations submitted in this report are preliminary, generalized, and based on the available subsurface information obtained by PSI and details furnished by representatives of Baker and Lawson, Inc. for the proposed project. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation recommendations are required. If PSI is not notified of such changes, PSI will not be responsible for the impact of those changes on the project.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the geotechnical engineer should be retained and provided the opportunity to perform a final geotechnical exploration that could be used as a basis of design and construction. If PSI is not retained to perform these functions, PSI will not be responsible for the final design. This report has been prepared for the exclusive use of Baker and Lawson, Inc. for the specific application to the proposed shore stabilization and pedestrian bridge in Surfside, Texas.

APPENDIX

BORING LOCATION PLAN

NOTES

APPROXIMATE BORING LOCATIONS

N

PEDESTRIAN BRIDGE AND
SHORE STABILIZATION
SURFSIDE, TEXAS

Professional Service Industries, Inc.
1714 Memorial Drive
Houston, Texas 77007

Drawn:	Scale:	Project No.:
EC	NOT TO SCALE	286-373
Chkd:	Date:	
EC	12/9/2010	

PLATE 1

NO.	DATE	DESCRIPTION	APPROVED
		REVISIONS	

DESIGNED DBR
DRAWN MEP
CHECKED
DATE

B&L BAKER & LAWSON, INC.
ENGINEERS • PLANNERS • SURVEYORS
300 E. CEDAR ST. ANGLETON, TEXAS 77515
PHONE: (979) 849-6681 FAX: (979) 849-4689

STATE OF TEXAS
DOUGLAS B. ROESLER
56739
REGISTERED PROFESSIONAL ENGINEER

The seal appearing on this document was authorized by Douglas B. Roesler P.E. 56739

Date:

OWNER:
BRAZORIA COUNTY
TEXAS

PLAN: 1" = 20'
PROFILE:
HORIZONTAL:
VERTICAL:

COUNTY PARK
RESTORATION

LAKE JACKSON, TEXAS

SHEET 6 OF 17 SHEETS

EXISTING CONDITIONS
SITE PLAN

PROJECT NO. 10580

Plate 2 1

10580_SITE.DGN

LOG OF BORING B-1

Surfside Pedestrian Bridge and Shore Stabilization
Surfside,Texas

TYPE OF BORING: Wet Rotary

PSI Project No.: 286-373

COORDINATE (X) OR EASTING: 100
COORDINATE (Y) OR NORTHING: 100
APPROXIMATE SURFACE ELEVATION: 7 feet
LATITUDE:
LONGITUDE:

SHEAR STRENGTH (tons/square foot): ○ HP ● UC △ TV ▲ UU — scale 0.0 0.5 1.0 1.5 2.0 2.5

DEPTH, FT.	USCS SYMBOL	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	UNIT WEIGHT (pcf)
		FILL: SILTY SAND, LOOSE, LIGHT BROWN	5					14	
		FILL: SANDY LEAN CLAY, FIRM, BROWN	7					17	
5			4	67	38	16	22	23	
								24	
	SM	**SILTY SAND (SM)** DENSE, GRAY	30					21	
10									
		- with seashells, 13 to 15 feet	31	4				23	
15									
	CH	**FAT CLAY (CH)** FIRM TO VERY STIFF, GRAY AND YELLOWISH BROWN		89	51	21	30	32	107
20									
		- with silt seams, 23 to 30 feet						22	
25									
								21	
30									
		- yellowish brown, 33 to 40 feet - with calcareous nodules, 33 to 40 feet		98	56	25	31	27	99
35									
								26	
40									
		- gray and light reddish brown, 43 to 45 feet						27	101
45									
	CL	**LEAN CLAY (CL)** STIFF, LIGHT REDDISH BROWN AND GRAY - with silt seams						19	
50									

DEPTH OF BORING: 60 FEET

DATE DRILLED: 10/25/10

NOTES:

INITIAL GROUND WATER: 8 FEET
FINAL GROUND WATER: 5 FEET, 24 HOURS AFTER DRILLING

psi Information To Build On
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1714 Memorial Drive
Houston, Texas 77007

LOG OF BORING B-1

Surfside Pedestrian Bridge and Shore Stabilization
Surfside,Texas

TYPE OF BORING: Wet Rotary

PSI Project No.: 286-373

DEPTH, FT.	SOIL TYPE	USCS SYMBOL	SAMPLES	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	SHEAR STRENGTH (tons/square foot) ○ HP ● UC △ TV ▲ UU 0.0 0.5 1.0 1.5 2.0 2.5	UNIT WEIGHT (pcf)
		CH		**FAT CLAY (CH)** VERY STIFF TO HARD, GRAY AND BROWN		99	79	36	43	38		92
55												
										28		
60												
65												
70												
75												
80												
85												
90												
95												
100												

DEPTH OF BORING: 60 FEET

DATE DRILLED: 10/25/10

NOTES:

INITIAL GROUND WATER: 8 FEET

FINAL GROUND WATER: 5 FEET, 24 HOURS AFTER DRILLING

BORINGLOG_HOUSTON - PSIHOUSTON.GDT - 12/21/10 10:21 - P:\286 REPORTS\286 2010 REPORTS\286-373 SURFSIDE FOOTBRIDGE\286-373 BORING LOGS.GPJ

LOG OF BORING B-2

Surfside Pedestrian Bridge and Shore Stabilization
Surfside,Texas

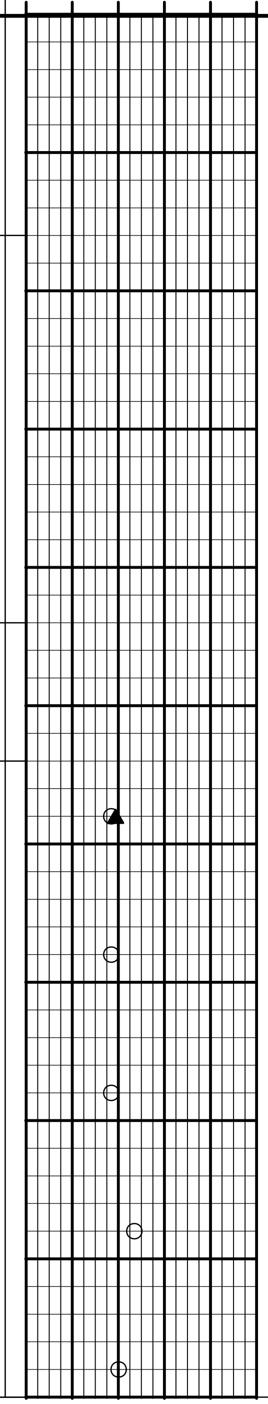
TYPE OF BORING: Wet Rotary

PSI Project No.: 286-373

COORDINATE (X) OR EASTING: 300
COORDINATE (Y) OR NORTHING: 100
APPROXIMATE SURFACE ELEVATION: 10 feet
LATITUDE:
LONGITUDE:

DEPTH, FT.	SOIL TYPE	USCS SYMBOL	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	UNIT WEIGHT (pcf)
0	FILL		**FILL: CLAYEY SAND**, LOOSE TO MEDIUM DENSE, BROWN - with organic material, 0 to 2 feet	17					19	
				12	32	28	12	16	16	
5			- with gravel and asphalt, 4 to 6 feet	15					22	
				8		48	20	28	17	
		SM	**SILTY SAND (SM)** MEDIUM DENSE TO DENSE, LIGHT BROWN TO GRAY	14	11				23	
10										
			- gray, 13 to 20 feet	39					23	
15										
			- with seashells, 18 to 20 feet	35					22	
20										
		CL	**LEAN CLAY (CL)** VERY SOFT, BROWN AND GRAY - WOH: weight of hammer	WOH	87	46	20	26	34	
25										
		CH	**FAT CLAY (CH)** STIFF TO VERY STIFF, GRAY AND REDDISH BROWN - with calcareous nodules, 28 to 30 feet						23	101
30										
									31	
35										
									27	
40										
			- with calcareous nodules, 43 to 50 feet		98	65	14	51	28	
45										
									26	
50										

SHEAR STRENGTH (tons/square foot)
○ HP ● UC △ TV ▲ UU
0.0 0.5 1.0 1.5 2.0 2.5



DEPTH OF BORING: 60 FEET
DATE DRILLED: 10/22/10
NOTES:

INITIAL GROUND WATER: 8 FEET
FINAL GROUND WATER: CAVED AT 10 FEET

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LOG OF BORING B-2

Surfside Pedestrian Bridge and Shore Stabilization
Surfside,Texas

TYPE OF BORING: Wet Rotary

PSI Project No.: 286-373

DEPTH, FT.	SOIL TYPE	USCS SYMBOL	SAMPLES	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)	SHEAR STRENGTH (tons/square foot) ○ HP ● UC △ TV ▲ UU 0.0 0.5 1.0 1.5 2.0 2.5	UNIT WEIGHT (pcf)
		CH		**FAT CLAY (CH)** STIFF TO VERY STIFF, REDDISH BROWN								
										22		107
55												
										33		
60												
65												
70												
75												
80												
85												
90												
95												
100												

DEPTH OF BORING: 60 FEET

DATE DRILLED: 10/22/10

NOTES:

INITIAL GROUND WATER: 8 FEET

FINAL GROUND WATER: CAVED AT 10 FEET

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LOG OF BORING B-3

Surfside Pedestrian Bridge and Shore Stabilization
Surfside,Texas

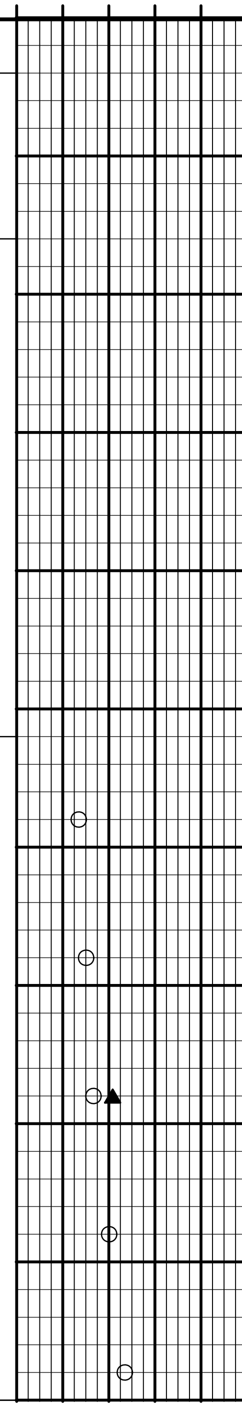
TYPE OF BORING: Wet Rotary

PSI Project No.: 286-373

COORDINATE (X) OR EASTING: 400
COORDINATE (Y) OR NORTHING: 150
APPROXIMATE SURFACE ELEVATION: 10 feet
LATITUDE:
LONGITUDE:

DEPTH, FT.	USCS SYMBOL	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	UNIT WEIGHT (pcf)
		FILL: SILTY SAND, MEDIUM DENSE, BROWN						11	
		- with organic material	12					13	
		FILL: CLAYEY SAND, MEDIUM DENSE, BROWN - with organic material, 2 to 4 feet	11					23	
5			7		34	15	19	26	
			23	45				26	
10	SM	**SILTY SAND (SM)** LOOSE TO DENSE, GRAY	35					22	
15		- with seashells, 13 to 15 feet	34					26	
20			32	5				22	
25		- loose, 23 to 25 feet - with seashells, 23 to 25 feet	9					23	
	CH	**FAT CLAY (CH)** STIFF TO VERY STIFF, GRAY AND REDDISH BROWN							
30		- with calcareous nodules, 28 to 30 feet						27	
35				99	78	29	49	35	
40		- with calcareous nodules, 38 to 45 feet						26	99
45								29	
50								26	

SHEAR STRENGTH (tons/square foot)



DEPTH OF BORING: 60 FEET

DATE DRILLED: 10/22/10

INITIAL GROUND WATER: 8 FEET
FINAL GROUND WATER: CAVED AT 9 FEET

NOTES:

LOG OF BORING B-3

Surfside Pedestrian Bridge and Shore Stabilization
Surfside,Texas

TYPE OF BORING: Wet Rotary

PSI Project No.: 286-373

DEPTH, FT.	SOIL TYPE	USCS SYMBOL	SAMPLES	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	SHEAR STRENGTH (tons/square foot) ○ HP ● UC △ TV ▲ UU 0.0 0.5 1.0 1.5 2.0 2.5	UNIT WEIGHT (pcf)
	▨	CH		**FAT CLAY (CH)** STIFF TO VERY STIFF, GRAY AND BROWN								
	▨		■				54	20	34	20	○	
55	▨											
	▨		■							31	○ ▲	91
60												
65												
70												
75												
80												
85												
90												
95												
100												

DEPTH OF BORING: 60 FEET

DATE DRILLED: 10/22/10

NOTES:

INITIAL GROUND WATER: 8 FEET

FINAL GROUND WATER: CAVED AT 9 FEET

LOG OF BORING B-4

Surfside Pedestrian Bridge and Shore Stabilization
Surfside,Texas

TYPE OF BORING: Wet Rotary

PSI Project No.: 286-373

COORDINATE (X) OR EASTING: 400
COORDINATE (Y) OR NORTHING: 50
APPROXIMATE SURFACE ELEVATION: 10 feet
LATITUDE:
LONGITUDE:

DEPTH, FT.	USCS SYMBOL	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)	SHEAR STRENGTH (tons/square foot) ○ HP ● UC △ TV ▲ UU 0.0 0.5 1.0 1.5 2.0 2.5	UNIT WEIGHT (pcf)
0–8		**FILL: CLAYEY SAND**, MEDIUM DENSE, GRAYISH BROWN - with organic material, 0 to 2 feet - with seashells, 2 to 4 feet	17					12		
			19	33	31	14	17	12		
5			12					17		
			11					18		
8–22	SM	**SILTY SAND (SM)** DENSE, GRAY	33					24		
10		- with seashells, 13 to 20 feet	43	5				23		
15–20			37					21		
22–25	SC	**CLAYEY SAND (SC)** LOOSE, BROWNISH GRAY - with seashells	7	18				33		
25–50	CH	**FAT CLAY (CH)** STIFF TO VERY STIFF, GRAY AND BROWN - with calcareous nodules, 28 to 30 feet						28	● ○	101
30–35								22	○	
35–40				98	68	24	44	26	○	
40–45		- with calcareous nodules, 43 to 45 feet						28	● ○	97
45–50								25	○	

DEPTH OF BORING: 60 FEET
DATE DRILLED: 10/22/10

INITIAL GROUND WATER: 8 FEET
FINAL GROUND WATER: 5 FEET, 24 HOURS AFTER DRILLING

NOTES:

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LOG OF BORING B-4

Surfside Pedestrian Bridge and Shore Stabilization
Surfside,Texas

TYPE OF BORING: Wet Rotary

PSI Project No.: 286-373

DEPTH, FT.	SOIL TYPE	USCS SYMBOL	SAMPLES	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	SHEAR STRENGTH (tons/square foot) ○ HP ● UC △ TV ▲ UU 0.0 0.5 1.0 1.5 2.0 2.5	UNIT WEIGHT (pcf)
		CH		**FAT CLAY (CH)** STIFF TO VERY STIFF, GRAY AND REDDISH BROWN								
						94	53	21	32	23		105
55												
										25		
60												
65												
70												
75												
80												
85												
90												
95												
100												

DEPTH OF BORING: 60 FEET

DATE DRILLED: 10/22/10

INITIAL GROUND WATER: 8 FEET

FINAL GROUND WATER: 5 FEET, 24 HOURS AFTER DRILLING

NOTES:

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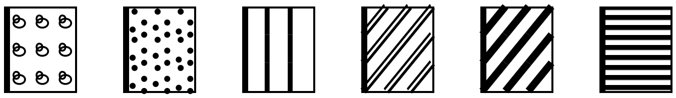
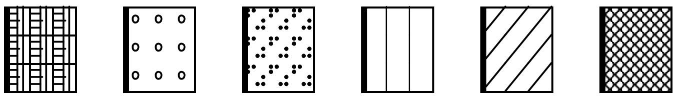
KEY TO TERMS AND SYMBOLS USED ON LOGS

SOIL TYPE



GRAVEL | SAND | SILT | LEAN CLAY | FAT CLAY | PEAT

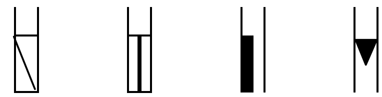
MODIFIERS



STONE | GRAVELY | SANDY | SILTY | CLAYEY | MISC. (SEE TEXT ON LOG)

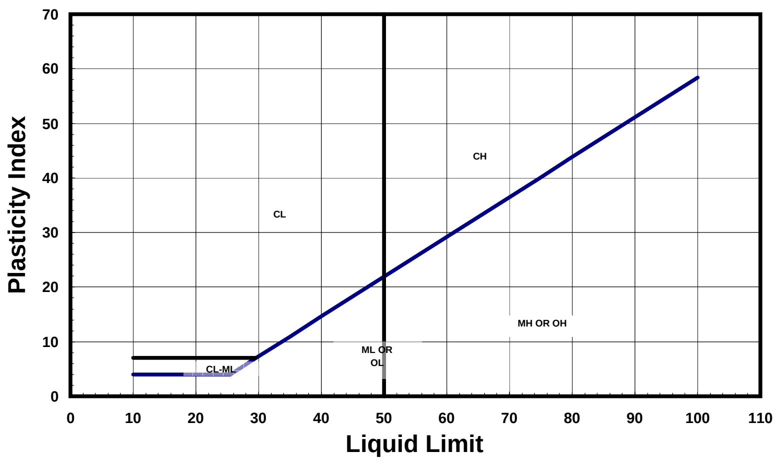
SAMPLER TYPE

NO SAMPLE | AUGER SAMPLE | SHELBY TUBE | SPLIT SPOON



NO RECOVERY | ROCK CORE | 2" SHELBY TUBE | TXDOT CONE

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D 2487



CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	SHEAR STRENGTH IN TONS/FT^2
VERY SOFT	0 TO 0.125
SOFT	0.125 TO 0.25
FIRM	0.25 TO 0.5
STIFF	0.5 TO 1.0
VERY STIFF	1.0 TO 2.0
HARD	> 2.0 OR 2.0+

RELATIVE DENSITY - GRANULAR SOILS

CONSISTENCY	N-VALUE (BLOWS/FOOT)
VERY LOOSE	0 TO 4
LOOSE	5 TO 9
MEDIUM DENSE	10 TO 29
DENSE	30 TO 50
VERY DENSE	> 50 OR 50+

DEGREE OF PLASTICITY OF COHESIVE SOILS

DEGREE OF PLASTICITY	PLASTICITY INDEX	SWELL POTENTIAL
NONE OR SLIGHT	0 TO 4	NONE
LOW	4 TO 20	LOW
MEDIUM	20 TO 30	MEDIUM
HIGH	30 TO 40	HIGH
VERY HIGH	> 40	VERY HIGH

MOISTURE CONDITION COHESIVE SOILS

DESCRIPTION	CONDITION
Absence of moisture, dusty, dry to touch	DRY
Damp but no visible water	MOIST
Visible free water	WET

CONSISTENCY OF COHESIVE SOILS AFTER TERZAGHI (1948)

CONSISTENCY	N-VALUE (BLOWS/FOOT)
VERY SOFT	< 2
SOFT	2 TO 4
FIRM	4 TO 8
STIFF	8 TO 15
VERY STIFF	15 TO 30
HARD	> 30

ABBREVIATIONS

HP - HAND PENETROMETER
TV - TORVANE
MV - MINIATURE VANE

UC - UNCONFINED COMPRESSION TEST
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL
CU - CONSOLIDATED UNDRAINED

NOTE: PLOT INDICATES SHEAR STRENGTH AS OBTAINED BY ABOVE TESTS

FINAL GROUND WATER LEVEL

INITIAL GROUND WATER LEVEL

CLASSIFICATION OF GRANULAR SOILS

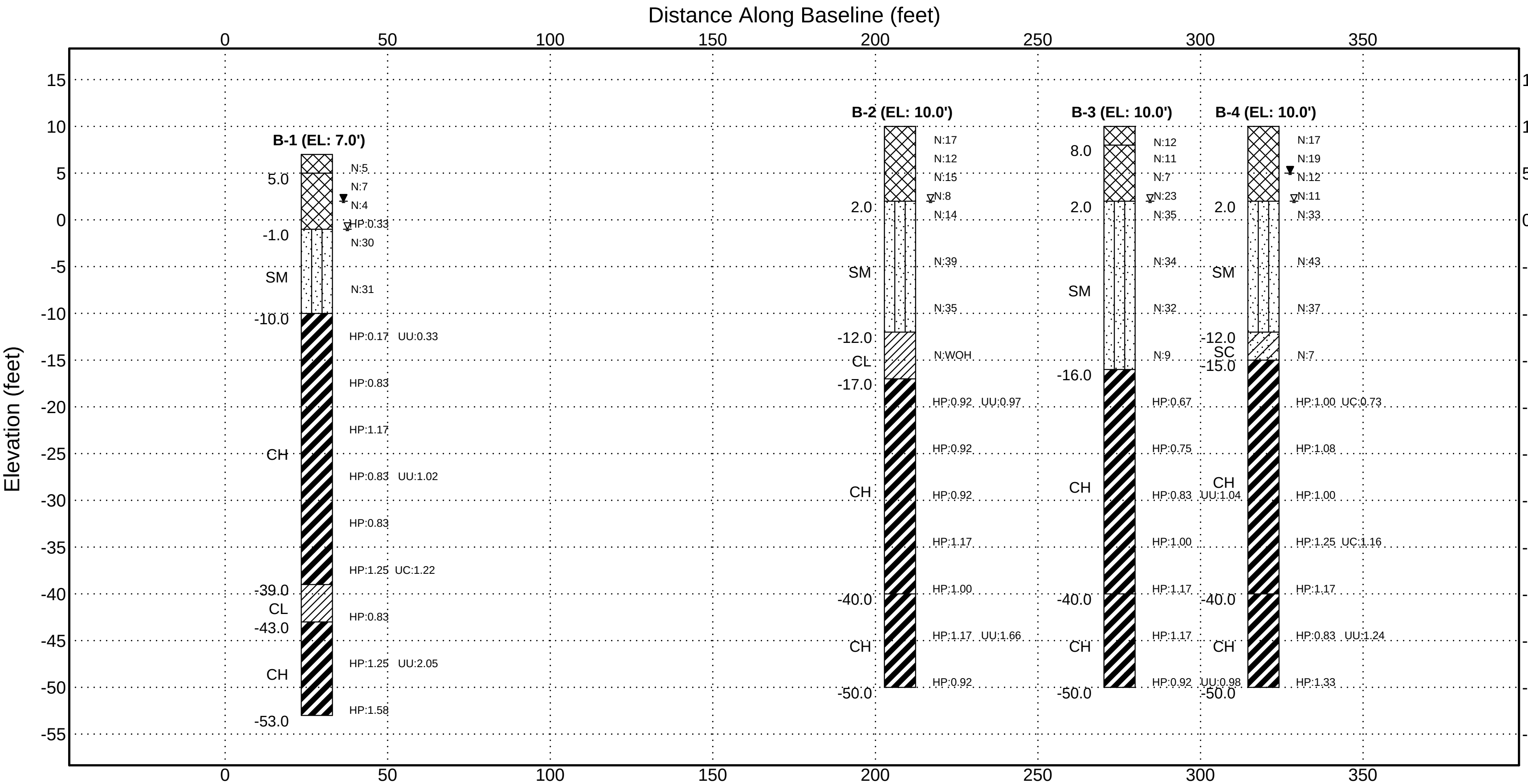
U.S. STANDARD SIEVE SIZE(S)

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

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 | 76.2 | 19.1 | 4.76 | 2.0 | 0.42 | 0.074 | 0.002

GRAIN SIZE IN MM



Stratifications shown are generalized and variations could occur in the field. The Hand Penetrometer (HP), Unconfined Compression (UC) values are shear strengths in tsf. N is blows per foot.

Fill Material | Silty Sand (SM) | Fat Clay (CH) | Lean Clay (CL) | Clayey Sand (SC)

STRATUM	START	END	STRATUM DESCRIPTION

GENERALIZED SUBSURFACE PROFILE

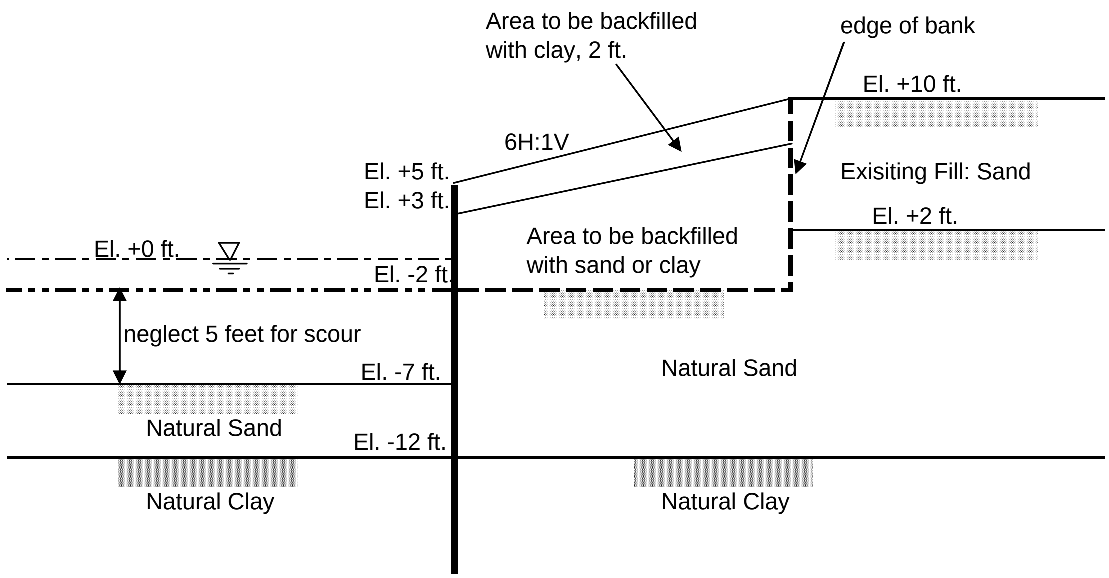
Surfside Pedestrian Bridge and Shore Stabilization

Surfside,Texas

PROJECT No.	DATE	PLATE
286-373	Dec 2010	8

SCHEMATIC FOR

CANTILEVERED SHEET PILE WALL



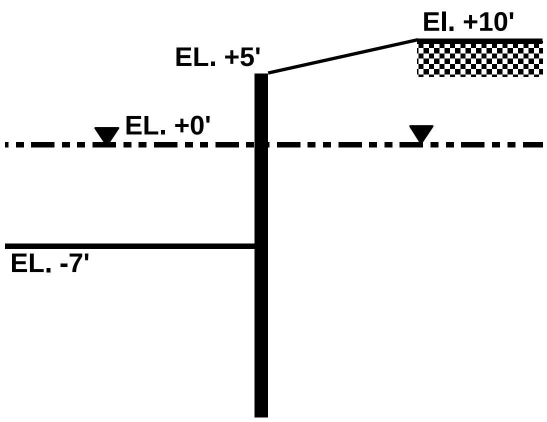
Shore Stabilization
Surfside, Texas
PSI REPORT 286-373

PLATE 9

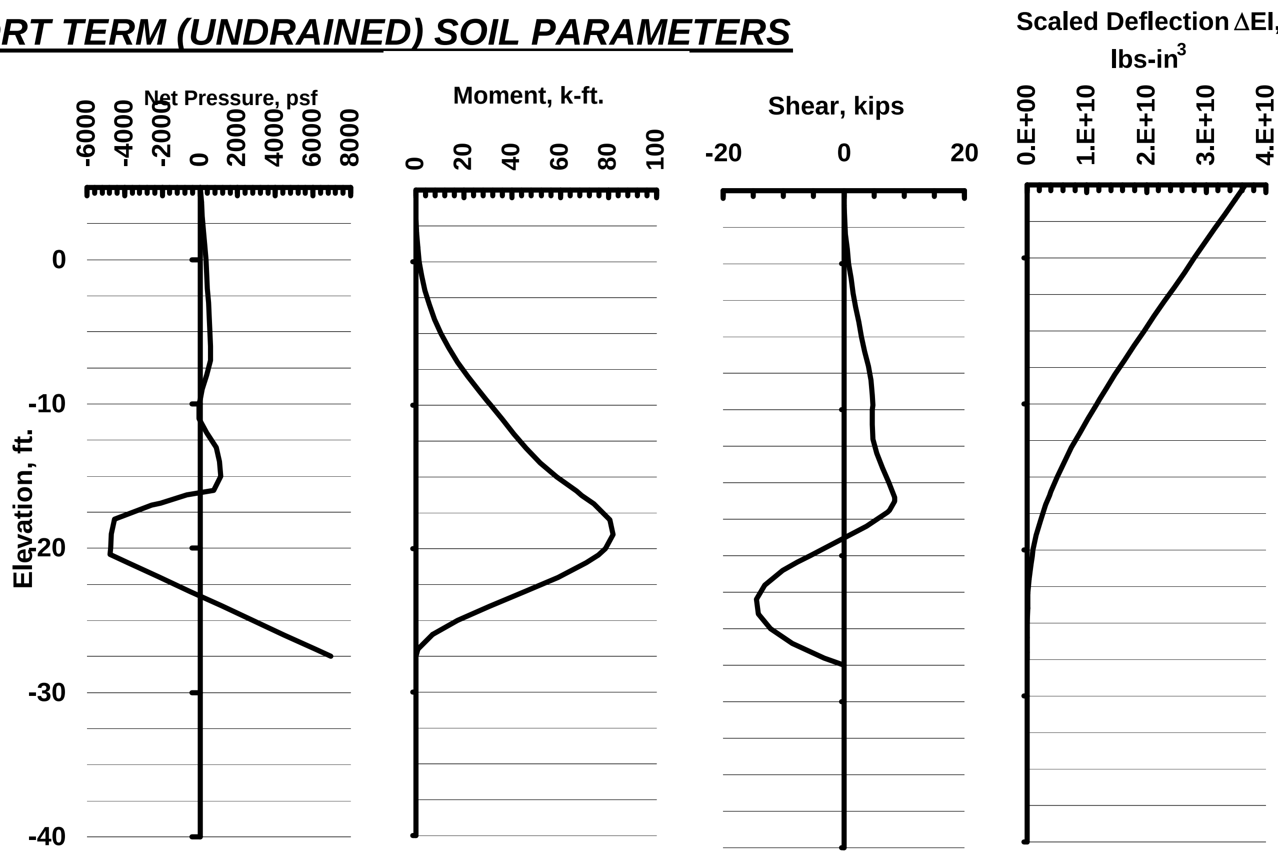
Geotechnical Consulting Services
Houston, Texas.

CANTILEVERED SHEET PILE WALL

SHORT TERM (UNDRAINED) SOIL PARAMETERS



Note: Not drawn to scale



Wall Bottom Elevation: EL. -27.5 ft.
Penetration Below El. -7': 20.5 ft.

Max. Moment: 82.0 kip-ft.
Occurs at: EL. -18.8 ft.

Max. Shear: -14.4 kips
Occurs at: EL. -23 ft

Max. Deflection*
3.66E+10 lbs-in^3

Note: **ΔEI, Scaled Deflection, where Δ is the deflection and EI is the flexural rigidity of the Sheet Pile**
The above numbers are for a Factor of Safety (F.S) of 1.0.
The penetration should be increased by 30%

Shore Stabilization: Surfside, Texas

PSI REPORT 286-373

PLATE 10

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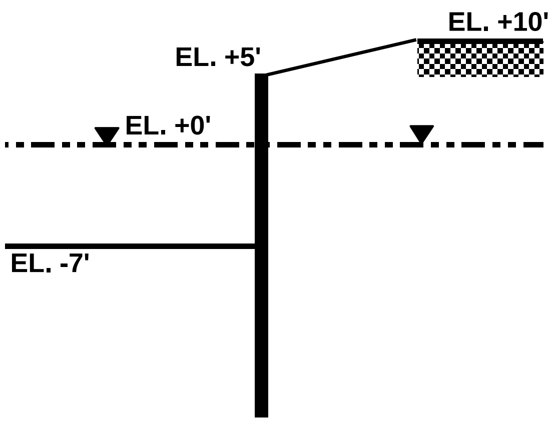
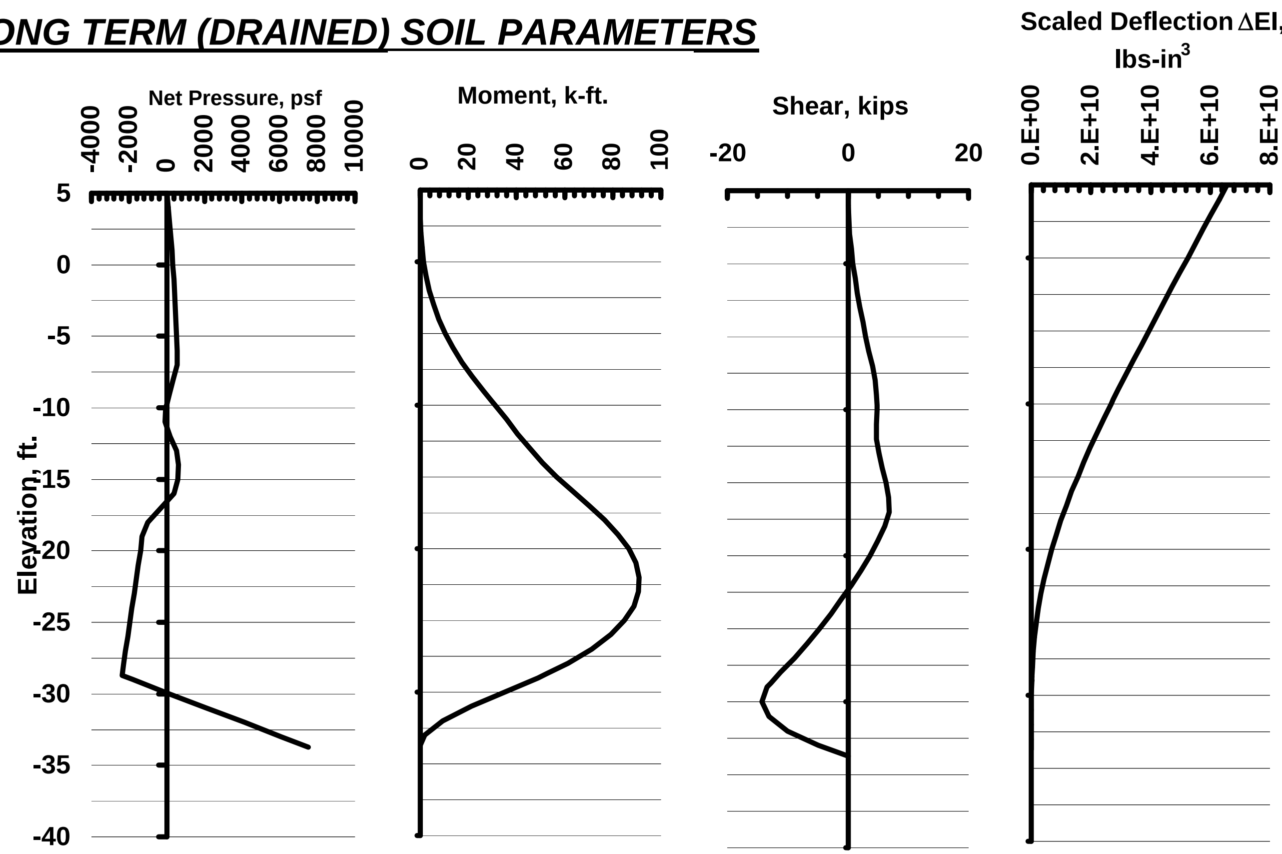
CANTILEVERED SHEET PILE WALL

LONG TERM (DRAINED) SOIL PARAMETERS





Wall Bottom Elevation: EL. -33.7 ft.
Penetration Below El. -7': 26.7 ft.

Max. Moment: 91.0 kip-ft.
Occurs at: EL. -22.4 ft.

Max. Shear: -14.3 kips
Occurs at: EL. -30 ft

Max. Deflection*
6.56E+10 lbs-in³

Note: ΔEI, Scaled Deflection, where Δ is the deflection and EI is the flexural rigidity of the Sheet Pile
The above numbers are for a Factor of Safety (F.S) of 1.0.
The penetration should be increased by 30%

Shore Stabilization: Surfside, Texas

PSI REPORT 286-373

PLATE 11

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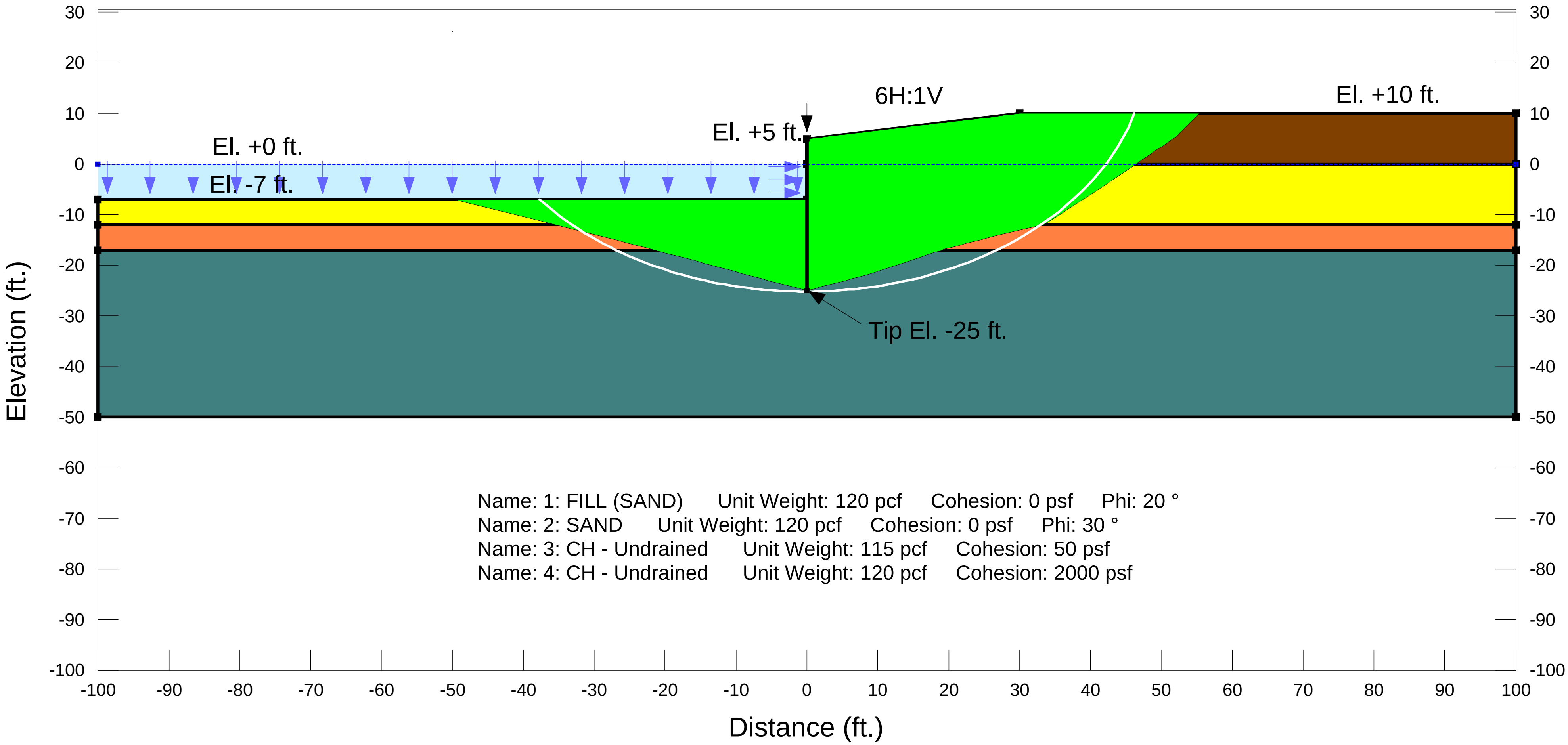
Surfside Slope Stabilization
Slope Stability Analysis
PSI Report 286-373

Short Term Conditions
Method: Spencer
FOS: 3.40



Surfside Slope Stabilization
Slope Stability Analysis
PSI Report 286-373

Long Term Conditions
Method: Spencer
FOS: 1.63

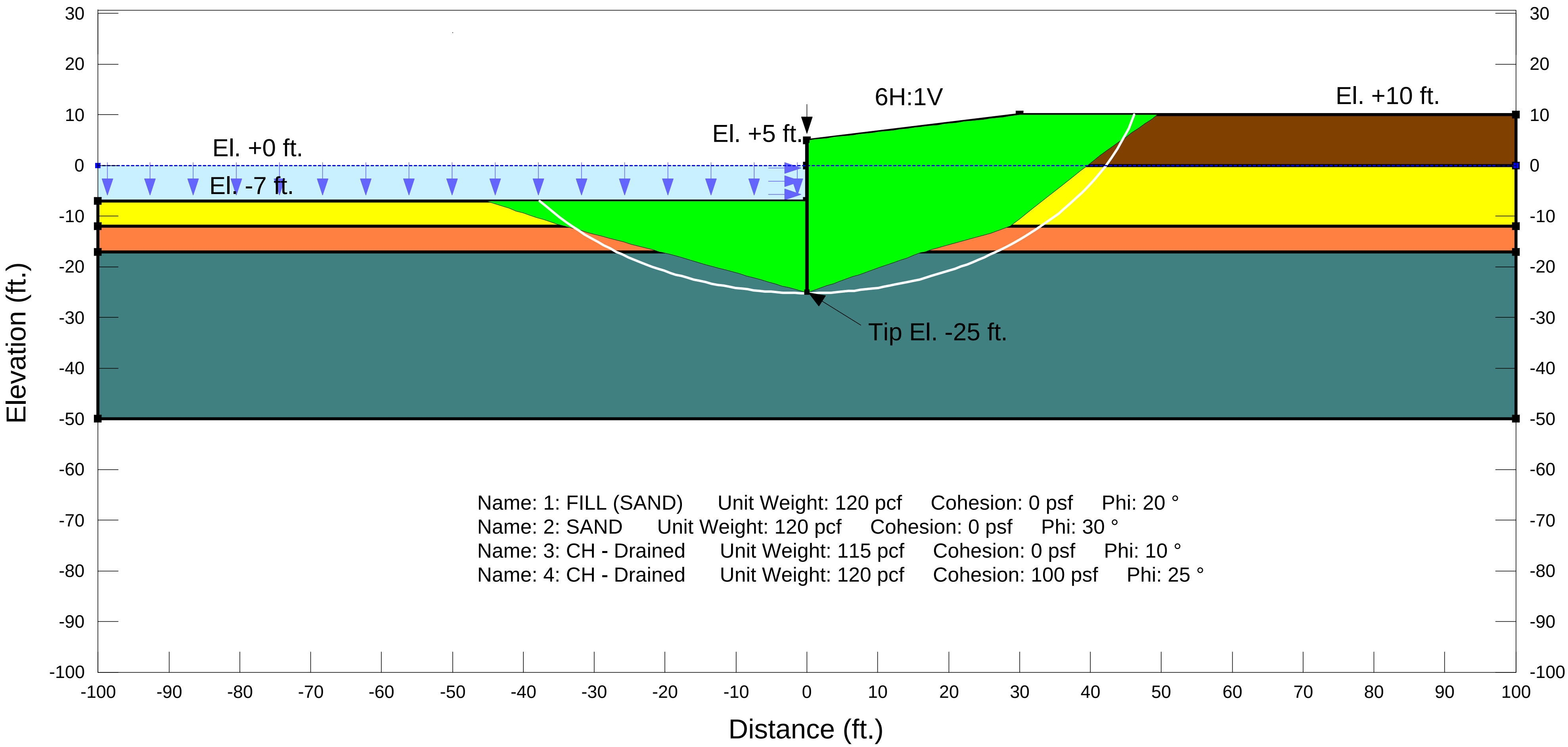


PLATE 13

Surfside Slope Stabilization
Slope Stability Analysis
PSI Report 286-373

Rapid Draw Down Conditions
Method: Spencer
FOS: 1.22

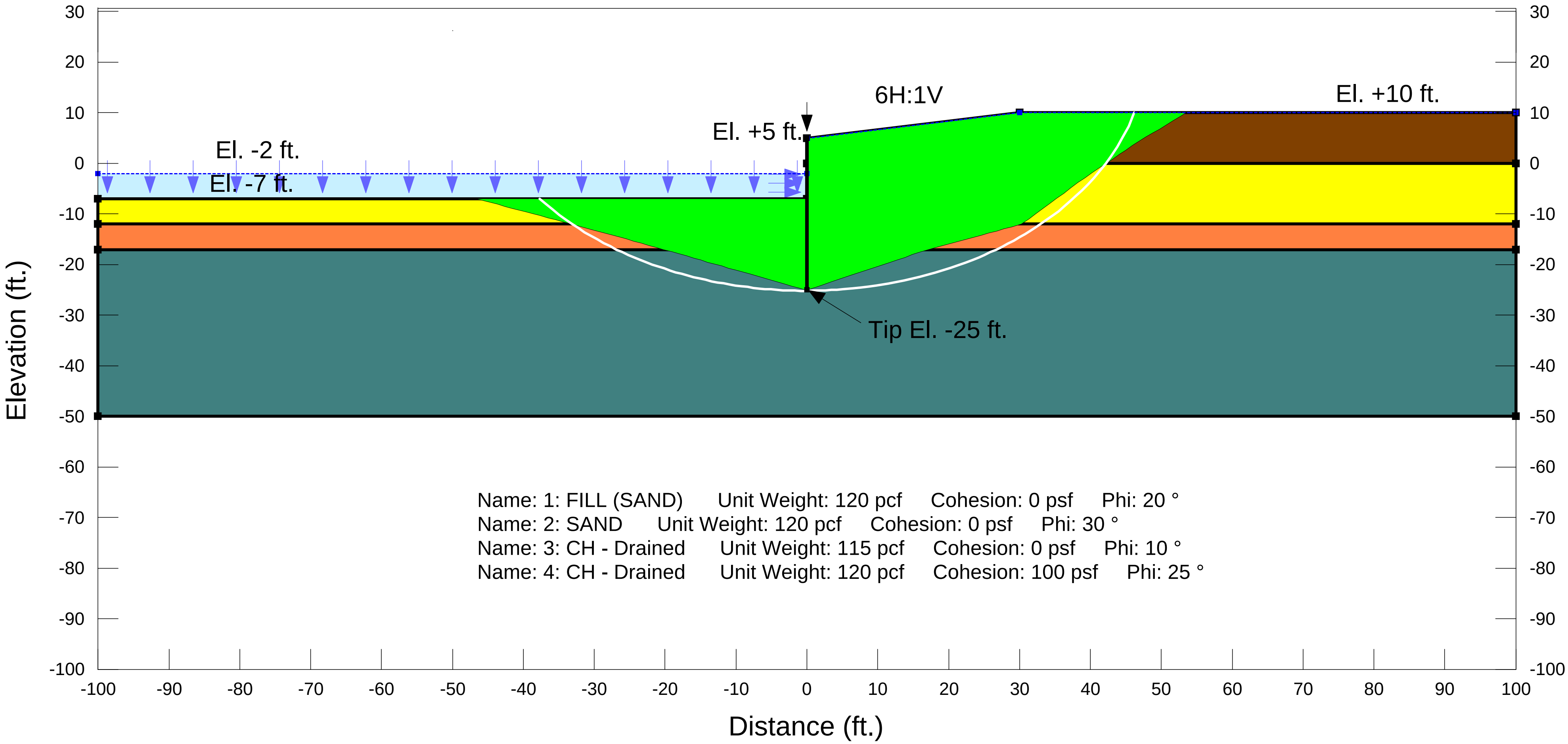


PLATE 14