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Sabine Pass Port Authority
TGLO/TDRA Round 1 Post-Ike Recovery Project
Jefferson County, Texas
Paving and Drainage Phase

ADDENDUM NO. 2

November 16, 2012

Item 1:

The following paving notes are hereby appended to the General Notes sheet of the Drawings. The referenced TxDOT standards supersede and replace the Asphalt Paving and Flex Base specifications contained in the bid package.

1. ALL PAVING TO BE DONE PER TEXAS DEPARTMENT OF TRANSPORTATION (TXDOT) STANDARDS.
2. SITE SHALL BE STRIPPED OF LOOSE TOPSOIL AND STABILIZED WITH HYDRATED LIME IN ACCORDANCE WITH THE REQUIREMENTS OF THE TXDOT STANDARD SPECIFICATIONS ITEM 264. LIME USED SHALL BE TYPE B -COMMERCIAL LIME SLURRY UNLESS OTHERWISE APPROVED BY OWNER.
3. APPLICATION AND MIXING OF LIME SHALL CONFORM TO TXDOT ITEM 260 FOR APPROPRIATE CURING TIME BETWEEN MIXES AND COMPACTION. APPROPRIATE COMPACTION AND MOIST CURING SHALL OCCUR PRIOR TO PLACING ADDITIONAL COURSES.
4. HMAC SHALL BE TYPE "D" AND THE FLEXIBLE BASE MATERIAL SHALL BE TYPE A, GRADE 2 AS DEFINED IN THE TXDOT STANDARD SPECIFICATIONS.
5. APPLICATION AND GRADING OF THE LIMESTONE COURSE MUST CONFORM TO TXDOT ITEM 247.
6. THE HMAC SURFACE COURSE SHALL BE COMPACTED TO CONTAIN 3 TO 9 PERCENT AIR VOIDS WHEN TESTED IN ACCORDANCE WITH TEX-207-F AND TEX-227-F.
7. STABILIZED SUBGRADE SHALL EXTEND A MINIMUM OF 12" BEYOND THE SUPPORTED PAVEMENT SECTION IN ALL DIRECTIONS.

Item 2:

Reinforced concrete pipe (RCP) is to be used for new drainage culverts indicated in the plans. References to new CMP drainage pipe in the bid form, drawings, and specifications are hereby deleted and replaced with RCP Class III. As a result of the change to RCP, the specification for the Precast Drainage Structures on pages C-1005 and C-1006 are changed to the following:

> “HANSON PRECAST CONCRETE TYPE E-3’ CURB INLET, TOTAL HEIGHT 5’-4” OR ENGINEER APPROVED EQUAL”

Item 3:

The following are Answers to Questions submitted by potential bidders.

Question 1:

Is there a geotechnical report?

A geotechnical report was prepared for a pile foundation at the SPPA site and is included in this Addendum as “Appendix A”.

Question 2:

What is the percentage of lime stabilization required?

The soil unit weight per Table 6-1 of the geotech report is 104 pcf (0.06 pci). Assume a 6% lime component for bidding purposes.

Question 3:

Is there water available on site for construction purposes?

The Sabine Pass Port Authority will not have water available for construction. Bidders may assume availability of City water via a fire hydrant, metered by the City and paid for by the Contractor per Appendix K, Section 17(e).

Question 4, Special Conditions

Sec. 1.7: Is there a list of permits the Contractor must obtain?

Previous project on the uplands portion of the site required the acquisition of an overall general building permit. A City of Port Arthur permit will likely be required for general construction and/or grading. This response is not intended to modify Appendix K, Section 17 (c), stating that Contractor is responsible for permits.

Sec. 1.8: Do I need Longshore or Maritime Insurance?

Yes, if Contractor intends to execute the work in a manner covered by laws regulating maritime employment. We believe that the work may be, but is not required to be executed without triggering this insurance requirement.

From the US Department of Labor:

> “You need longshore insurance if you are an employer with employees covered under the Longshore Act (LHWCS) and its extensions.”

From Technical Specifications Section 01 00 00 – Special Conditions:

> “Any employees who may fall under the Death on the High Seas Act, Jones Act, or any other federal or state acts relating to maritime employment must be covered by Maritime Employers Liability Insurance...”

Sec. 3.7: Provided by Owner

As discussed in the meeting, the survey control points shown on the drawing set are existing and provided by the owner. All other surveying activities, including the generation of final “As Built” elevations are the responsibility of the Contractor.

Sec. 3.9: Underground Telephone Not Shown; Conduit and Light poles within Area East of D-Dock Road.

Telephone cable location is uncertain. No utility work east of D-Dock is included in the project.

Question 5

In Addition to Utilities Shown:

At small slip, assuming new bulk-head work where will existing culvert at +/- 5+45 drain.

Main road, +/- 14+95, existing culvert will be within Cut., line is rusted steel with +/- 10" cover.

Dock road "A: +/- 4+50 (CL)and Rt existing culverts will be in Cut.(CL) culver +/- 8" HDPE, Rt Culvert +/- 12" steel, both have +/- 10" cover.

Existing Culvert at +/- 5+45 will drain ditch to small slip. Existing steel culvert at +/- 14+95 will remain, increase in road elevation from existing provides 14"+ of cover. Existing HDPE culvert at Dock Road A +/- 4+50 should be reburied with 14" of cover during paving operations.

Existing steel culvert that drains to Sabine Pass through the bulkhead shall remain, increased cover will be provided as part of the work specified in the grading plan.

Question 6

What is Ending Station for Main Road: Sheet C-1003 (Sta. 15+95), Sheet C-1004 (Sta. 16+95), Sheet C-1012 (Sta. 15+77)?

Sheet C-1003 is correct. Sta.15+95 is the ending station for the Main Road. Sta. 15+77 is the ending station for the ditch.

Question 7

What is the % of slope or upstream FE elevations for 18" C.M.P.?

The slope of the culvert invert will be approximately 1%, depending on punch out of drain box. See the note above (Item 2) about the change in the drain box specification.

Question 8

Will construction joint be allowed at the top of footing for walls, on proposed headwalls?

Yes.

Question 9

Type of material for proposed 6" water main pipe and bedding

Proposed 6" water main pipe should be PVC pipe with integral restrained joint system per AWWA C900 DR 18.

Question 10

Locations of vertical risers for 4" elect.

To be coordinated with local energy company, as described in the Construction Notes, Sheet C-1002.

Question 11

To allow for traffic assess, to work lime excavation, for ½ roadway, will require at least 1' beyond (CL), assuming vertical cut, traffic cones or barrels will be set leaving +/- 6' for traffic flow, one way.

Mixing of lime, along with depth of base and asphalt will create a depth of +/-25".

With mixer, compactors, etc. will possibly get into all existing and proposed lines. Area would need several days for installation and cure time.

Is there any option to lime subgrade, such as tensor with additional base depth, that may cut down time process?

The Bid Form has been revised to include an additional 400 SY of 8” flex base that will serve as construction material for use in construction additional road access outside of the paved surface as necessary to maintain access.

Only the road work between stations 0+00 and 6+50 on South First Ave is expected to require the installation of this additional material to maintain continuous access. This quantity includes the necessary work for placement such that a continuous surface is installed.

Sequencing of the repaving work will be developed in coordination with Sabine Pass Port Authority and Jefferson County Sheriffs personnel access requirements after contract award and shall be submitted by contractor for approval.

For bidding purposes, there in not an option to the lime stabilized subgrade. Engineer will review all substitution submittals made by Contractor.

Question 12

Will fill or other exposed areas require hydro mulch seeding?

No.

Question 13

Will area of high weeds along ditches and to property line along east property require mowing or clear and grub?

The area east of D Dock Road need not be addressed. Ditches not subject to clean out or grading as part of the work need not be addressed.

Question 14

Since topo elevations were taken in 2009 and 2010, will updated elevations for verification be required?

The design drawings indicate final lines and grade elevations required for paving and drainage construction. Contractor should not rely on prior topo to achieve final lines and grades.

Question 15

Are well points required or will pumping be acceptable?

There is no requirement for Contractor’s construction methods.

Question 16

Is this Certified Payroll?

Yes.

Revised Bid Form

Base Bid - Paving and Drainage Improvements

Bid Item	Description	Estimated Qty's	Units	Unit Price	Total
01 73 20 – Selective Demolition					
17320-1	Demo existing culvert structures and associated miscellaneous material and components	1	LS		
17320-2	Demo existing road, road base and associated miscellaneous material and components	7660	SY		
03 30 00 – Cast In Place Concrete					
033000-1	Culvert Headwalls	7	CY		
03 41 00 – Pre-Cast Concrete					
034100-2	Culvert Inlet	2	EA		
05 50 10 – Corrugated Metal Pipe					
055010-1	18" Aluminum Pipe	80	LF		
31 11 00 – Clearing and Grubbing					
311100-1	Clearing and Grubbing upland site elements	5000	SY		
311100-2	Grading	11960	SY		
311100-3	Ditch Clean Out	510	CY		
31 23 00 – Excavation and Fill					
312300-1	Structural Backfill - Culverts	8	CY		
32 12 16 – Asphalt Paving					
321216-1	3" Road Surface	1174	Ton		
32 12 20 – Flex Base					
321220-1	**8" Road Base**	**400**	**SY**		
321220-2	14" Road Base	7660	SY		
321220-3	8" stabilized subgrade	7660	SY		
33 06 41 Pipe and Appurtenaces					
330641-1	6" dia. Water Line	425	LF		
				Base Bid Subtotal:	___________
01 71 13 – Mobilization and Demobilization					
017113-1	Mob / Demob	1	LS		
				Base Bid Total:	___________

Additive Bid 1 - Buried Electrical Service

Bid Item	Description	Estimated Qty's	Units	Unit Price	Total
26 05 43 - Underground Ducts and Raceways for Electrical Systems					
260543-1A	Install buried 4" dia. Conduit	1090	LF		
01 71 13 – Mobilization and Demobilization					
017113-1A	Mob / Demob	1	LS		
				Additive Bid 1 Total:	___________

Additive Bid 2 - Buried Electrical Service

Bid Item	Description	Estimated Qty's	Units	Unit Price	Total
26 05 43 - Underground Ducts and Raceways for Electrical Systems					
260543-1B	Install buried 4" dia. Conduit	200	LF		
01 71 13 – Mobilization and Demobilization					
017113-1B	Mob / Demob	1	LS		
				Additive Bid 2 Total:	___________

Tolunay-Wong Engineers, Inc.

GEOTECHNICAL ENGINEERING STUDY
REGIONAL MARINE SECURITY CENTER
SABINE PASS, TEXAS

Prepared for:

Leap Engineering, LLC
550 Fannin, Suite 510
Beaumont, Texas 77701

Prepared by:

Tolunay-Wong Engineers, Inc.
2455 W Cardinal Drive, Suite A
Beaumont, Texas 77705
(409) 840-4214

January 26, 2010

Project No. 10.23.002 / Report No. 27759

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January 26, 2010

Leap Engineering, LLC
550 Fannin, Suite 510
Beaumont, TX 77701

Attn: Mr. Robert Hickman, P.E.

Re: Geotechnical Engineering Study
Regional Marine Security Center
Sabine Pass, Texas
TWE Project No: 10.23.002 / Report No: 27759

Dear Mr. Hickman,

Tolunay-Wong Engineers, Inc. is pleased to submit this report of our geotechnical study for the above referenced project. This report contains a detailed description of the field and laboratory work performed for this study, as well as soil boring logs including tabulated laboratory test results. Also included in this report are soil parameters for sheet pile design and recommendations for deep foundation design.

We appreciate the opportunity to work with you on this phase of the project, and look forward to the opportunity to provide additional services as the project progresses. If you have any questions regarding the report or if we can be of further assistance, please contact us.

Sincerely,
TOLUNAY-WONG ENGINEERS, INC.
(TX Firm Registration No. F-000124)

STATE OF TEXAS
PATRICK J. KENNEY
87894
LICENSED
PROFESSIONAL ENGINEER

Patrick J. Kenney, P.E.
Vice President – Southeast Region

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1 INTRODUCTION AND PROJECT DESCRIPTION

1.1 Introduction

This report presents the results of the geotechnical study for the proposed installation of a new steel sheet pile bulkhead on the Sabine Ship Channel in Sabine Pass, Texas.

This study was conducted in general accordance with TWE Proposal P09-B271 dated December 10, 2009, and authorized by Mr. Robert Hickman, P.E. on 12/29/09.

1.2 Project Description

We understand that the project will consist of the construction of a new sheetpile bulkhead system and a single-story structure. The bulkhead will be approximately 500-feet in total length. The bulkhead will be a U-Shaped anchored sheet pile system tied-back to anchor piles or other tieback system. The maximum wall height will be approximately 15-feet. We have been requested to provide geotechnical design parameters needed for analysis of the sheet pile wall and anchor system to be performed by the client. Axial capacity has been requested for driven piles to support the proposed building. Lateral analyses of driven piles will be performed by the client based on the geotechnical design parameters provided in this report.

2 PURPOSE AND SCOPE OF SERVICES

The purposes of the geotechnical study were to investigate the soil and groundwater conditions and to interpret this data to develop geotechnical design parameters for proposed sheet pile bulkhead and building foundations. The scope of services for this project consisted of:

- Drilling one (1) soil test boring to a depth of one-hundred twenty (120) feet at a selected location within the project area to evaluate subsurface stratigraphy and groundwater conditions.

- Performing geotechnical laboratory tests on recovered soil samples to evaluate the physical and engineering properties of the strata encountered.

- Preparation of a report documenting the findings of this investigation and presenting geotechnical engineering design parameters for sheet pile design and recommendations for deep foundation design.

Environmental assessments, a geologic fault study, and recommendations for areas outside the area covered by the project-boring layout were beyond the scope of this study.

3 FIELD EXPLORATION

3.1 Test Borings

Tolunay-Wong Engineers, Inc. conducted an exploration of subsurface soil and groundwater conditions at the proposed project site on January 8, 2010 by drilling one (1) soil test boring to a depth of one-hundred twenty (120) feet below ground surface.

The boring location is shown on Drawing 10.23.002-01 attached to this report. Drilling, sampling and grouting of the test boring was performed by using an all-terrain buggy mounted drill rig. Our field personnel coordinated the field activities and logged the boreholes.

3.2 Drilling Methods

Field operations were performed in general accordance with *Standard Practice for Soil Investigation and Sampling by Auger Borings* [American Society for Testing and Materials (ASTM) D 1452]. Soil borings were drilled using a buggy drilling rig equipped with a rotary head. Boreholes were advanced using dry-auger and wet-rotary drilling methods. Typically, borings are dry-augered using a flight auger to advance the boreholes until groundwater is encountered or until the borehole becomes unstable and collapses. At that point, the borings are completed using wet-rotary drilling techniques. Samples were obtained continuously at intervals of 2-feet from the ground surface to a depth of 12-feet, at the 13-feet to 15-feet depth interval and then at intervals of 5-feet to boring completion depth.

3.3 Soil Sampling

Cohesive/semi-cohesive soil samples were recovered from the test borings by hydraulically pushing a 3-in. diameter, thin-walled tube a distance of about 24 inches. The field sampling procedures were conducted in general accordance with the *Standard Practice for Thin-Walled Tube Sampling of Soils* (ASTM D 1587). The field technician visually classified the recovered soils, and obtained a penetration resistance measurement of the recovered soils using a calibrated pocket penetrometer. A factor of 0.67 is typically applied to the penetrometer measurement to estimate the undrained shear strength of the Gulf Coast cohesive soils. The samples were extruded in the field, sealed and placed into secure containers, protected from disturbance, and transported to the laboratory. The recovered soil sample depths and pocket penetrometer measurements are shown on the test boring logs in Appendix A.

Cohesionless sands and semi-cohesionless silts, and soil samples inferred to be granular were collected with the Standard Penetration Test (SPT) sampler driven 18in. by blows from a 140 pound hammer falling 30-inches (ASTM D1586). The number of blows required to advance the sampler three consecutive 6 in. depths are recorded for each corresponding sample on the boring log. The N-value, in blows per foot, is obtained from SPT by adding the last two blow count numbers. The compactness of the cohesionless/semi-cohesionless samples and the consistency of the cohesive samples are inferred from the N-value. The samples obtained from the split barrel sampler were visually classified, sealed in plastic bags, and transported to our laboratory. The SPT sampling intervals and blow counts are presented on the boring logs in Appendix A.

3.4 Boring Logs

Our interpretations of general subsurface soil and groundwater conditions at the boring locations are included on the boring logs. The interpretations of the soil types throughout the boring depth and the locations of strata changes were based on visual classifications during field sampling and laboratory testing using ASTM D 2487, Unified Soil Classification System, and ASTM D 2488, Description and Identification of Soils. The boring logs include the type and interval depth for each sample along with the corresponding pocket penetrometer readings for cohesive soils. The project boring logs and a key to the terms and symbols used on the logs are presented in Appendix A.

3.5 Groundwater Measurements

Boring B-1 was dry augered in an attempt to measure groundwater levels. Water was encountered in the test boring at a depth of 8-feet. Static water level was not measure due to them hole squeezing at a depth of 6-feet after ten minutes. It should be noted that the groundwater level may fluctuate with climatic and seasonal variations and should be verified before construction. In addition, groundwater level in cohesive soil is time dependent.

Accurate determination of the static groundwater level is usually made with a standpipe piezometer. Installation of a piezometer to evaluate the long-term groundwater level was not included in the work scope.

4 LABORATORY TESTING

A laboratory testing program was conducted on selected samples to assist in classification of the soils encountered in the borings, and to evaluate the engineering properties of the soils pertinent to the deep foundation design parameters for this project.

4.1 Soil Classification Tests

All samples obtained during the field program were visually classified in the laboratory according to procedures outlined in ASTM D 2488. In addition, tests for natural moisture content, Atterberg Limits, and particle size analysis were conducted on selected samples obtained from the borings. These laboratory test results were used to classify the soils encountered in general accordance with the Unified Soil Classification System (ASTM D 2487). Results of the classification tests are presented on Boring Log, B-1in Appendix A.

4.2 Soil Strength Tests

The approximate undrained shear strength of selected samples of cohesive soils obtained in the borings was determined by performing unconfined compression (UC) tests. Natural moisture content and dry unit weight was determined for each sample tested for shear strength. Results of the UC tests are presented on Boring Log, B-1in Appendix A.

4.3 Laboratory Procedures

Laboratory tests were performed in general accordance with ASTM Standards to measure physical and engineering properties of the soil samples obtained for this project. The types of laboratory tests performed are presented in Table 4-1.

Table 4-1
Laboratory Testing Program

Type of Test	Testing Method
Natural Water Content	ASTM D 2216
Atterberg Limits	ASTM D 4318
Material Passing Sieve No. 200	ASTM D 1140
Dry Unit Weight	ASTM D 2937
Unconfined Compression	ASTM D 2166

The tests results are shown on the boring logs in Appendix A.

5 SITE CONDITIONS

5.1 General

Our interpretations of soil and groundwater conditions at the site are based on information obtained at the soil boring location only. The project boring log is presented in Appendix A. This information has been used as the basis for our conclusions and recommendations. Subsurface conditions may vary at areas not explored by the project soil borings. Significant variations at areas not explored by the project borings will require re-evaluation of our recommendations.

5.2 Subsurface Soil Stratigraphy

The soil profile, as interpreted from the project boring B-1, consists of soft to very soft clays from the ground surface to a depth of 73-feet. Clayey sands and poorly graded sands with clay were encountered from 73-feet to 93-feet. Stiff to very stiff clays were encountered below the sand strata from 93-feet to boring completion depth of 120-feet. The cohesive soils were comprised of soft to very stiff, high plasticity fat clays. Ferrous nodules, sand seams, silt pockets, shell fragments, wood, and slickensided substructure were observed within the clay soil matrix.

The upper 12 to 24-inches of soils observed in the project boring was described as fill on the boring log. The fill consisted of fat clay with base material. In practice, it is relatively difficult to delineate fill from adjacent natural soil. Fill identification is based on visual observation and requires considerable experience and the use of judgment. Actual fill depths may vary somewhat from those indicated on the boring logs.

A detailed description of the soils encountered at the boring location is presented on the boring log included in Appendix A.

5.3 Subsurface Soil Properties

We measured liquid limits of 52 to 94, and corresponding plasticity indices of 33 and 66 on seven selected cohesive soil sample recovered from various depths in the project borings. In situ moisture contents of the samples were four to fifty-one percentage points greater than their corresponding plastic limits, indicating a relatively wet condition at the time of the field investigation. Fines contents ranging from 6% to 27% were determined on selected cohesionless material in the project boring.

Undrained shear strengths ranging from 430 psf to 1,810 psf were measured on cohesive samples recovered at various depths in the project boring during unconfined compression testing. Corresponding dry unit weights of the tested samples were 54 pcf and 87 pcf. SPT N-values of 1 and 3 blows per foot were registered within the fat clays at a depth range of 13-ft to 50-ft. Pocket penetrometer readings taken on recovered cohesive soil samples ranged from 0.25 tsf to 4.25 tsf.

The cohesionless poorly graded sands with clay at the depth range of 79-ft to 93-ft recorded N-values of 50 blows per foot and greater, indicative of very dense compactness. Selected clayey sand and poorly graded sand with clay recovered from the project boring had fines contents of 27% and 6%.

6 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 General

As previously mentioned, this project consists of a sheet pile bulkhead tied-back to anchor piles or other tieback system. We have been requested to provide geotechnical design parameters needed for analysis of the sheet pile wall and anchor system to be performed by the client. The project will also include a single-story light framed structured supported on driven piles. Axial capacity has been requested for driven piles. Lateral analyses of driven piles will be performed by the client based on the geotechnical design parameters provided in this report.

6.2 Geotechnical Design Parameters

Soil parameters for analysis and design of sheet pile as well as deep foundations (axial and lateral) were developed based on the subsurface data obtained from this investigation.

For the conditions observed at this site, we recommend the following soil parameters be used for sheet pile analyses as well as for axial and lateral analysis of pile foundations.

Table 6-1

GEOTECHNICAL DESIGN PARAMETERS FOR SHEET PILE AND DEEP FOUNDATION DESIGN					
Depth Range	**LPILE Soil Type**	**Shear Strength C (psi) or Φ**	**Unit Weight, pci**	**Lateral Modulus, k, pci**	**Strain Factor, E_{50}**
0' – 8'	Soft Clay (Matlock)	1.74	0.060	30	0.020
8' – 25'	Soft Clay (Matlock)	1.74	0.024	30	0.020
25' – 43'	Soft Clay (Matlock)	2.78	0.020	30	0.020
43' – 63'	Soft Clay (Matlock)	3.00	0.020	30	0.020
63' – 78'	Soft Clay (Matlock)	4.17	0.021	100	0.010
78' – 93'	Sand (Reese)	**Φ** = 42°	0.039	125	--
93' – 110'	Stiff Clay with Free Water	12.50	0.032	800	0.007
110' – 120'	Stiff Clay with Free Waer	5.07	0.037	300	0.010

6.3 Driven Pile Foundation Design

6.3.1 Axial Pile Capacity

We have developed unit friction and end bearing capacity curves for calculating allowable pile capacity for use with driven piles for deep foundations in the areas of the proposed new building. If open-ended pipe piles are going to be considered for this project, TWE should be contacted to provide specific pile capacity for the size and length of open-ended steel pipe pile proposed. Design factor curves (F and E) are provided for driven piles on Sheet B-1in Appendix B. Example calculations illustrating the proper use of these curves are provided on Sheet B-1. The unit friction (F) and end bearing (E) curves include a minimum factor of safety of 2.0. The values presented are based on the assumption that the piles to be installed will have a minimum center-to-center spacing of three pile diameters. If groups of piles having spacing of less than three diameters are designed for this project, Tolunay-Wong Engineers, Inc. should be contacted to analyze group capacities and settlements.

The pile capacity curves presented are also based on the assumption that less than 2 feet of fill will be placed above grade in the vicinity of the pile foundations. If new fill is placed to raise the site grade above the existing elevation, significant settlement will occur as the soft to very soft clays consolidate. Depths for driven piles will depend on the design loads and required pile capacities, however, we recommend that the piles be tipped in the competent sand stratum encountered in the boring at a depth of approximately 75 feet. The recommended minimum pile length for this project is 80 feet. Pile capacities will also be dependent on the amount of fill placed above grade at the location of the pile foundation. Negative skin friction may be caused by placement of sufficient quantities of fill such that the overburden pressure exerted by the fill exceeds the preconsolidation pressure of the underlying soft to very soft clays resulting in consolidation of the compressible clays. Negative skin friction is a downward shear drag acting on piles due to downward movement of surrounding soil strata relative to the piles. Depending on the quantities of fill and corresponding overburden pressure, this load can become large and must be considered in the design of pile foundations for this project. If more than 2 feet of fill above grade will be required in the vicinity of the planned pile foundations, TWE should be contacted to re-evaluate pile capacity and settlement based on the proposed construction.

Some general guidelines for estimating group pile capacities are provided in Section 6.3.3 of this report. It should be noted that the tension capacity is based solely on soil-pile interaction. Piles and pile cap connections should be structurally capable of resisting design uplift loads.

For single isolated piles, designed in accordance with the computed allowable values of side friction and end bearing, foundation settlements should be less than about ½ inch.

6.3.2 Lateral Pile Capacity

For deep foundations, the lateral loads are resisted by the soil as well as the rigidity of the pile. Lateral capacity will vary with pile type and properties, degree of fixity and pile spacing. The table provided in Section 6.2 of this report contains design parameters which can be used for lateral analyses. We understand that these analyses will be performed by the client.

6.3.3 Pile Groups

As indicated above, groups of piles having a center-to-center spacing of less than three diameters should be analyzed for group efficiency. If pile groups are planned for this project, Tolunay-Wong Engineers, Inc. should be contacted to analyze group capacities and settlements once the final pile size, depth and group configurations are selected. Some general guidelines for estimating group pile capacities are provided below.

6.3.3.1 Pile Settlement and Spacing

Vertical movement (settlement) of individual piles when subjected to structural loading will be the sum of elastic pile deformation and pile tip movement. Settlement of pile groups will depend on individual pile movements, pile spacing and the compressibility of the soils below the pile tips. Pile spacing is important in reducing pile group movement. A minimum pile spacing of three pile diameters, center-to-center, is assumed and should be maintained if possible. Closer spacing could result in increased group settlement and a reduction of load-carrying capacity of individual piles as indicated below.

6.3.3.2 Axial Group Efficiency

The following method can be used to determine the axial capacity of pile groups. This method assumes that the piles and confined soil mass encompassed by the group act as a unit like a pier. The ultimate bearing capacity of the cluster, Q_c, is equal to the ultimate load carried in friction by the circumferential area of the group plus the ultimate load resistance derived from the base of the assumed equivalent pier. In equation form:

$$Q_c = f_s A_c + 9 C_u A_b$$

Where:

- f_s = ultimate unit soil-pile adhesion
- A_c = circumferential embedded area of equivalent pier
- C_u = soil shear strength at pile tips
- A_b = base area of equivalent pier

The pile group is considered safe against a bearing failure if the number of piles in the group times the applied design load per pile does not exceed Q_c/F.S. If the total group design load is greater than Q_c/F.S., then one alternative is to reduce the design load for individual piles within the group accordingly. Based on this approach to pile group capacity analysis, a pile spacing can be determined which utilizes the full capacity of individual piles. Generally, a pile spacing of three (3) pile diameters, center-to-center, is selected as a first approximation.

Total settlements of the group, primarily elastic in nature, will occur during loading and may be on the order of one-half (½) to one (1) inch for normal operating conditions. Differential settlements between adjacent groups may occur as a result of variation in applied load, group size and group location. Structural connections also supported on adjacent pile foundations may be designed for differential settlements between adjacent pile groups on the order of one-half (½) to three-fourths (¾) inch.

6.3.3.3 Lateral Group Effect

The reduction of the lateral pile capacity due to group action involves factors such as pile spacing, location of the pile within the group, soil to pile stiffness ratio, direction of loading and other factors. When the lateral load has been selected for design purposes, group reductions can be estimated by using the following lateral group efficiency factors.

Static Lateral Group Efficiency Factors	
S/D (Center to Center Spacing/Diameter)	**Group Efficiency**
3	0.55
3.5	0.65
4	0.75
5	0.85
6	1.0

The group lateral efficiency factors above should be applied as follows:

Allowable lateral load of pile group = (N)(GE)(SPALL)

Where:

N = Number of piles in group

GE = Group efficiency factor

SPALL = Single pile allowable lateral load

The above procedure for determining lateral group reduction is considered to provide a general estimate of group efficiency. A more detailed approach to determining the lateral grouping effects is provided in *"Analysis and Design of Shallow and Deep Foundations"* by Lymon C. Reese, William M. Isenhower, and Shin-Tower Wang (2006 edition). Article 15.5.3 of this publication describes a method in which the *p-y* curves for a single pile are modified to take into account the group effect. This article concludes that the group effect could be taken into account most favorably by reducing the value of *p* for the *p-y* curve of the single pile to obtain *p-y* curves for the pile group. The L-Pile computer program provides a mechanism whereby the *p-y* modification factor can be included in the input file. The *p-y* modification factor is calculated based on the number of piles in the group, pile spacing, pile diameter, location of the pile to be analyzed within the group and the direction of the horizontal loading on the group with respect to the group geometry. This method is considered to provide more realistic estimates of lateral group effects than the general procedure provided above.

6.4 Driven Pile Installation

Pile driving hammers should be selected according to pile type, length, size and weight of pile, as well as potential vibrations resulting from pile driving operations. Care should be taken to assure that the hammer selected is capable of achieving the desired penetration without causing damage to the piles or causing excessive vibrations which could damage existing, nearby structures.

Each pile should be driven to the desired tip elevation and driving resistance without interruption in the driving operations. Supplemental techniques like pilot holes or jetting are not considered necessary for this project based on the soils encountered and design pile capacities, and should be avoided. The supplemental techniques may reduce the pile capacity. Driving of the center piles in the cluster first will facilitate driving operations. Accurate records of the final tip elevation and driving resistances should be obtained during the pile driving operations.

Some pile heaving may be experienced during installation of adjacent displacement type piles. It is therefore recommended that the tip elevation of the piles be recorded and if significant heave is noted after driving of subsequent piles, provisions must be made for reseating them.

It is important that inspection of pile driving by qualified geotechnical technicians be maintained so as to detect unexpected conditions as indicated by the driving resistance as well as any potential problems with pile breakage or driving difficulties.

6.5 Pile Load Tests

It is recommended that the computed pile capacities be verified by field load tests. Since both axial and lateral loads are significant for this project and are both critical to foundation design, we recommend that piles be tested for both axial and lateral capacity. Axial and lateral load tests should be performed in accordance with the following ASTM procedures:

1. ASTM D 1143: Standard Test Method for Piles Under Static Axial Compressive Load

2. ASTM D 3689: Standard Test Method for Individual Piles Under Static Axial Tensile Load

3. ASTM D 3966: Standard Test Method for Piles Under Lateral Loads

For compression tests, the pile should be taken to the ultimate load or failure load. The failure load can be defined by the Davisson Offset Method which is based on pile top deflection exceeding an offset to the theoretical elastic pile deflection line. This method should carry the load to not more than 250 percent of the design load on the test pile. This test should be conducted prior to installation of production piles to establish the installation criteria and to confirm the design load.

6.6 Dynamic Pile Testing

We recommended that the computed pile capacities be further verified by performing Dynamic Pile Testing as a quality assurance tool during construction.

Dynamic Pile Testing is a high-strain testing process based on the theory of Stress Wave Propagation on Piles from the impact of a hammer blow to the pile. Dynamic pile testing can be used to evaluate the bearing capacity of driven piles. This technology has been used in the deep foundation industry for more than 30 years and the process is officially recognized by numerous organizations including the American Society for Testing Materials (ASTM D 4945) as well as FHWA, AASHTO, and the U.S. Army Corps of Engineers among others. The procedure involves accelerometers and strain transducers which are attached to the pile. For each impact by the pile driving hammer or drop weight, the sensors acquire acceleration and strain signals and send them to the Pile Driving Analyzer (PDA). The PDA conditions, digitizes, displays and stores the signals and performs automatic calculations. Dynamic Pile Monitoring is typically conducted during the impact driving of steel, concrete or timber piles to determine soil resistance to driving, hammer performance, dynamic pile stresses during driving and pile integrity. Dynamic pile load testing can also be performed on straight-sided drilled shafts or augercast piles using a drop weight device designed for this purpose after the shafts/piles have been installed. Results are obtained in real time. Tolunay-Wong Engineers, Inc. would be pleased to develop a plan for foundation monitoring and testing to be incorporated in the overall quality control program.

7 LIMITATIONS AND DESIGN REVIEW

7.1 Limitations

This report has been prepared for the exclusive use of Leap Engineering, LLC and their design team for specific application to the construction of the Regional Marine Security Center in Sabine Pass, Texas. Our report has been prepared in accordance with the generally accepted geotechnical engineering practice common to the local area. No other warranty, express or implied, is made.

The analyses and recommendations contained in this report are based on the data obtained from the referenced subsurface exploration. The borings indicated subsurface conditions only at the specific locations and times, and only to the depths penetrated. The borings do not necessarily reflect strata variations that may exist at other locations within the project site. The validity of the recommendations is based in part on assumptions about the stratigraphy made by the Geotechnical Engineer. Such assumptions may be confirmed only during earthwork and foundation installation. Our recommendations presented in this report must be re-evaluated if subsurface conditions during construction are different from those described in this report.

If any changes in the nature, design, or location of the project are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed, and the conclusions modified or verified in writing by TWE. TWE is not responsible for any claims, damages, or liability associated with interpretation or reuse of the subsurface data or engineering analyses without the expressed written authorization of TWE.

7.2 Design Review

Review of the design and construction drawings as well as the specifications should be performed by TWE before release. The review is aimed at determining if the geotechnical design and construction recommendations contained in this report have been properly interpreted. Design review is not within the authorized scope of work for this study.

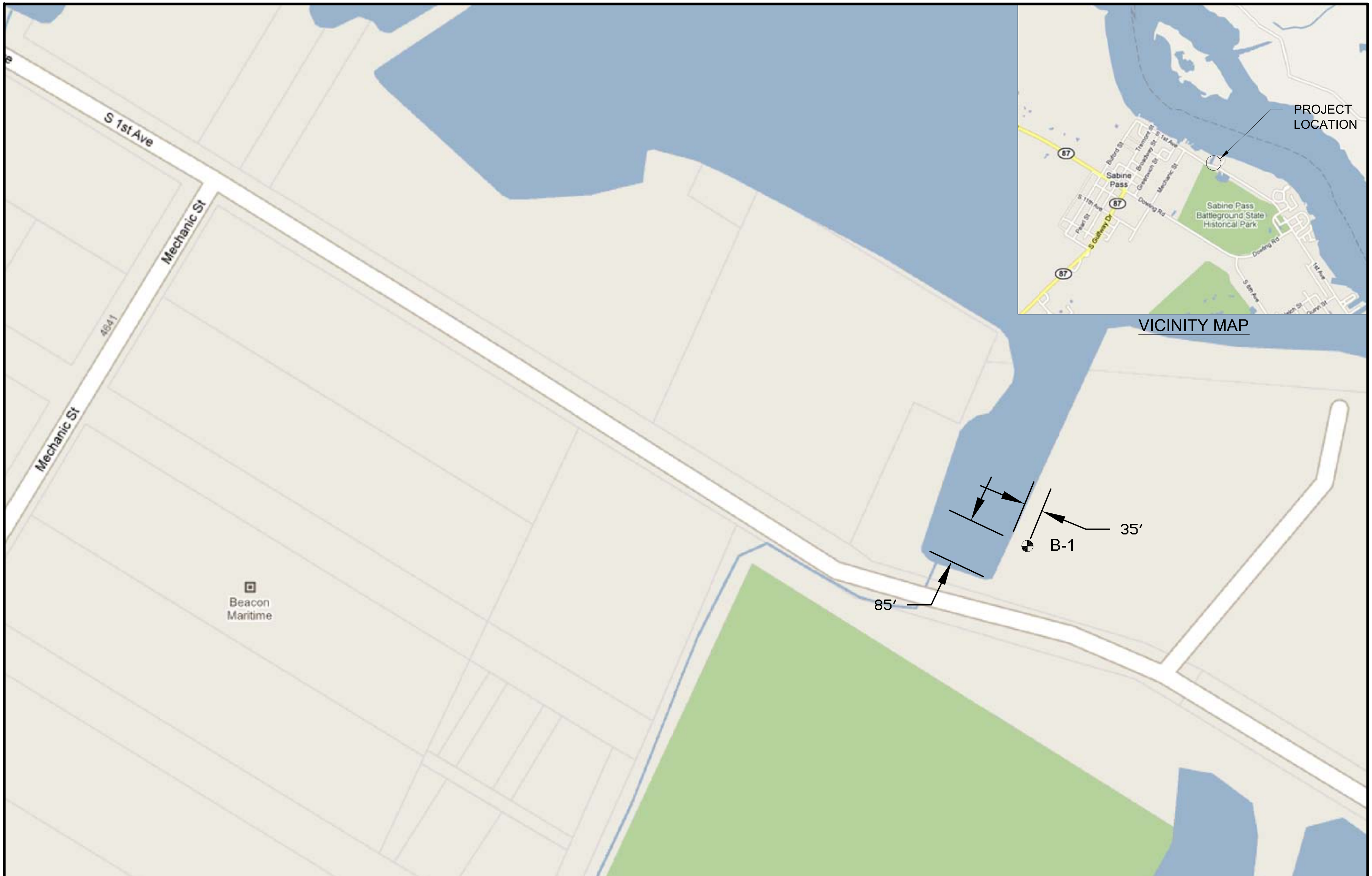
7.3 Construction Monitoring

Construction surveillance is recommended and has been assumed in preparing our recommendations. These field services are required to check for changes in conditions that may result in modifications to our recommendations. The quality of the construction practices will affect foundation performance and should be monitored.

7.4 Closing Remarks

We appreciate the opportunity to be of service during this phase of the project, and we look forward to continuing our services during the construction phase and on future projects.

BORING LOCATION MAP



LEGEND:

BORING LOCATION

Tolunay-Wong Engineers, Inc.
Beaumont, Texas

BORING LOCATION			
NEW MARINE SECURITY CENTER SABINE PASS, TEXAS			
LEAP ENGINEERING BEAUMONT, TEXAS			
DRAWN BY:	S.O.S.	DWG. NO.	10.23.002-01
CHECKED BY:	D.W.G.	SCALE:	NONE
APPROVED BY:	P.J.K.	DATE:	JANUARY 19, 2010

APPENDIX A

TWE PROJECT BORING LOG B-1
AND
KEY TO LOG TERMS AND SYMBOLS

LOG OF BORING B-1

PROJECT: Regional Marine Security Center
Sabine Pass, Texas

CLIENT: Leap Engineering
Beaumont, Texas

COORDINATES: N
W

SURFACE ELEVATION:

DRILLING METHOD:
Dry Augered: 0' to 15'
Wash Bored: 15' to 120'

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
0	Fill: Brown & tan FAT CLAY (CH), w/ base material											
	Dark gray FAT CLAY (CH)											
5	-becomes soft dark gray & gray -w/ organics @ 4'-6'	(P)0.75		55	67			0.46	3			
	-becomes very soft -w/ ferrous nodules @ 6'-8'	(P)0.25		75		94	66				92	
		(P)0.25										
10												
15	-w/ sand pockets @ 13'-15'		WOH	63		66	47					
20	no recovery		WOH									
25	-w/ sand seams & pockets @ 23'-25'		WOH 1/6 1/6									
30	-w/ shell fragments @ 28'-30'		1/6 WOH 1/6									
35	-w/ silt pockets @ 33'-35'		WOH	73		83	61				91	

COMPLETION DEPTH: 120 ft
DATE BORING STARTED: 1-8-10
DATE BORING COMPLETED: 1-8-10
LOGGER: J. Turner
PROJECT NO.: 10.23.002

NOTES: Free water was encountered at a depth of 8-feet. Static water level was not measured due to hole squeezing at 6-feet after ten minutes.

TOLUNAY-WONG ENGINEERS, INC.

LOG OF BORING B-1

PROJECT: Regional Marine Security Center
Sabine Pass, Texas

CLIENT: Leap Engineering
Beaumont, Texas

COORDINATES: N
W

SURFACE ELEVATION:

DRILLING METHOD:
Dry Augered: 0' to 15'
Wash Bored: 15' to 120'

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
35												
40			WOH 1/6 WOH									
45			WOH									
50	-w/ wood @ 48'-50'		1/6 1/6 2/6									
55		(P)0.25		70		93	67					
60	-soft @ 58'-60'	(P)0.50		78	54			0.43	10			
65	-becomes firm	(P)0.75										
70		(P)1.00		70	58			0.61	4			

COMPLETION DEPTH: 120 ft
DATE BORING STARTED: 1-8-10
DATE BORING COMPLETED: 1-8-10
LOGGER: J. Turner
PROJECT NO.: 10.23.002

NOTES: Free water was encountered at a depth of 8-feet. Static water level was not measured due to hole squeezing at 6-feet after ten minutes.

TOLUNAY-WONG ENGINEERS, INC.

LOG OF BORING B-1

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Sabine Pass, Texas

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Beaumont, Texas

COORDINATES: N
W

SURFACE ELEVATION:

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70												
73	Gray CLAYEY SAND (SC)	(P)0.50		26		27	8				27	
79	Dense gray POORLY GRADED SAND w/ CLAY (SP-SC)		9/6 21/6 29/6									
84	-becomes very dense		25/6 30/6 25/6									
89			25/6 35/6 38/6	25							6	
93	Very stiff gray & brown FAT CLAY (CH), w/ silt seams -w/ slickensides @ 93'-105'	(P)3.25		28		77	53				79	
98	-becomes stiff	(P)2.75		35	87			1.81	9			
103		(P)2.50		35		80	57				93	
105												

COMPLETION DEPTH: 120 ft
DATE BORING STARTED: 1-8-10
DATE BORING COMPLETED: 1-8-10
LOGGER: J. Turner
PROJECT NO.: 10.23.002

NOTES: Free water was encountered at a depth of 8-feet. Static water level was not measured due to hole squeezing at 6-feet after ten minutes.

LOG OF BORING B-1

PROJECT: Regional Marine Security Center
Sabine Pass, Texas

CLIENT: Leap Engineering
Beaumont, Texas

COORDINATES: N W
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: 0' to 15'
Wash Bored: 15' to 120'

ELEVATION (FT) / DEPTH (FT)	SAMPLE TYPE	SYMBOL	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
105														
			-becomes very stiff	(P)4.00										
110														
			-firm @ 113'-115'	(P)3.50		21	104			0.72	5			
115														
			-w/ silt seams @ 118'-120'	(P)4.25		29		52	33				96	
120			Bottom @ 120'											
125														
130														
135														
140														

COMPLETION DEPTH: 120 ft
DATE BORING STARTED: 1-8-10
DATE BORING COMPLETED: 1-8-10
LOGGER: J. Turner
PROJECT NO.: 10.23.002

NOTES: Free water was encountered at a depth of 8-feet. Static water level was not measured due to hole squeezing at 6-feet after ten minutes.

TOLUNAY-WONG ENGINEERS, INC.

SYMBOLS AND TERMS USED ON BORING LOGS

Most Common Unified Soil Classifications System Symbols

Fill	Silt w/ Sand (ML)
Pavement	Well Graded Sand (SW)
Lean Clay (CL)	Well Graded Sand w/ Gravel (SW-GM)
Lean Clay w/ Sand (CL)	Poorly Graded Sand (SP)
Sandy Lean Clay (CL)	Poorly Graded Sand w/ Silt (SP-SM)
Fat Clay (CH)	Silt (ML)
Fat Clay w/ Sand (CH)	Elastic Silt (MH)
Sandy Fat Clay (CH)	Elastic Silt w/ Sand (MH-SP)
Silty Clay (CL)	Silty Gravel (GM)
Sandy Silty Clay (CL-ML)	Clayey Gravel (GC)
Silty Clayey Sand (SC-SM)	Well Graded Gravel (GW)
Clayey Sand (SC)	Well Graded Gravel w/ Sand (SP-GM)
Sandy Silt (ML)	Poorly Graded Gravel (GP)
Silty Sand (SM)	Peat

Sampler Symbols	Meaning
	Pavement core
	Thin - walled tube sample
	Standard Penetration Test (SPT)
	Auger sample
	Sampling attempt with no recovery
	TxDOT Cone Penetrometer Test

Field Test Data

2.50	Pocket penetrometer reading in tons per square foot
8/6"	Blow count per 6 - in. interval of the Standard Penetration Test
	Observed free water during drilling
	Observed static water level

Laboratory Test Data

Wc (%)	Moisture content in percent
Dens. (pcf)	Dry unit weight in pounds per cubic foot
Qu (tsf)	Unconfined compressive strength in tons per square foot
UU (tsf)	Compressive strength under confining pressure in tons per square foot
Str. (%)	Strain at failure in percent
LL	Liquid Limit in percent
PI	Plasticity Index
#200 (%)	Percent passing the No. 200 mesh sieve
( )	Confining pressure in pounds per square inch
*	Slickensided failure
**	Did not fail @ 15% strain

RELATIVE DENSITY OF COHESIONLESS & SEMI-COHESIONLESS SOILS

The following descriptive terms for relative density apply to cohesionless soils such as gravels, silty sands, and sands as well as semi-cohesive and semi-cohesionless soils such as sandy silts, and clayey sands.

Relative Density	Typical N_{60} Value Range*
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	Over 50

* N_{60} is the number of blows from a 140-lb weight having a free fall of 30-in. required to penetrate the final 12-in. of an 18-in. sample interval, corrected for field procedure to an average energy ratio of 60% (Terzaghi, Peck, and Mesri, 1996).

CONSISTENCY OF COHESIVE SOILS

The following descriptive terms for consistency apply to cohesive soils such as clays, sandy clays, and silty clays.

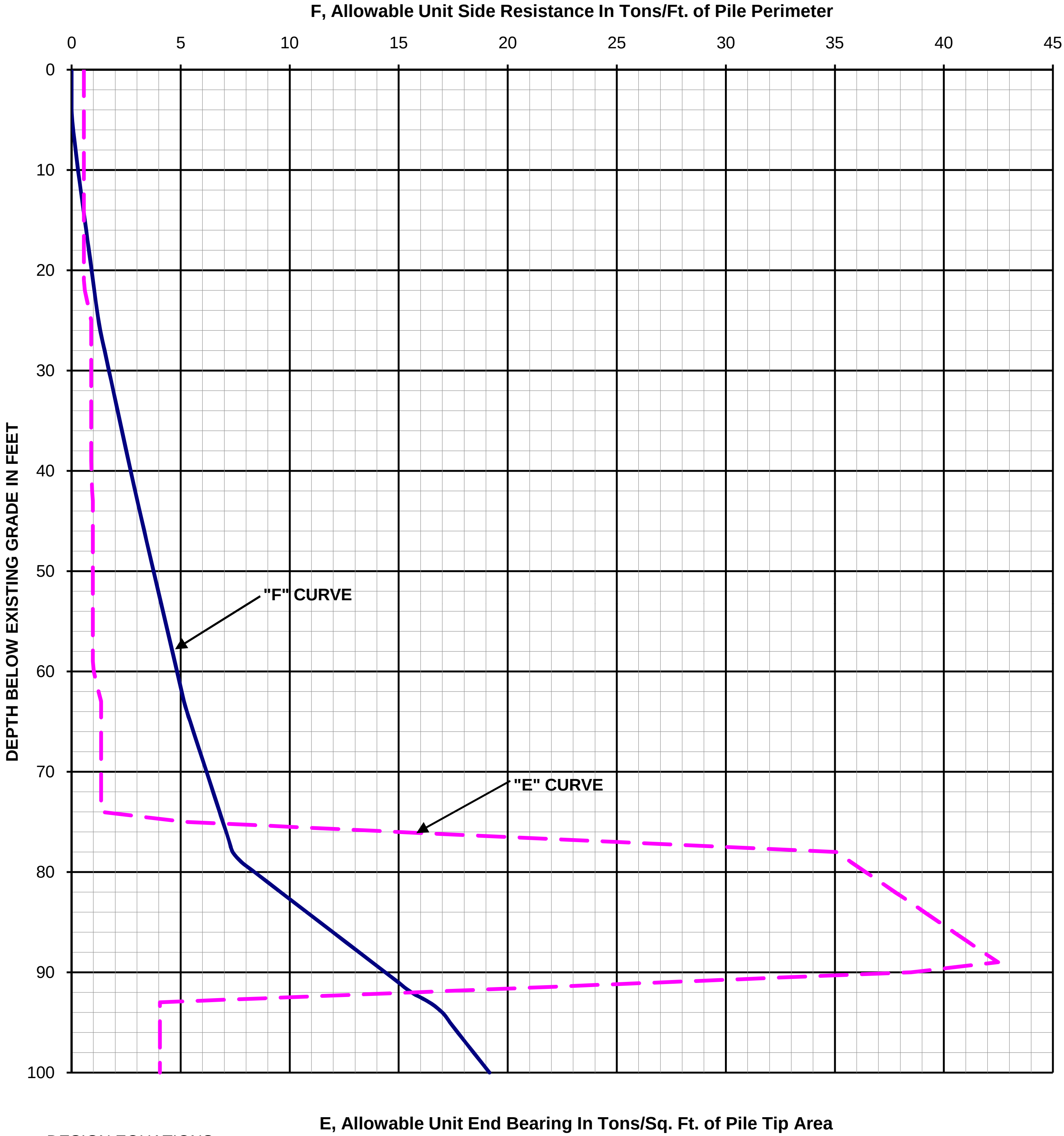
Pocket Penetrometer (tsf)	Typical Compressive Strength (tsf)	Consistency	Typical SPT "N_{60}" Value Range**
pp < 0.50	qu < 0.25	Very soft	≤ 2
0.50 ≤ pp < 0.75	0.25 ≤ qu < 0.50	Soft	3-4
0.75 ≤ pp < 1.50	0.50 ≤ qu < 1.00	Firm	5-8
1.50 ≤ pp < 3.00	1.00 ≤ qu < 2.00	Stiff	9-15
3.00 ≤ pp < 4.50	2.00 ≤ qu < 4.00	Very Stiff	16-30
pp ≥ 4.50	qu ≥ 4.00	Hard	≥ 31

** An "N_{60}" value of 31 or greater corresponds to a hard consistency. The correlation of consistency with a typical SPT "N_{60}" value range is approximate.

APPENDIX B

UNIT FRICTION AND END BEARING
PILE CAPACITY CURVES

ALLOWABLE UNIT SIDE FRICTION AND END BEARING RESISTANCE
DRIVEN TIMBER, CONCRETE OR CLOSED-ENDED STEEL PIPE PILES



DESIGN EQUATIONS:
Compression: $Q_C = PF + AE$
Tension: $Q_T = PF$

TERMS:
P = Average Pile Perimeter, Ft.
A = Pile Tip Area, Sq. Ft.
F, E = Unit Friction and End Bearing Factors From Curves
Q = Allowable Pile Capacity in Tons

EXAMPLE:
16" Square Precast Concrete Pile, 80' Length

P = 5.33 ft. F = 8.38Tons/Ft.
A = 1.77 ft^2 E = 36.41 Tons/Ft^2
Q_C = (5.33)(8.38) + (1.77)(36.41) = 109 Tons
Q_T = (5.33)(8.38) = 44 Tons

TWE Project No. 10.23.002
SHEET B-1