1 EXECUTIVE SUMMARY

Fugro-McClelland Marine Geosciences, Inc. (FMMG) performed geotechnical investigations for proposed offshore structures across the Texas Offshore Port System (TOPS), located in Block A-56, of the Galveston Area in the Gulf of Mexico. The primary purpose of the investigation was to obtain soil data to develop foundation design recommendations for the Single Point Mooring (SPM) facility, designated as SPM #1 and SPM #2, of the proposed TOPS facility in Block A-56.

The field investigation was performed on June 30, July 2 and 3, 2008, from the R/V *Seaprobe*. The soil conditions were determined by performing a total of four exploratory borings, with one boring at a selected anchor leg location and one boring at the proposed Pipe Line End Termination (PLET) location at each of the SPM locations. These borings were drilled to a penetration of 131-ft below mudline. The water depths ranged from 117 to 121 ft at the soil boring locations across Block A-56. An exploratory boring was also drilled at the Offshore Terminal location in Block A-56 and is presented in FMMG's Report No. 0201-6502.

Field and laboratory tests were conducted to determine the pertinent index and engineering properties of the soils encountered. Engineering analyses were then performed to develop the required design information. For the purposes of discussion and presentation, "driven pipe pile" is used in this report to represent foundation piles, caissons and conductors, unless otherwise specified.

This report presents axial and lateral design data for 24-in.-diameter driven pipe piles at the two PLET locations and 42-in.-diameter driven pipe piles for the two anchor leg locations. This data was developed using methods and recommendations presented in API RP 2A (2000). Pile penetrations should be based on allowable capacities with appropriate safety or load resistance factors.

The ultimate seafloor bearing capacity for mud mat and tubular members bearing on the seafloor at the PLET locations were computed using general bearing capacity methods and recommendations presented in API RP 2A-WSD (2000). Seafloor bearing capacity equations are presented in the appropriate sections of the main text of this report to facilitate design of the PLET foundations, if required.

A review of the geotechnical data indicates that the soil stratigraphy in the top 34-ft penetration is variable across the block, alluding to channel features across the investigation area. The exploratory boring revealed clay layers interbedded with silt, sandy silt, or silty fine sand layers of varying thickness within approximately the upper 34-ft of sediments. The underlying soils to at least 131-ft penetration consists of firm to stiff lean clay to clay, with a shear strength profile that increases with depth. Soil variability is demonstrated in a comparison of the log of boring and test results presented in Section 3 with the subbottom geophysical data at each of the boring locations. The geophysical survey was performed by Fugro Geoservices, Inc. (FGSI) of Lafayette, LA and is presented in detail in FGSI Report No. 2407-1298.

The information revealed in this investigation suggests that the piles can be driven with a properly sized hammer to a sufficient depth in the above stratigraphy to achieve the desired pile capacity. Supplementary installation procedures, if necessary, should be applied under close engineering supervision to determine the impact of the procedures on pile capacity.

2 GENERAL PROJECT INFORMATION

2.1 INTRODUCTION

2.1.1 Purpose and Scope

Fugro-McClelland Marine Geosciences, Inc. (FMMG) performed a geotechnical investigation program to investigate soil conditions at the proposed Single Point Mooring (SPM) facility locations in the Texas Offshore Port System (TOPS), located in Block A-56, of the Galveston Area in the Gulf of Mexico. The primary purpose of the investigation was to obtain data to develop foundation design recommendations for anchor leg and Pipe Line End Termination (PLET) locations at the facility sites designated as SPM #1 and SPM #2. To accomplish this objective, the following tasks were performed:

(1) Four soil borings, with one boring at a selected anchor leg location and one boring at the proposed PLET location at each of the SPM locations, were drilled to 131-ft penetration below seafloor to explore the subsurface stratigraphy and obtain soil samples for laboratory testing;

(2) Field and laboratory tests were conducted to evaluate pertinent index and engineering properties of the foundation materials;

(3) A comparison of the geotechnical and geophysical data was performed to investigate soil variability to help in selecting soil parameters; and

(4) Engineering analyses were performed to develop pile design information, seafloor bearing capacity, and a general pile installation assessment.

Enterprise Field Services, LLC specified the boring locations and designations. A plan of borings presenting the relative positions of the four borings is presented on Plate 2-1.

2.1.2 Report Format

The results of the geotechnical investigations completed for the TOPS campaign are presented in the following reports:

0201-6500:	Offshore Terminal Location, Block A-36, Galveston Area;
0201-6501:	SPM #1 and SPM #2 PLET and Anchor Leg locations, Block A-36, Galveston Area;
0201-6502:	Offshore Terminal Location, Block A-56, Galveston Area;
0201-6503:	SPM #1 and SPM #2 PLET and Anchor Leg locations, Block A-56, Galveston Area (this report);
0201-6504:	Offshore Terminal Location, Block A-59, Galveston Area; and
0201-6505:	SPM #1 and SPM #2 PLET and Anchor Leg locations, Block A-59, Galveston Area.

The initial section of this report contains brief descriptions of the field and laboratory phases of the study, including a general description of the soil stratigraphy and a summary of the findings of the geophysical survey across Block A-56. Also included in this section is a general discussion of the engineering methods, axial and lateral pile design, used at all the boring locations. Section 3 presents a detailed description of the site-specific conditions encountered at each boring location followed by brief discussions of axial pile design, lateral pile analyses, seafloor bearing capacity, and pile installation recommendations. Discussions of the field and laboratory investigations are presented in Appendix A.

Appendix B contains discussions of analytical procedures used in our engineering analyses. Appendix C contains a positioning report by Fugro Chance, Inc., of Lafayette, Louisiana.

For the purposes of discussion and presentation, "driven pipe pile" is used in this report to represent foundation piles, caissons and conductors, unless otherwise specified.

2.2 FIELD AND LABORATORY INVESTIGATIONS

The field investigation was performed on June 30, July 2 and 3, 2008, from the R/V *Seaprobe*. The soil conditions were determined by performing four exploratory borings, two at each SPM location with one boring at a selected anchor leg location, and one boring at the proposed PLET location. Enterprise Field Services selected the boring locations. These borings were drilled to a penetration of 131-ft below mudline. The water depths at the boring locations ranged from 117 to 121 ft. A chronological summary of field operations is presented in Appendix A.

2.2.1 Exploratory Borings

FMMG personnel drilled the soil borings with a DMX drill rig positioned over the centerwell of the R/V *Seaprobe*. The vessel was anchored at the boring location by a 4-point mooring system. Soil conditions at the site were explored by drilling a group of four soil borings to 131-ft penetration below the seafloor. The final coordinates for the boring locations are presented in Table 2-1. A plan of borings within Block A-56, of the Galveston Area is presented on Plate 2-1. Fugro Chance, Inc., of Lafayette, Louisiana, conducted surveying utilizing STARFIX and DGPS, and performed a 360-degree scanning sonar survey at each of the boring locations. The positioning report, prepared by Fugro Chance, is presented in Appendix C. The scanning sonar reports are available from Fugro Chance upon request.

Table 2-1: Final Boring Coordinates
(Texas South Central Zone Coordinates)

FMMG Boring Designation	Fugro Chance Boring Designation	Proposed Boring Coordinates	Final Boring Coordinates	Boring Termination Depth (ft)
SPM #1 PLET	Core 3	X = 3,258,627.75 ft Y = 252,334.60 ft	X = 3,258,639 ft Y = 252,312 ft	131
SPM #1 ANCHOR LEG #2	Core 1	X = 3,257,224.19 ft Y = 251,897.66 ft	X = 3,257,199 ft Y = 251,890 ft	131
SPM #2 PLET	Core 4	X = 3,265,632.42 ft Y = 256,177.08 ft	X = 3,265,650 ft Y = 256,155 ft	131
SPM #2 ANCHOR LEG #6	Core 2	X = 3,266,735.50 ft Y = 257,148.73 ft	X = 3,266,759 ft Y = 257,169 ft	131

Samples were obtained through 5.0-in.-OD, 4.5-in.-IF drill pipe at all the locations. Samples were generally spaced at 3-ft intervals to 20-ft penetration, at 5-ft intervals to 68-ft penetration, and at 10-ft intervals thereafter to the final boring depth at all the locations. The drilling and sampling techniques used to complete this boring are explained in detail in Appendix A.

Two water depths were measured at each boring location using a seafloor sensor seated in the drill bit. The water depth measurements are tabulated in Table 2-2. The water depth measurements are intended for the purpose of the geotechnical investigation only, and are not corrected for tidal or other variations. If utilized for other purposes, the water depth measurement should be adjusted to account for meteorological tide and datum corrections. The water depths measuring procedures are explained in detail in Appendix B.

Table 2-2: Measured Water Depths

Boring Designation	Water Depth (ft)	Time and Date of Measurement	Supplemental Water Depth (ft)	Time and Date of Measurement
SPM #1 PLET	118	2125 hours on July 2, 2008	119	0245 hours on July 3, 2008
SPM #1 ANCHOR LEG #2	121	0650 hours on July 3, 2008	121	1105 hours on July 3, 2008
SPM #2 PLET	118	1205 hours on July 2, 2008	117	1725 hours on July 2, 2008
SPM #2 ANCHOR LEG #6	117	0345 hours on June 30, 2008	117	0945 hours on June 30, 2008

2.2.2 Field and Laboratory Tests

The soil testing program was designed to evaluate pertinent index and engineering properties of the foundation soils. During the field operation, all samples were extruded from the sampler and classified by the soil technician or field engineer. Unit weight, Torvane, pocket penetrometer, miniature vane and unconsolidated-undrained triaxial compression tests were performed in the field on selected cohesive samples. All of the samples were shipped to Fugro's Houston laboratory where Atterberg limit tests, water content tests, and grain-size analyses, as well as additional density tests, unconsolidated-undrained triaxial compression tests, and miniature vane tests, were performed.

A description of relevant laboratory procedures is provided in Appendix A. The strength and classification test results are presented graphically on the Logs of Boring and Test Results in Section 3. Grain-size distribution curves from sieve-analysis and stress-strain curves from triaxial compression tests are presented in Appendix A.

2.3 GENERAL SOIL CONDITIONS

2.3.1 Soil Stratigraphy

The soil stratigraphy at each of the boring locations disclosed by the field and laboratory investigations is presented in Section 3. The soil stratigraphy is based on the classification of soil samples recovered from the boring and observations made during drilling operations. Detailed soil descriptions, for each location, that include textural variations and inclusions are noted on the respective boring log presented in Section 3. A key to the terms and symbols used on the boring log is presented on Plate 2-2.

The Roman numeral representing each stratum is also shown on the respective boring log and on relevant plates.

In general, the exploratory borings revealed stratified cohesive and granular soil profiles with clay layers interbedded with silt, sandy silt, or silty fine sand layers of varying thickness, within approximately the upper 34 ft of sediments. Alternating clay and silty fine sand layers were also encountered at the Offshore Terminal boring location in Block A-59 (FMMG Report No. 0201-6502) to a depth of about 46-ft penetration. The soil below the interlayered zone consists predominately of firm to stiff clay with a shear strength profile that increases with increasing depth.

2.3.2 Interpretation of Soil Properties

The shear strength and submerged unit weight profiles best represent the assembled test results plotted on the boring logs are shown on the respective "Design Strength Parameters" and "Design Submerged Unit Weight" plots in Section 3. These profiles were used in the engineering analyses.

In developing the shear strength profile for the cohesive soils, undrained shear strength test results from miniature vane and unconsolidated-undrained triaxial compression tests were analyzed. The selection of shear strength profiles for clay soils and the effects of the type of sampling procedure on the profiles are discussed by Dennis and Olson (1983) and Quiros, et al. (1983). Strength parameters for granular soils were selected based on their gradation and relative density estimated from sampler blow count information. The submerged unit weight profile was developed from actual density measurements and calculated unit weight values based on sample moisture content and the assumption of 100 percent sample saturation.

The recommendations for foundation design and installation were developed with the assumption that the soil conditions revealed by the borings are continuous throughout the general area of the proposed foundation structure. Consideration of possible stratigraphic changes, faulting, or geologic conditions which may influence foundation design were beyond the scope of this investigation. **Variations in soil conditions may become evident during PLET or pile installation.** If variations are found, a re-evaluation of the recommendations in this report may be necessary. We recommend that additional soil borings be obtained to determine the site-specific conditions within the immediate proximity of the remaining proposed PLET and anchor locations.

2.4 GEOPHYSICAL SURVEY SUMMARY

2.4.1 Introduction

The purpose of this section is to briefly summarize the results and findings of the high-resolution geophysical survey conducted in the area encompassing Block A-56 in the Galveston Area as related to the proposed SPM locations. A map indicating the subbottom profiler data lines is presented in Plate 2-3, with the geophysical lines used for interpretation of the highlighted soil borings. Section 3 contains cross-section plots of integrated data that compares the soil stratigraphy from the borings and geophysical data within the immediate vicinity of each boring. A detailed assessment of the seafloor and shallow geological conditions in Block A-56 is presented in the geophysical report, FGSI Report No. 2407-1298 (Fugro Geoservices, Inc., 2008).

2.4.2 Water Depth and Seafloor Topography

Water depths across the survey area range from -111 feet MLLW in the northwest corner to -122 feet MLLW in the southeast corner, with zero datum equal to Mean Lower Low Water. Bathymetric contours within Block A-56 define a seafloor that slopes gently to the southeast at a gradient of 4 feet per mile (0.04°).

2.4.3 Soil Conditions

Five geotechnical borings across the project field indicate an upper unit of very soft to stiff gray clay to lean clay to greenish gray stiff to hard clay up to 11 feet Below Mud Line (BML). Underlying these soils, a gray lean clay to silty fine sand to silt unit extends to approximately 20 to 34 feet BML. From 34 feet BML, a firm to stiff lean clay is present to approximately 131 feet BML. The borings represent the generalized soil stratigraphy for the study area, but soil properties vary between boring locations as revealed by the mapped channel features and acoustic voids indicated in the geophysical data.

2.4.4 Geological Features and Hazards

Biogenic gas accumulations "acoustic voids" attenuated the subbottom profiler records in some locations result in reduced penetration and resolution within those areas. Extra caution should be exercised in these areas. However, the soil borings could be used as an indicator that shallow gas was not encountered at the boring locations.

Channels buried 2 to 18 feet below the seafloor were noted throughout the survey area. Where discernable, thalweg depths range from 18 to 113 feet BML. Areas of channelized sediment represent seafloor locations where geotechnical sediment properties may vary significantly. Site-specific soil borings would be necessary to determine the specific geotechnical properties of the sediments within the channels.

2.5 PILE DESIGN INFORMATION

The pile design information developed for this study includes ultimate axial capacities, axial load-pile movement data, and lateral soil resistance-pile deflection (p-y) characteristics. The analytical methods used to develop this information are presented briefly in the following paragraphs and in more detail in Appendix B.

2.5.1 Axial Pile Design

Method of Analysis. The ultimate axial capacity of piles was computed using the static method of analysis described in API RP 2A (2000). In this method, the ultimate compressive capacity of a pile for a given penetration is taken as the sum of the skin friction on the pile wall and the end bearing on the pile tip. The weight of the pile and soil plug is neglected in the computations. When computing the ultimate tensile capacity of piles, as well as the compressive capacity of conductors or caissons, the end bearing component is also neglected.

Ultimate Axial Capacity. The unit skin friction and unit end bearing values are presented in Section 3, and were calculated using the API RP 2A methods described in Appendix B. These values were used to calculate the ultimate axial compressive and tensile capacities for 24-in.-diameter pipe piles at the PLET locations and 42-in.-diameter pipe piles at the anchor leg locations, driven to final penetration. Capacity curves for driven pipe piles (conductors, caissons, anchor and foundation piles) are also presented in Section 3.

API RP 2A recommends that pile penetrations be selected using appropriate factors of safety or pile resistance factors. For working stress design (WSD), API RP 2A recommends that pile penetrations be selected to provide factors of safety of at least 2.0 with respect to normal operating loads and at least 1.5 with respect to maximum design storm loads. These factors of safety should be applied to the design compressive and tensile loads. For load and resistance factor design (LRFD), API RP 2A recommends pile resistance factors of 0.7 and 0.8 for operating and maximum storm loads, respectively. Also, appropriate load factors should be used to determine operating and maximum storm loads for LRFD design.

Axial Load Transfer Data. Axial load-pile movement analyses are usually performed using a computer solution based on methods developed by Reese (1964) or Matlock, et al. (1976). These programs treat the pile as a series of discrete elements, represented by linear springs that are acted upon by nonlinear springs representing the soil. The nonlinear soil springs are referred to as t-z and Q-z curves. Input data for the program include: (1) pile dimensions and material properties, (2) load transfer characteristics of the soil surrounding the pile, and (3) the pile tip load-tip movement relationship. The axial load transfer curves were computed using procedures described in API RP 2A and outlined in Appendix B.

The results of side load-side movement (t-z) and tip load-tip movement (Q-z) data for 24-in.-diameter driven pipe piles at the PLET locations and 42-in.-diameter driven pipe piles at the anchor leg locations are presented in Section 3. The presented Q-z data should be used for foundation piles and neglected for caissons and conductor design. In developing the axial load transfer data in the cohesive soils, a post-peak adhesion ratio of 0.90 was utilized.

2.5.2 Lateral Pile Design Data

The soil resistance-pile deflection (p-y) characteristics of the soils at the boring locations were developed for individual 24-in.-diameter driven pipe piles at the PLET locations and 42-in.-diameter driven pipe piles at the anchor leg locations. These data may be used in lateral load analyses of driven piles, conductors and caissons. The p-y data for cyclic loading were developed to 100-ft penetration using the procedures proposed by Matlock (1970) for soft clays and O'Neill and Murchison (1983) for sands. These procedures have been outlined in API RP 2A and briefly explained in Appendix B. The stratigraphy and parameters used to develop the p-y data at the boring locations are presented in Section 3, together with the p-y data for 24-in.-diameter driven pipe piles at the PLET locations and 42-in.-diameter driven pipe piles at the anchor leg locations. P-y values presented at 100-ft penetration may be used for lateral load analyses at greater depths.

2.5.3 Pile Group Effects

API RP 2A recommends that a pile spacing of less than eight pile diameters be evaluated for group effects. This additional analysis can be performed by FMMG when pile spacing has been selected.

2.5.4 Pile and Spud Can Interaction

When a spud can penetrates into the seafloor, a cylindrical zone of remolded and lower (degraded) shear strength is created. This zone of lower shear strength soil is called a spud can depression or pockmark. Piles located near existing, or future, spud can depressions may have degraded axial and lateral capacities. This degradation is a function of spud can and pile diameter, depth of spud can penetration, distance between spud can depression and pile, and soil type. Consideration should also be given to the effects on pile performance associated with the potential use of jack-up rigs and the formation of future spud can depressions. FMMG can perform this additional evaluation when the geometry and layout of the piles and spud can depressions are determined.

2.6 SEAFLOOR BEARING CAPACITY

2.6.1 Bearing Capacity

Ultimate bearing capacity equations for the surface soils were taken from a design method developed by Skempton (1951) based on undisturbed shear strength. Equations are presented in Section 3 for each PLET boring location and can be used to determine the ultimate bearing capacity for horizontal tubular members and mud mats resting on the seafloor.

The ultimate bearing capacity of the near-seafloor soils is a function of the size and configuration of the mud mats. A more detailed analysis of soil deformation and bearing capacity can be undertaken when the actual configuration and loading conditions are determined.

For Working Stress Design (WSD), API RP 2A recommends that a safety factor of at least 2.0 be used with the ultimate bearing capacity determined from the above equations. For Load and Resistance Factor Design (LRFD), a resistance factor of 0.67 is recommended. Also, an appropriate load factor should be used to determine the jacket load in the LRFD design procedure. The ultimate bearing (load-carrying) capacity of a horizontal tubular member or mud mat may be calculated as the ultimate bearing capacity of the soil multiplied by the base area of the mat or member. The equations for ultimate bearing capacity presented above are based on static bearing capacity conditions. Significant vertical PLET velocities at the time of its placement could cause large or uneven foundation settlements.

2.6.2 Degraded Bearing Capacity

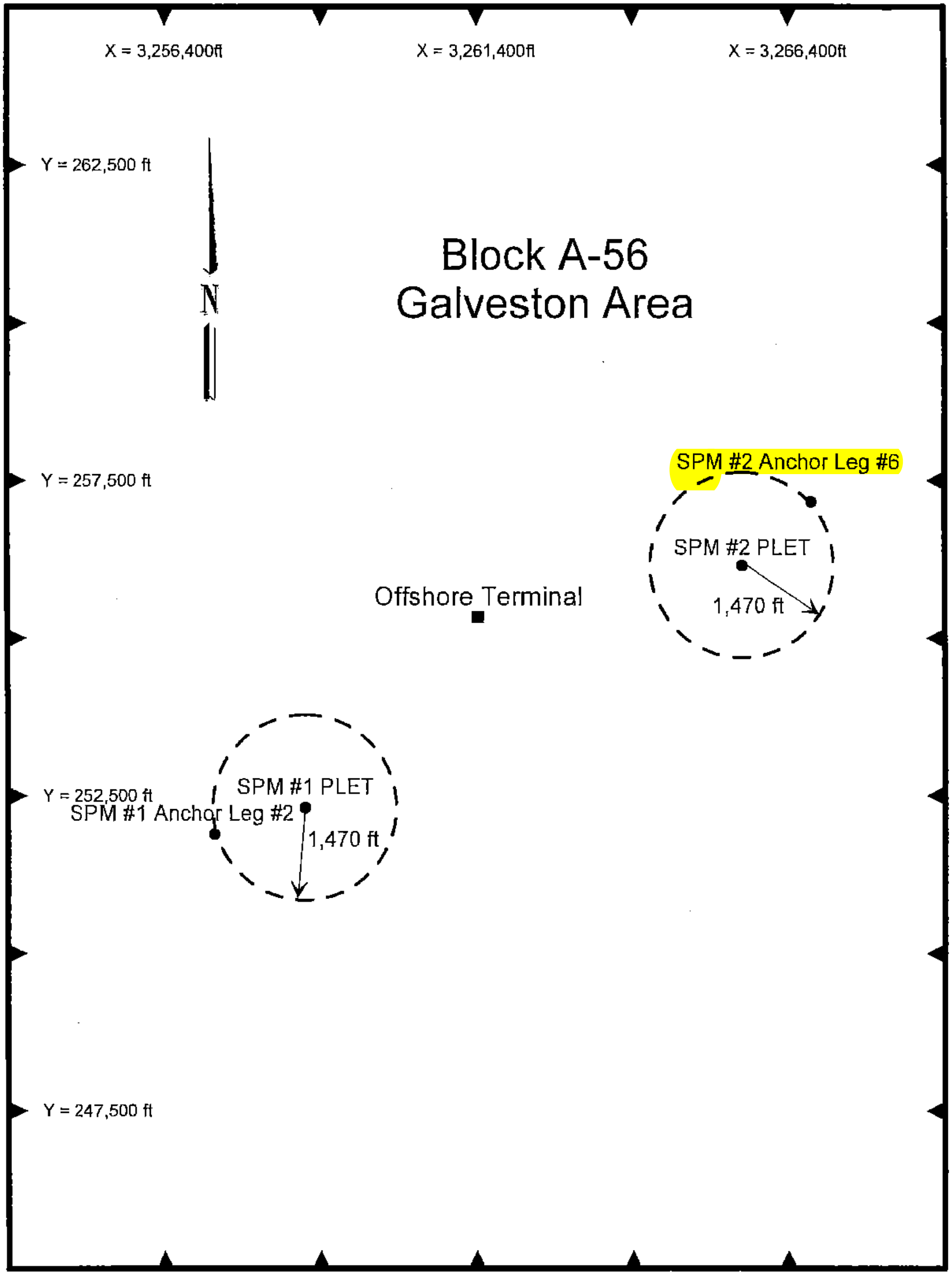
When a spud can penetrates into the seafloor, a cylindrical zone of remolded and lower (degraded) shear strength is created. This zone of lower shear strength soil is called a spud can depression or pockmark. Mud mats located in, or near, existing depressions may have reduced (degraded) bearing capacity. This degradation is a function of spudcan diameter, depth of penetration, distance between spud can depression and mud mat, and soil type. FMMG can perform this additional evaluation when the geometry and layout of the mud mats and spudcan depressions have been determined.

2.7 PILE INSTALLATION CONSIDERATIONS

Pile driving problems are not expected at these boring sites. The information in this site investigation suggests that the piles can likely be driven with a properly sized hammer to a sufficient depth in the stratigraphies, presented in Section 3, to achieve the desired pile capacity. A pile drivability analysis can be used to evaluate the proper hammer-pile system. Unfavorable soil conditions or driving equipment problems may prevent piles from being driven to the desired penetration. Interruptions in driving should be as short as possible to reduce set-up of the soil around the piles. Supplementary installation procedures, if necessary, should be applied under close engineering supervision to determine the impact of the procedures on pile capacity.

2.8 SERVICE WARRANTY

The section entitled "Service Warranty" at the end of Appendix B outlines the limitations and constraints of this report in terms of a range of considerations including, but not limited to, its purpose, its scope, the data on which it is based, its use by third parties, possible future changes in design procedures and possible changes in the conditions at the site with time. This section represents a clear description of the constraints, which apply to all reports issued by FMMG. It should be noted that the Service Warranty does not in any way supersede the terms and conditions of the contract between FMMG and the Client.



Projection: Texas South Central Zone Coordinates

PLAN OF BORINGS

Texas Offshore Port System
Block A-56, Galveston Area

Drawn By: AW Date: 8/19/08
Checked By: MB Date: 8/19/08
Approved By: Date:

Drawn By: Tomel Date: 9/4/08

Checked By: Date: 9/5/08

Approved By: Date: 9/5/08

TERMS AND SYMBOLS USED ON BORING LOG

SOIL TYPES

Sand, Silt, Clay, Gravel, Debris

Silty Sand, Sandy Silt, Sandy Clay, Peat or Highly Organic, Coral

Clayey Sand, Clayey Silt, Silty Clay, Rock, Shell

SAMPLER TYPES

Thin-Walled Tube, Liner, In Situ Test

Rock Core, Piston, No Recovery

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

	6"		3"		3/4"		4		10		40		200		
BOULDERS		COBBLES		GRAVEL COARSE		GRAVEL FINE		SAND COARSE		SAND MEDIUM		SAND FINE		SILT	CLAY
	152		76.2		19.1		4.76		2.00		0.420		0.074		0.002

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS(1)

Consistency	Undrained Shear Strength, Kips Per Sq Ft
Very Soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	greater than 4.00

DENSITY OF GRANULAR SOILS(2,3)

Descriptive Term	*Relative Density, %
Very Loose	less than 15
Loose	15 to 35
Medium Dense	35 to 65
Dense	65 to 85
Very Dense	greater than 85

*Estimated from sampler driving record

SOIL STRUCTURE(1)

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

REFERENCES:

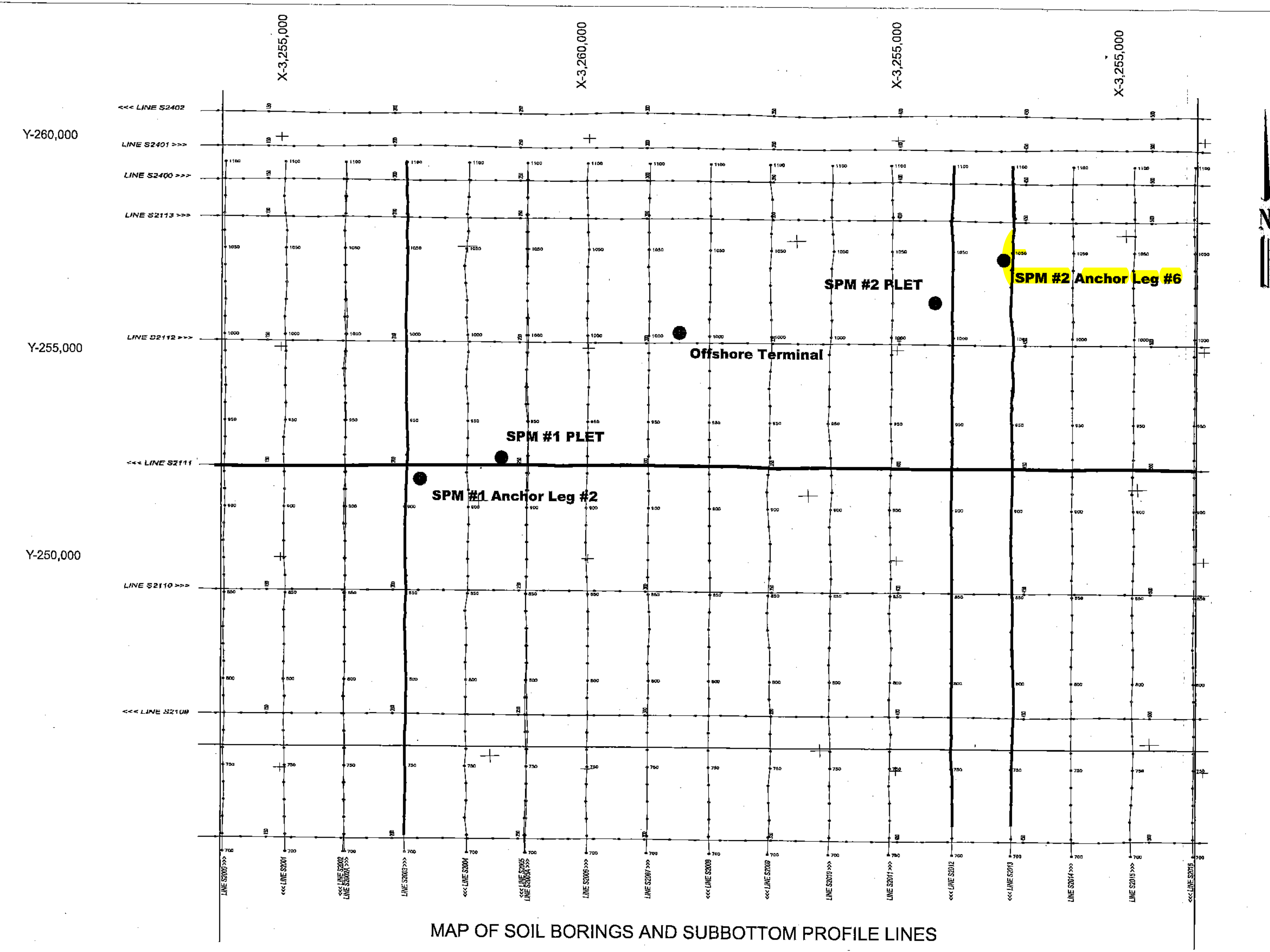
(1) ASTM D 2488

(2) ASCE Manual 56 (1976)

(3) ASTM D 2049

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

MB 4/5/08



MAP OF SOIL BORINGS AND SUBBOTTOM PROFILE LINES

Texas Offshore Port System

Block A-56, Galveston Area

3.4 SPM #2 ANCHOR LEG #6 LOCATION

3.4.1 Introduction

The field investigation at the location designated as SPM #2 ANCHOR LEG #6 was performed on June 30, 2008. Soil sampling was performed to 131-ft penetration at Texas South Central Zone Coordinates X = 3,266,759 ft and Y = 257,169 ft. The measured water depth was 117 ft.

3.4.2 Soil Stratigraphy

The soil stratigraphy disclosed by the field and laboratory investigations is presented on the boring log, Plate 3-34. The soil stratigraphy is based on the classification of soil samples recovered from the boring and observations made during drilling operations. A generalized summary of the major soil strata is tabulated below.

Stratum	Penetration, ft From	To	Description
I	0	6.5	Soft to stiff clay
II	6.5	19	Medium dense silty fine sand interlayered with firm to stiff lean clay
III	19	131	Firm to stiff lean clay

Detailed soil descriptions that include textural variations and inclusions are noted on the boring log. A key to the terms and symbols used on the boring log is presented on Plate 2-2. The Roman numeral representing each stratum is also shown on the boring log and on relevant plates. The variation in soil stratigraphy across this site is indicated in a comparison (integration) of the geophysical and geotechnical soil information presented on Plate 3-35.

3.4.2.1 Interpretation of Soil Properties

The shear strength and submerged unit weight profiles shown on Plates 3-36 and 3-37, respectively, best represent the assembled test results plotted on the boring log. These profiles were used in the engineering analyses.

3.4.3 Pile Design Information

The pile design information developed for this study includes ultimate axial capacities, axial load-pile movement data, and lateral soil resistance-pile deflection (p-y) characteristics. The analytical methods used to develop this information are presented briefly in Section 2.5 and in more detail in Appendix B.

3.4.3.1 Axial Pile Design

Ultimate Axial Capacity. The unit skin friction and unit end bearing values plotted on Plates 3-38 and 3-39, respectively, was calculated using the API RP 2A methods described in Appendix B. These values were used to calculate the ultimate axial compressive and tensile capacities for 42-in.-diameter pipe piles, driven to final penetration at the boring location. Capacity curves for driven pipe piles (conductors, caissons, anchor and foundation piles) are presented on Plate 3-40.

API RP 2A recommends that pile penetrations be selected using appropriate factors of safety or pile resistance factors. These factors are discussed in Section 2.5.1 of this report.

Axial Load Transfer Data. Axial load-pile movement analyses are usually performed using a computer solution based on methods developed by Reese (1964) or Matlock, et al. (1976). Plates 3-41 and 3-42 present the results as side load-side movement (t-z) and tip load-tip movement (Q-z) data for 42-in.-diameter driven pipe piles, respectively. The Q-z data should be used for foundation piles and neglected for caissons and conductor design. In developing the axial load transfer data in the cohesive soils, a post-peak adhesion ratio of 0.90 was utilized.

3.4.3.2 Lateral Pile Design Data

The soil resistance-pile deflection (p-y) characteristics of the soils at the boring location were developed for individual 42-in.-diameter driven pipe piles. These data may be used in lateral load analyses of driven piles, conductors and caissons. The p-y data for cyclic loading were developed to 100-ft penetration using procedures that have been outlined in API RP 2A and briefly explained in Appendix B. The stratigraphy and parameters used to develop the p-y data are presented on Plate 3-43. The p-y data for 42-in.-diameter driven pipe piles are presented on Plate 3-44. P-y values presented at 100-ft penetration may be used for lateral load analyses at greater depths.

SECTION 3.4

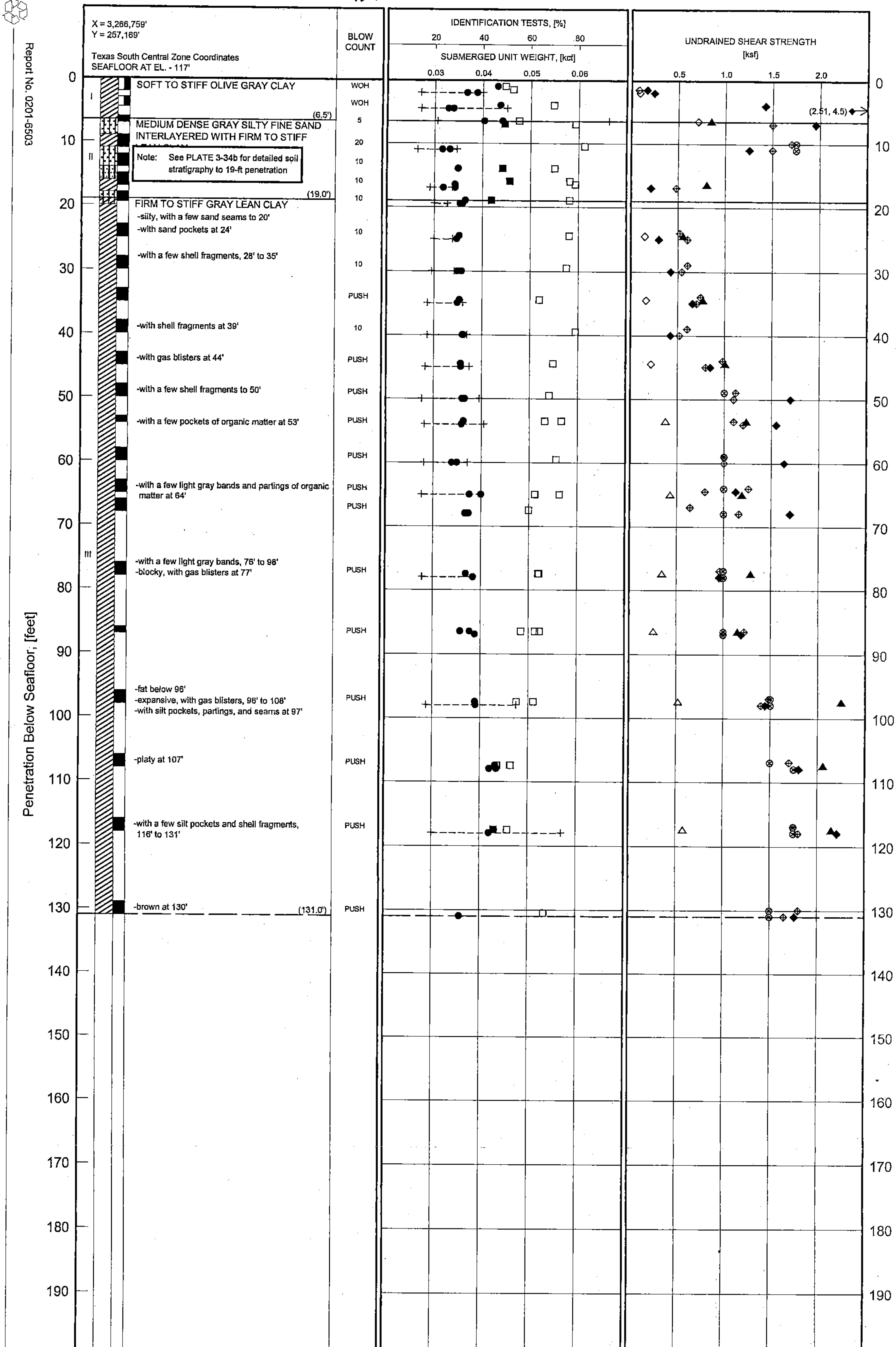
ILLUSTRATIONS

Checked By: Date: 7/9

Approved By: Date: 9/10/02

Drawn By: Date: 9/9

Report No. 0201-6503



SAMPLING TECHNIQUES

Number of blows of a 175-lb weight (hammer) dropped approximately 5 ft to produce a maximum of 24 in. of penetration of a 2.25-in.-OD, 2.125-in.-ID thin-walled tube sampler. "PUSH" denotes a 3.00-in.-OD, 2.83-in.-ID thin-walled tube sampler was advanced 24 in. with the weight of the drill string. "WOH" denotes a 2.50-in.-OD, 2.125-in.-ID liner sampler was advanced 24 in. with the weight of the hammer.

CLASSIFICATION TESTS

- ▼ SOLUBILITY IN HCL, [%]
- ■ PERCENT PASSING -200 SIEVE, [%]
- ● WATER CONTENT (W), [%]
- □ SUBMERGED UNIT WEIGHT (SUW)

PLASTIC LIMIT (PL) +-------+ LIQUID LIMIT (LL)

STRENGTH TESTS

- ⊗ POCKET PENETROMETER (PP)
- ⊕ TORVANE (TV)
- ◇ REMOTE VANE (RV)
- ◆ MINIATURE VANE (MV) (⊕ RESIDUAL (MVres) VALUE)
- ▲ UNCONSOLIDATED UNDRAINED TRIAXIAL (UU)

(Open symbols indicate remolded (r) tests)

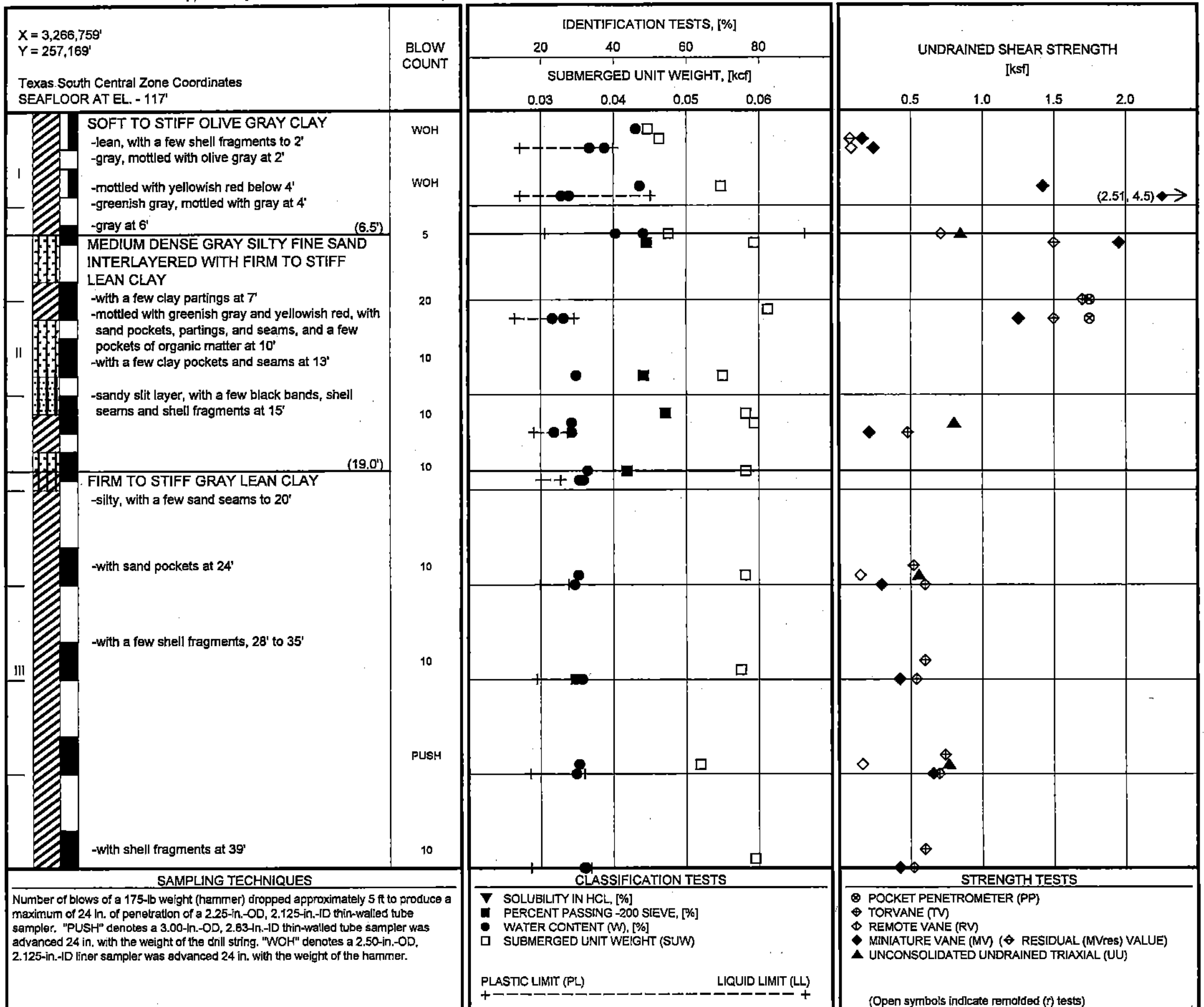
LOG OF BORING AND TEST RESULTS

Texas Offshore Port System, SPM #2 Anchor Leg #6

Block A-56, Galveston Area

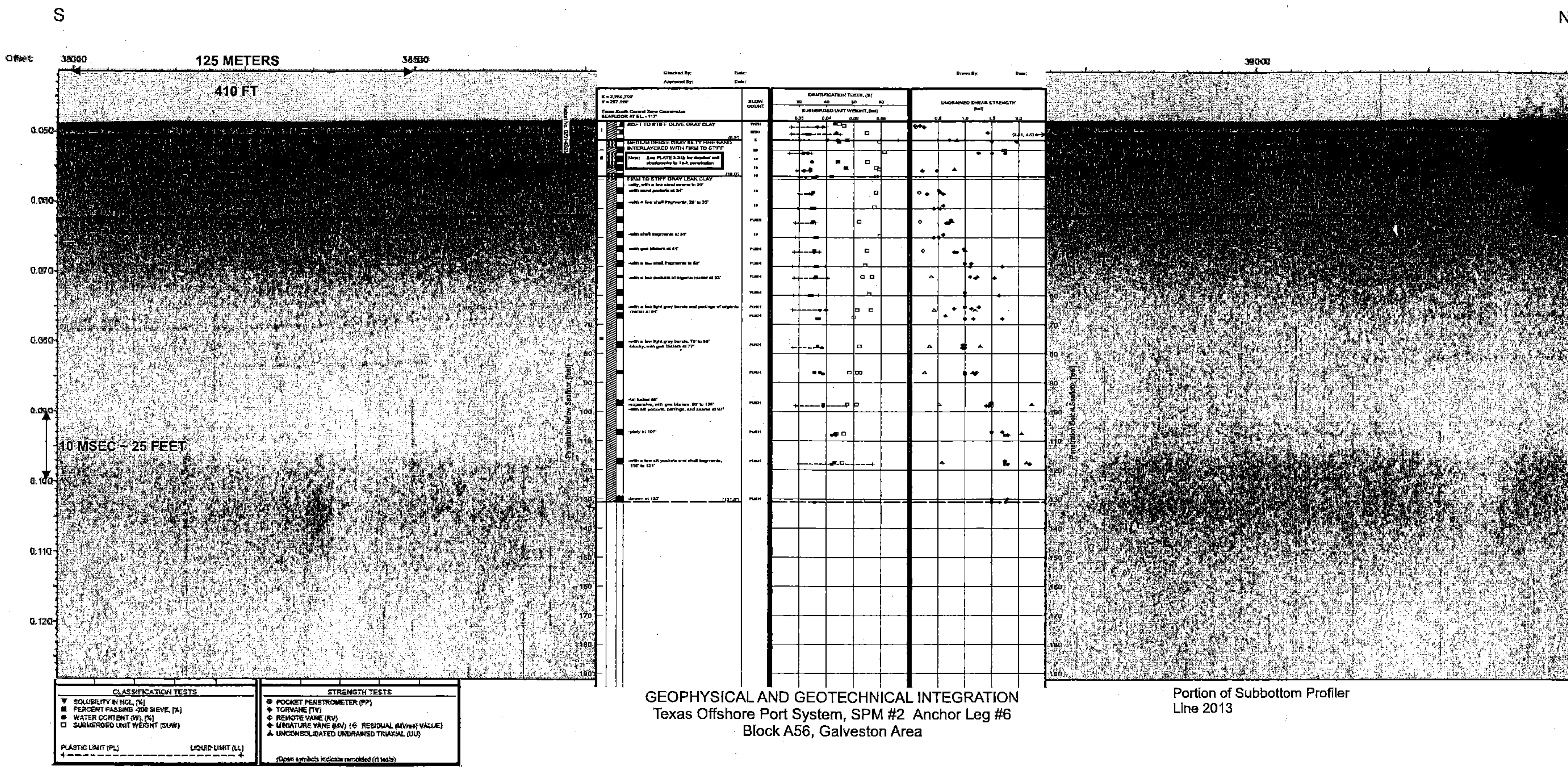
PLATE 3-34a

FUGRO



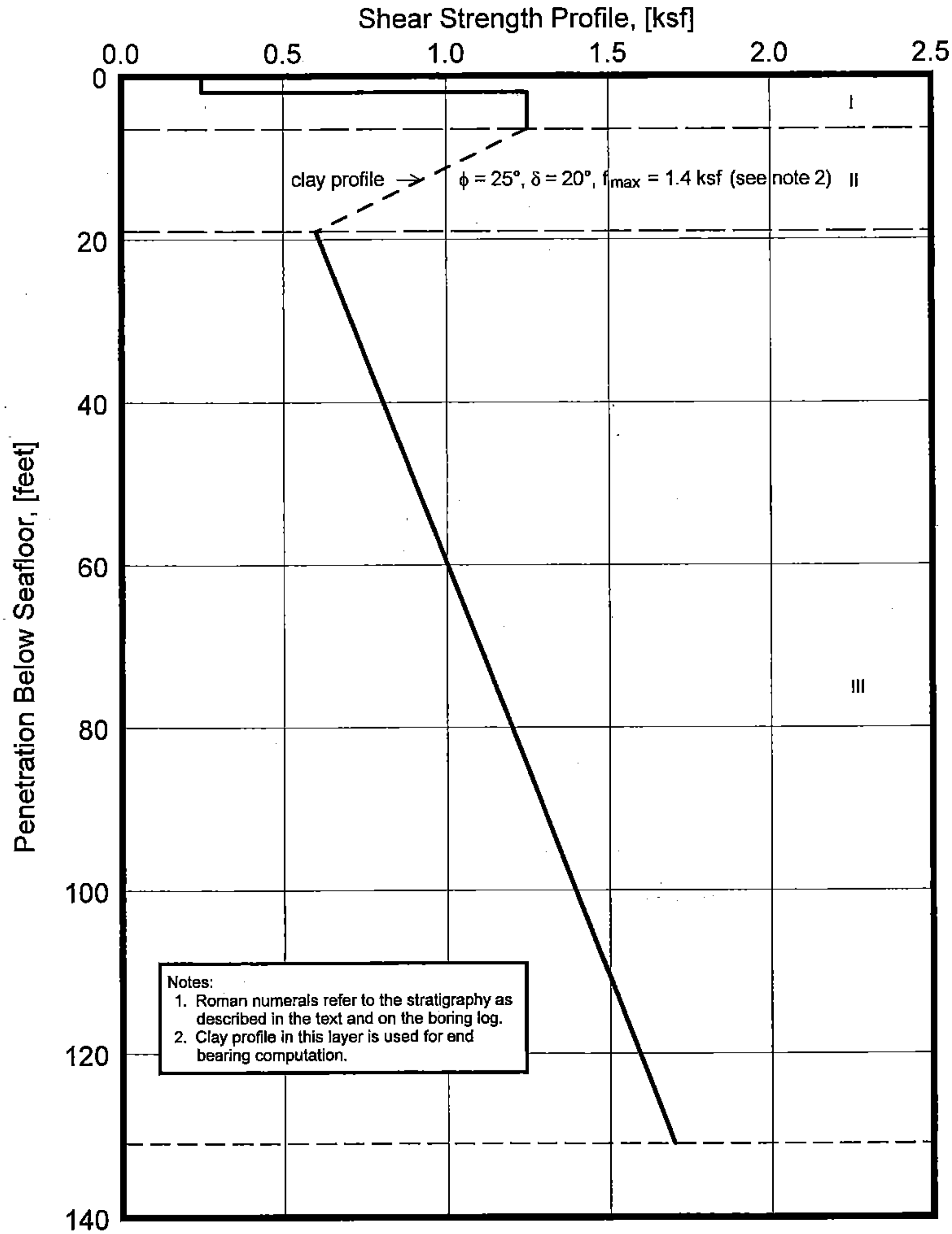
LOG OF BORING AND TEST RESULTS

Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area



GEOPHYSICAL AND GEOTECHNICAL INTEGRATION
Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A56, Galveston Area

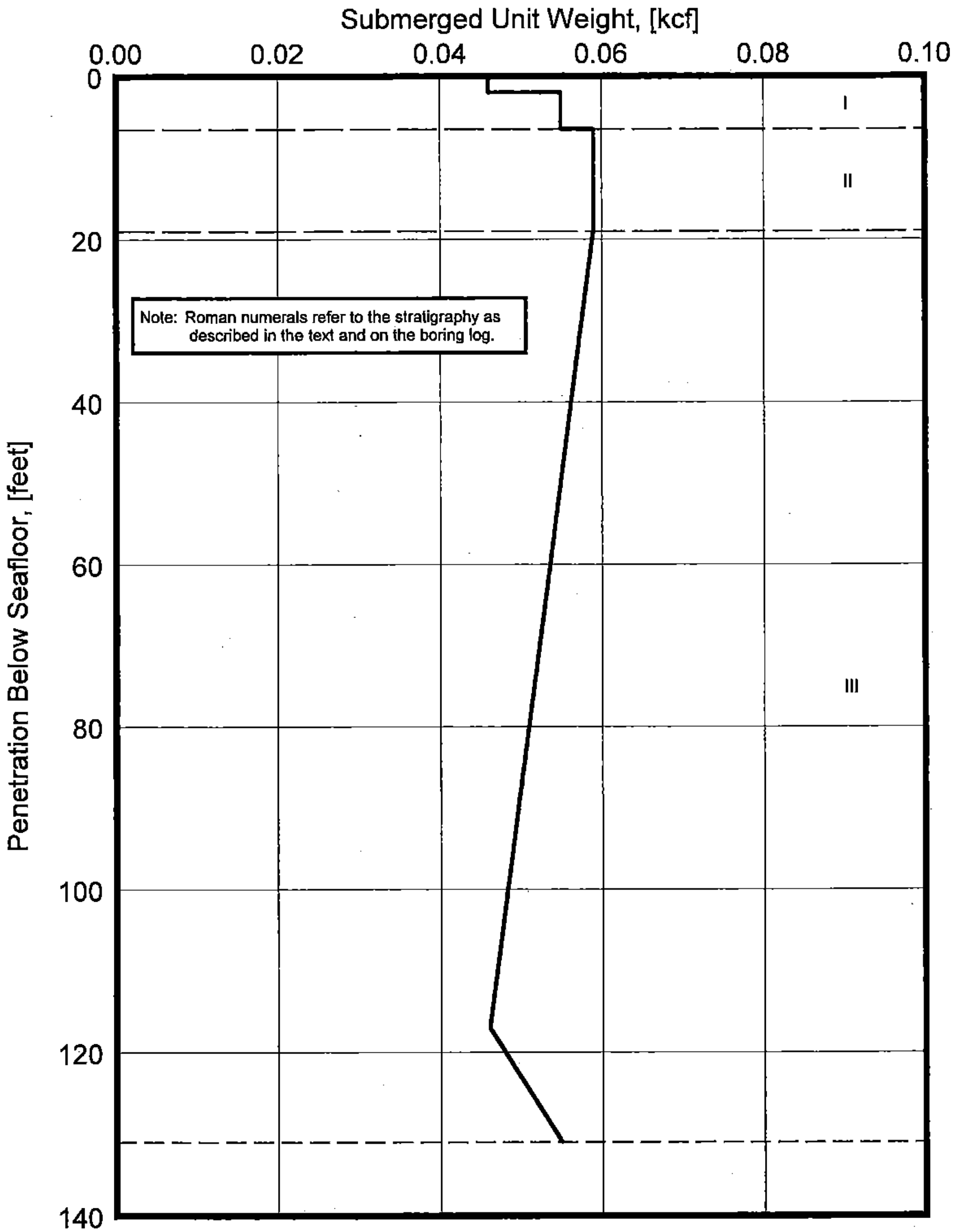
Portion of Subbottom Profiler
Line 2013



DESIGN STRENGTH PARAMETERS

Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

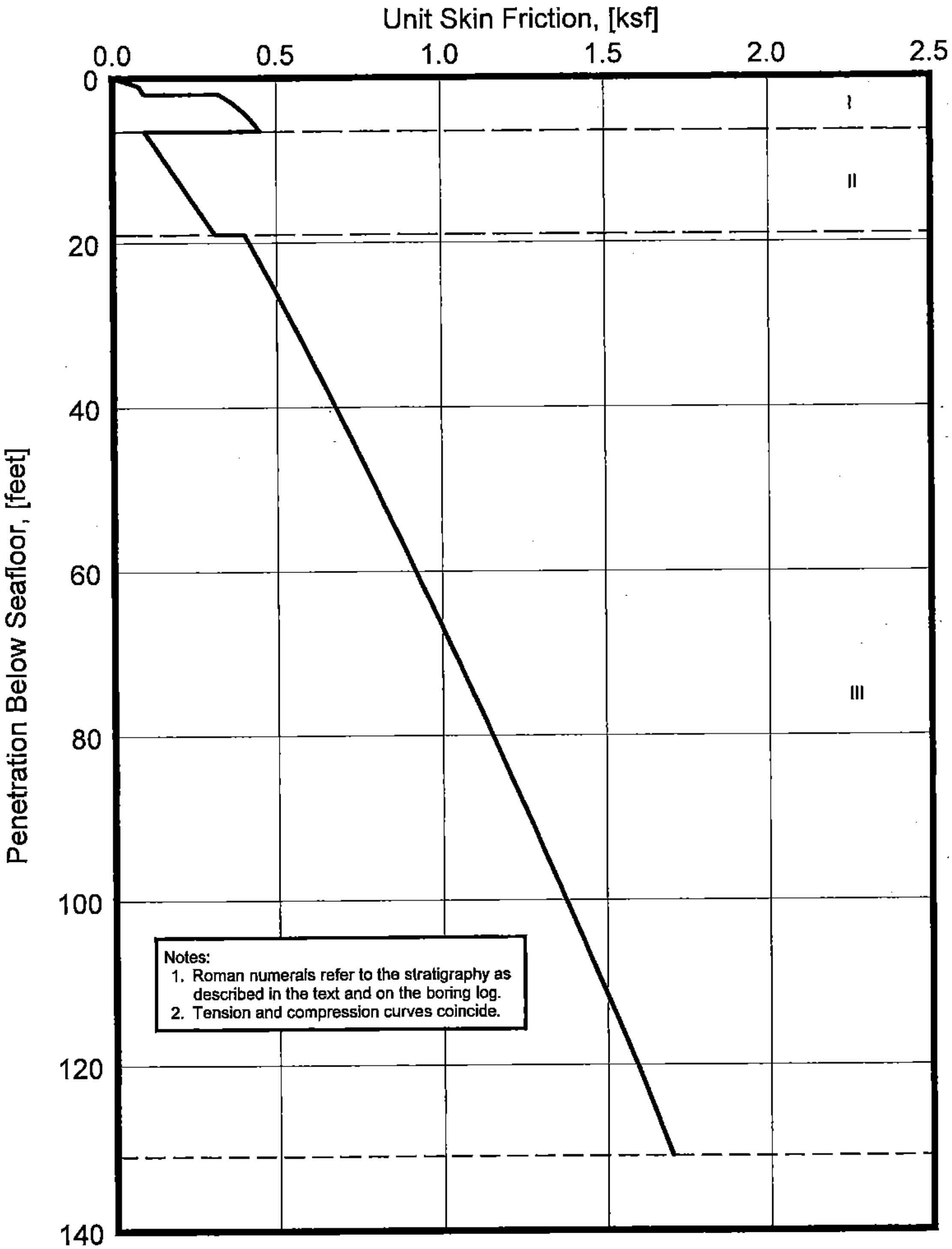
Drawn By: AW Date: 8/29/08
Checked By: NB Date: 9/3/08
Approved By: MR Date: 9/5/08



DESIGN SUBMERGED UNIT WEIGHT

Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

Drawn By: AW Date: 8/29/08
Checked By: MB Date: 9/3/08
Approved By: RL Date: 9/5/08



UNIT SKIN FRICTION
API RP 2A (2000) Method

Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

Drawn By: AW Date: 8/29/08
Checked By: AB Date: 9/3/08
Approved By: Date: 9/

PENETRATION BELOW MUDLINE (feet)		CURVE POINTS							
		1	2	3	4	5	6	7	8
0.0	t	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
1.0	t	0.00	0.02	0.04	0.06	0.07	0.08	0.07	0.07
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
2.0	t	0.00	0.03	0.05	0.07	0.09	0.10	0.09	0.09
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
2.0	t	0.00	0.10	0.16	0.24	0.29	0.33	0.29	0.29
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
4.0	t	0.00	0.12	0.20	0.30	0.36	0.40	0.36	0.36
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
6.5	t	0.00	0.14	0.23	0.34	0.41	0.45	0.41	0.41
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
6.5	t	0.00	0.10	0.10					
	z	0.00	0.10	42.00					
19.0	t	0.00	0.31	0.31					
	z	0.00	0.10	42.00					
19.0	t	0.00	0.12	0.20	0.30	0.36	0.40	0.36	0.36
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
70.0	t	0.00	0.31	0.52	0.78	0.93	1.04	0.93	0.93
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
117.0	t	0.00	0.47	0.78	1.17	1.40	1.56	1.40	1.40
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00
131.0	t	0.00	0.51	0.85	1.27	1.53	1.70	1.53	1.53
	z	0.00	0.07	0.13	0.24	0.34	0.42	0.84	42.00

Notes: 1. "t" is mobilized soil-pile adhesion, [ksf].
2. "z" is axial pile displacement, [in.].
3. Data for tension and compression coincide.

AXIAL LOAD TRANSFER DATA
(T-Z DATA)
API RP 2A (2000) Method
42-in.-Diameter Driven Pipe Piles
Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

Drawn By: AW Date: 8/29/08
Checked By: HB Date: 9/3/08
Approved By: [illegible] Date: 9/5/08

PENETRATION BELOW MUDLINE (feet)		CURVE POINTS						
		1	2	3	4	5	6	7
40.0	Q	0	17	35	52	63	70	70
	z	0.00	0.08	0.55	1.76	3.07	4.20	42.00
131.0	Q	0	37	74	110	132	147	147
	z	0.00	0.08	0.55	1.76	3.07	4.20	42.00

Notes: 1. "Q" is mobilized end bearing capacity, [kips].
2. "z" is axial tip displacement, [in.].

AXIAL LOAD TRANSFER DATA
(Q-Z DATA)
API RP 2A (2000) Method
42-in.-Diameter Driven Pipe Piles
Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

Drawn By: AW Date: 8/29/08
Checked By: NB Date: 9/3/08
Approved By: JR Date: 9/5/08

PENETRATION BELOW MUDLINE (feet)		CURVE POINTS							
		1	2	3	4	5	6	7	8
0.0	p	0	33	50	74	109	158	0	0
	y	0.00	0.06	0.21	0.63	2.10	6.30	31.50	42.00
2.0	p	0	40	61	91	133	192	21	21
	y	0.00	0.06	0.21	0.63	2.10	6.30	31.50	42.00
2.0	p	0	184	282	416	612	882	53	53
	y	0.00	0.06	0.21	0.63	2.10	6.30	31.50	42.00
6.5	p	0	230	352	521	766	1103	221	221
	y	0.00	0.06	0.21	0.63	2.10	6.30	31.50	42.00
6.5	p	0	116	193	254	320	366	381	385
	y	0.00	0.08	0.14	0.20	0.29	0.45	0.65	42.00
10.0	p	0	238	397	524	659	754	786	794
	y	0.00	0.10	0.18	0.26	0.39	0.61	0.88	42.00
14.0	p	0	427	711	939	1181	1352	1408	1423
	y	0.00	0.13	0.23	0.34	0.50	0.78	1.12	42.00
19.0	p	0	735	1226	1618	2035	2329	2427	2451
	y	0.00	0.17	0.30	0.43	0.64	0.98	1.42	42.00
19.0	p	0	197	302	447	657	946	711	711
	y	0.00	0.04	0.16	0.47	1.57	4.72	23.62	42.00
27.0	p	0	267	410	606	891	1283	1283	
	y	0.00	0.04	0.15	0.46	1.52	4.57	42.00	
100.0 (and below)	p	0	550	843	1246	1832	2638	2638	
	y	0.00	0.03	0.10	0.31	1.05	3.15	42.00	

Notes: 1. "p" is soil resistance, [lb/in.].
2. "y" is lateral deflection, [in.].

Drawn By: AW Date: 8/29/08
Checked By: MB Date: 9/3/08
Approved By: DL Date: 9/5/08

P-Y DATA
(CYCLIC LOADING)
API RP 2A (2000) Method
42-in.-Diameter Driven Pipe Piles
Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

4 CONCLUSIONS AND RECOMMENDATIONS

The TOPS geotechnical investigation program to investigate soil conditions at the proposed SPM #1 and SPM #2 facility locations located in Block A-56, of the Galveston Area in the Gulf of Mexico consisted of four soil borings, field and laboratory testing, and engineering analyses. A summary of the pertinent conclusions and recommendations follows:

- Soil borings across the proposed facility locations indicate a significant degree of near-surface soil variability. Soil conditions above 34-ft penetration show channel features within the block. These channel features vary both in depth and width across the block and result in variable soil stratigraphy and properties. FMMG recommends that a site-specific soil boring be completed at each of the anchor and PLET locations prior to design of the foundation elements.
- A scanning sonar survey was performed at each boring location and is available upon request from Fugro Chance.
- The water depth ranged from 117 ft to 121 ft across the boring locations within Block A-56 in the Galveston Area.
- Final engineering design data are presented for 24- and 42-in.-diameter driven pipe piles for the PLET and anchor locations, respectively.
- The safety and load resistance factors should be carefully reviewed based on API RP 2A guidelines and appropriately applied to the engineering analyses presented in this report.
- Pile group effects and pile interaction with spudcan depressions should be evaluated when the geometry and location of these elements are determined.
- Mud mat bearing capacities at the PLET locations should be reviewed when the final size and configurations and proximity to spudcan depressions are determined.
- Pile driving problems are not expected based on the soil information presented in this study but a drivability study could be performed to select an appropriate hammer-pile combination.

FMMG would be pleased to assist in re-evaluations and additional analyses.

Checked By: AW Date: 9/4/08
Approved By: [illegible] Date: 9/5/08

Drawn By: Tomol Date: 9/4/08

Report No. 0201-6503

Summary of Test Results

Job No.: 0201-6503-4 04-Sep-2008 (Ver. #5)
Boring: Texas Offshore Port System, SPM #2 Anchor Leg #6
Block: A-56
Area: Galveston

Sample No.	Depth (ft)	Identification Tests						Strength Estimate (ksf)		Miniature Vane Tests (ksf)			Compression Tests								
		Liquidity Index	Liquid Limit (%)	Plastic Limit (%)	Moisture Content (%)	Submerged Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer	Torvane	Undisturbed	Remolded	Residual	Type Test	Moisture Content (%)	Confining Pressure (psi)	Undisturbed Strength (ksf)	Remolded Strength (ksf)	ε_{50} Strain (%)	Submerged Unit Weight (pcf)	Failure Strain (%)	Type of Failure
1	1.00				46	45															
2	1.50					46				0.17		0.08									
3	2.00	.75	40	14	33					0.24											
3	2.00				38						0.09										
4	4.00				47	55				1.42											
5	4.50	.32	50	14	26																
5	4.50				28					2.51											
6	6.50	.38	93	21	48								UU	41	39	0.85		0.9	48	8	A
6	6.50										0.71										
7	7.00					59	49		1.50	1.96											
8	10.00							1.75	1.70												
9	10.50					61															
10	11.00	.82	29	13	26																
10	11.00				23			1.75	1.50	1.25											
11	14.00				30	55	48														
12	16.00					58	54														
13	16.50												UU	29	54	0.80		10.2	59	21	A
14	17.00				24				0.48	0.21											
14	17.00	1.11	27	18	29																
15	19.00				33	58	44														
16	19.50				31																
16	19.50	2.16	26	20	32																

NOTES:

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

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

Plus Signs [+] denote tests which exceeded the capacity of the measuring device.

NP = Non Plastic Material

FUGRO

PLATE A-8a

Checked By: AW Date: 9/4/08
Approved By: [illegible] Date: 9/5/08

Drawn By: Tomol Date: 9/4/08

Report No. 0201-6503

Summary of Test Results

Job No.: 0201-6503-4 04-Sep-2008 (Ver. #5)
Boring: Texas Offshore Port System, SPM #2 Anchor Leg #6
Block: A-56
Area: Galveston

		Identification Tests						Strength Estimate (ksf)		Miniature Vane Tests (ksf)			Compression Tests								
Sample No.	Depth (ft)	Liquidity Index	Liquid Limit (%)	Plastic Limit (%)	Moisture Content (%)	Submerged Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer	Torvane	Undisturbed	Remolded	Residual	Type Test	Moisture Content (%)	Confining Pressure (psi)	Undisturbed Strength (ksf)	Remolded Strength (ksf)	ε_{50} Strain (%)	Submerged Unit Weight (pcf)	Failure Strain (%)	Type of Failure
17	24.00								0.52												
18	24.50										0.15										
18	24.50												UU	30	65	0.56		6.2	58	20	A
19	25.00	1.21	28	20	29				0.60	0.30											
20	29.00								0.60												
21	29.50					58															
22	30.00				30				0.54	0.43											
22	30.00	1.33	28	19	31																
23	34.00								0.74												
24	34.50												UU	31	65	0.77		1.2	52	10	A
24	34.50										0.16										
25	35.00	.85	32	17	30				0.70	0.66											
26	39.00								0.60												
27	39.50					59															
28	40.00				32				0.52	0.43											
28	40.00	.91	34	17	32																
29	44.00								0.98												
30	44.50												UU	31	80	1.01		1.2	55	5	A
30	44.50										0.22										
31	45.00	.81	35	16	31				0.80	0.85											
32	49.00							1.00	1.12												
33	49.50					54															

NOTES:

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

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

Plus Signs [+] denote tests which exceeded the capacity of the measuring device.

NP = Non Plastic Material

FUGRO

Checked By: AW Date: 9/4/08
Approved By: m Date: 9/5/8

Drawn By: Tomol Date: 9/4/08

Report No. 0201-6503

Summary of Test Results

Job No.: 0201-6503-4 04-Sep-2008 (Ver. #5)
Boring: Texas Offshore Port System, SPM #2 Anchor Leg #6
Block: A-56
Area: Galveston

		Identification Tests						Strength Estimate (ksf)		Miniature Vane Tests (ksf)			Compression Tests								
Sample No.	Depth (ft)	Liquidity Index	Liquid Limit (%)	Plastic Limit (%)	Moisture Content (%)	Submerged Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer	Torvane	Undisturbed	Remolded	Residual	Type Test	Moisture Content (%)	Confining Pressure (psi)	Undisturbed Strength (ksf)	Remolded Strength (ksf)	ε_{50} Strain (%)	Submerged Unit Weight (pcf)	Failure Strain (%)	Type of Failure
34	50.00	.70	39	15	32																
34	50.00				33				1.10	1.70											
35	53.50												UU	32	121	1.23		1.0	53	4	AC
35	53.50								1.10				UU		121		0.38		57		
36	54.00	.62	41	16	32				1.20	1.55											
37	59.00							1.00	1.00												
38	59.50					56															
39	60.00				30				1.00	1.64											
39	60.00	.64	34	16	28																
40	64.00							1.00	1.26												
41	64.50								0.80	1.13											
42	65.00	.82	40	15	35								UU	40	121	1.19		1.2	51	9	AB
42	65.00												UU		121		0.43		56		
43	67.00								0.64												
44	67.50					50															
45	68.00				33			1.00	1.16	1.70											
45	68.00				35																
46	77.00							1.00	0.96												
47	77.50												UU		120		0.35		52		
47	77.50												UU	34	120	1.29		0.9	52	3	B
48	78.00	1.02	36	16	37			1.00	0.98	0.96											
49	86.50					48		1.00	1.22				UU	31	120	1.15		1.1	52	5	A

NOTES:

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

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

Plus Signs [+] denote tests which exceeded the capacity of the measuring device.

NP = Non Plastic Material

PLATE A-8c

FUGRO

Checked By: AW Date: 9/4/08
Approved By: ML Date: 9/5/08

Drawn By: Tomal Date: 9/4/08

Report No. 0201-6503

Summary of Test Results

Job No.: 0201-6503-4 04-Sep-2008 (Ver. #5)
Boring: Texas Offshore Port System, SPM #2 Anchor Leg #6
Block: A-56
Area: Galveston

		Identification Tests						Strength Estimate (ksf)		Miniature Vane Tests (ksf)			Compression Tests								
Sample No.	Depth (ft)	Liquidity Index	Liquid Limit (%)	Plastic Limit (%)	Moisture Content (%)	Submerged Unit Weight (pcf)	Passing No. 200 Sieve (%)	Penetrometer	Torvane	Undisturbed	Remolded	Residual	Type Test	Moisture Content (%)	Confining Pressure (psi)	Undisturbed Strength (ksf)	Remolded Strength (ksf)	ε_{50} Strain (%)	Submerged Unit Weight (pcf)	Failure Strain (%)	Type of Failure
49	86.50												UU	35	120		0.26		51		
50	87.00				38			1.00	1.00	1.19											
51	97.00							1.50	1.48												
52	97.50												UU	38	121	2.25		0.7	48	2	B
52	97.50												UU		121		0.52		51		
53	98.00	.55	55	17	38			1.50	1.40	1.45											
54	107.00							1.50	1.70												
55	107.50					44							UU	46	120	2.06		0.4	46	2	AB
56	108.00				47																
56	108.00				44			1.75	1.80	1.81											
57	117.00							1.75	1.75												
58	117.50												UU	46	121	2.15		0.8	43	3	B
58	117.50												UU		120		0.58		46		
59	118.00	.44	74	20	44			1.75	1.80	2.21											
60	130.00							1.50	1.80												
61	130.50					53															
62	131.00				32			1.50	1.65	1.76											

NOTES:

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

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

Plus Signs [+] denote tests which exceeded the capacity of the measuring device.

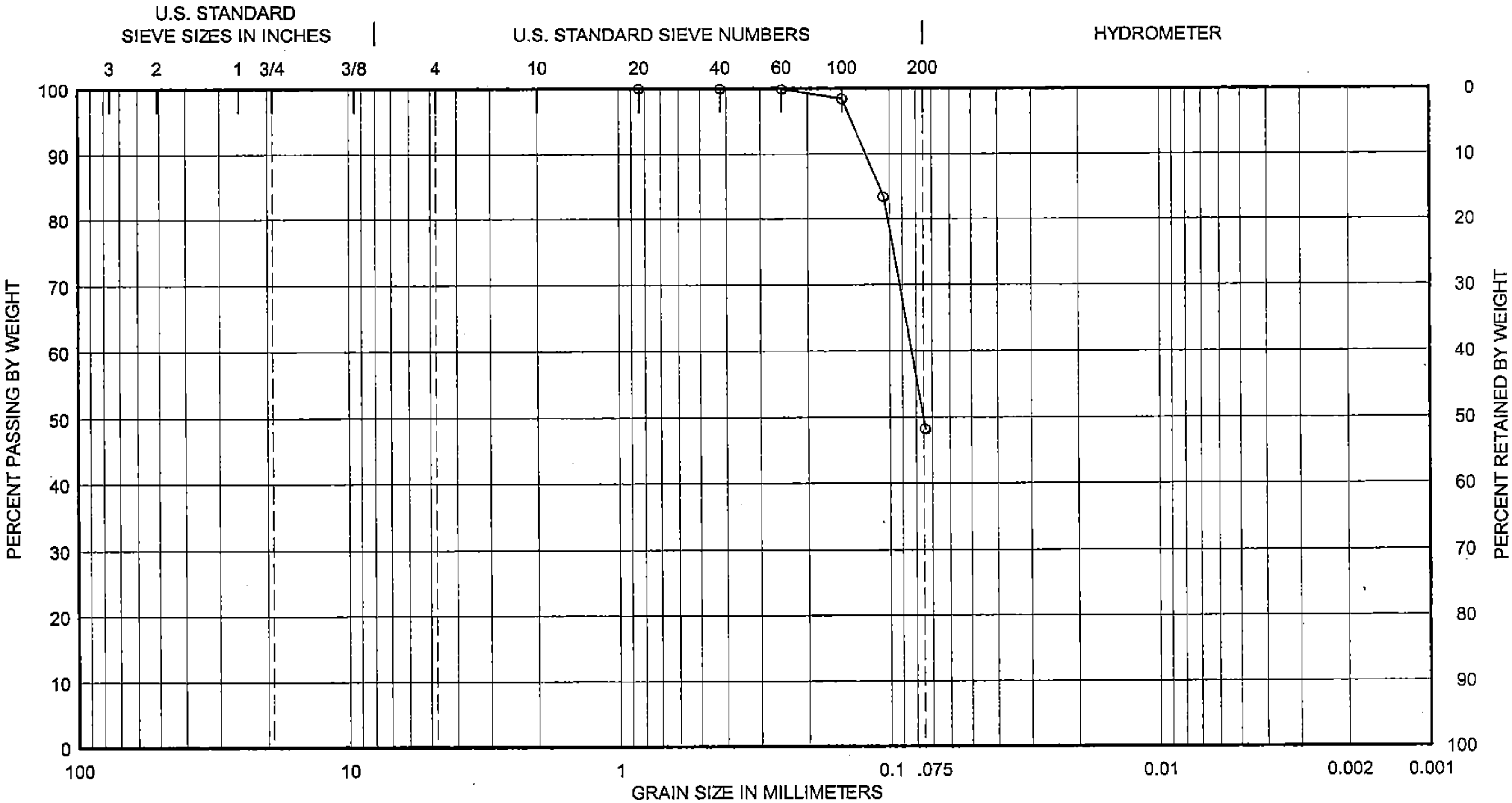
NP = Non Plastic Material

FUGRO

Checked by: AM Date: 7/4/08 Drawn by: Thomas Date: 7/4/08

Approved by: Date: 7/8/08

Report No. 0201-6503



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

SAMPLE NO.	DEPTH, FT	SYMBOL	CLASSIFICATION
11	14.00	o	SILTY FINE SAND (SM) with a few clay pockets and seams

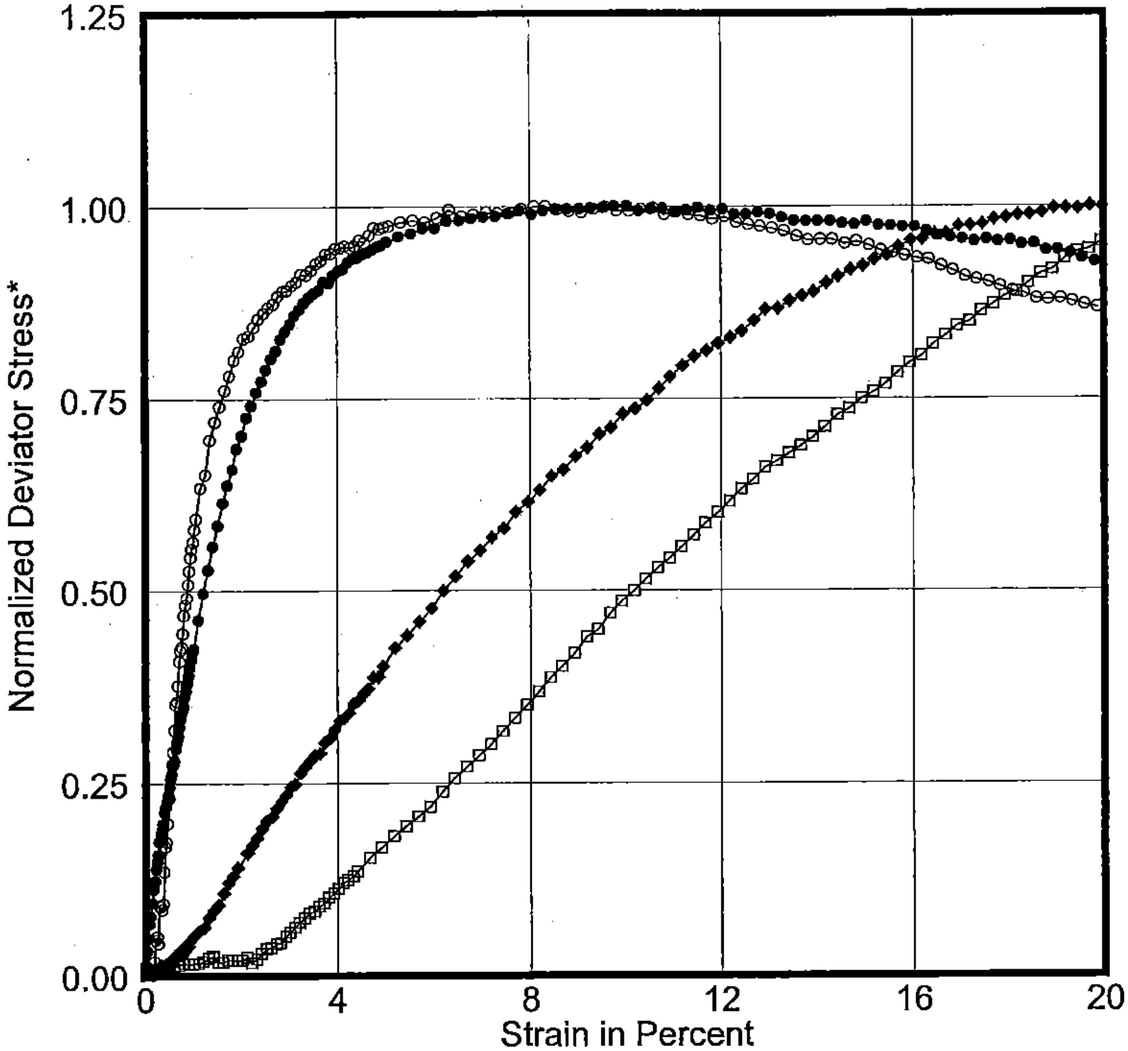
PLATE A-12

GRAIN-SIZE DISTRIBUTION CURVES

Texas Offshore Port System, SPM #2 Anchor Leg #6

Block A-56, Galveston Area

FUGRO



Curve	Sample No.	Depth [ft]	Test Type	Confining Pressure [psi]	Maximum Deviator Stress [ksf]	ε_{50} [%]
○—○	6	6.50	UU	39.4	1.70	0.9
□—□	13	16.50	UU	54.2	1.61	10.2
◆—◆	18	24.50	UU	64.7	1.11	6.2
●—●	24	34.50	UU	64.9	1.53	1.2

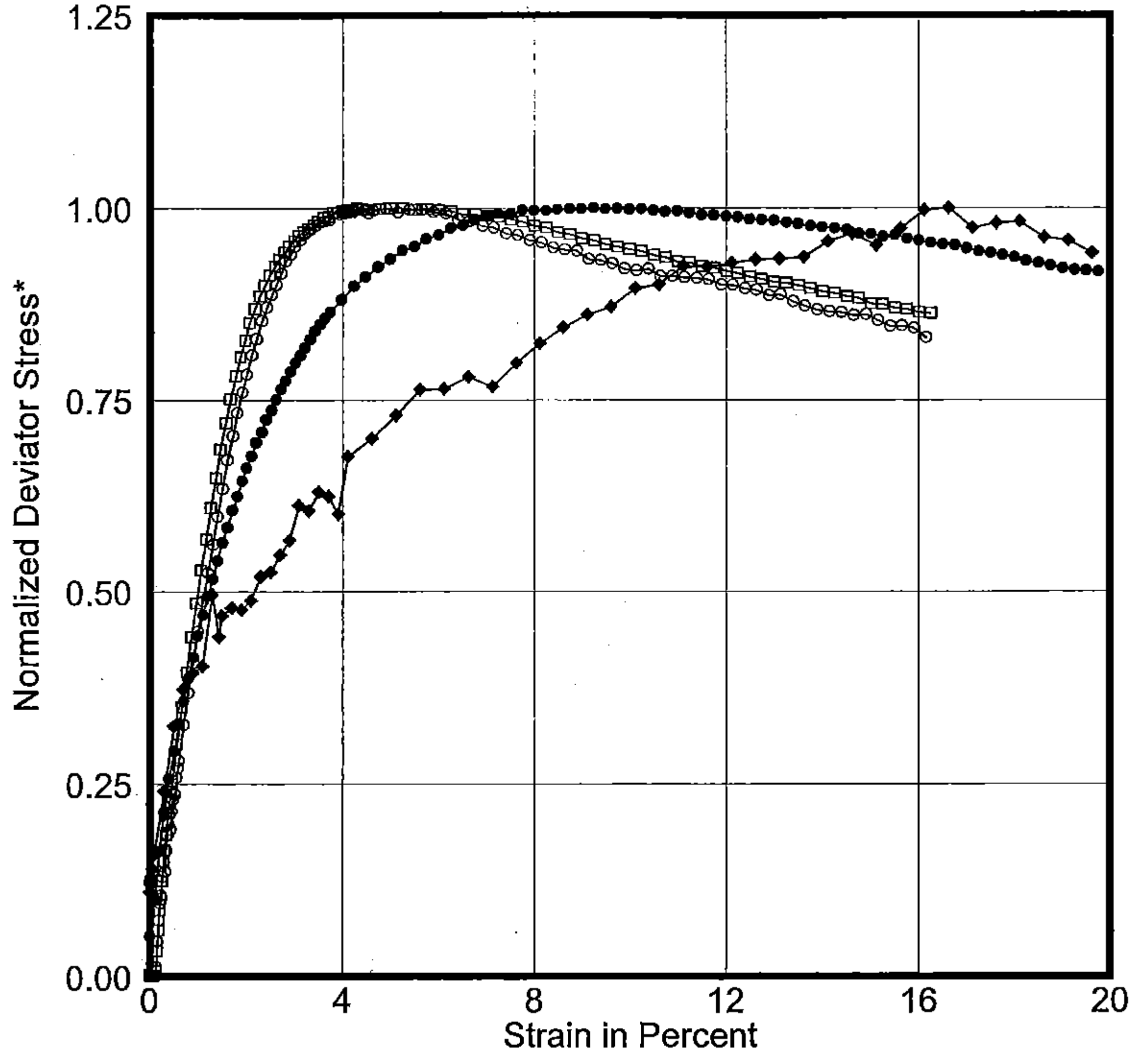
* Normalized with respect to maximum deviator stress.

STRESS-STRAIN CURVES
Unconsolidated-Undrained Triaxial Compression Test

Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

Drawn By: Tom T Date: 9/4/08
Checked By: [illegible] Date: 9/4/08
Approved By: [illegible] Date: 9/[illegible]

Drawn By: Tomo Date: 9/4/08
Checked By: Date: 9/4/08
Approved By: Date: 9/5/08



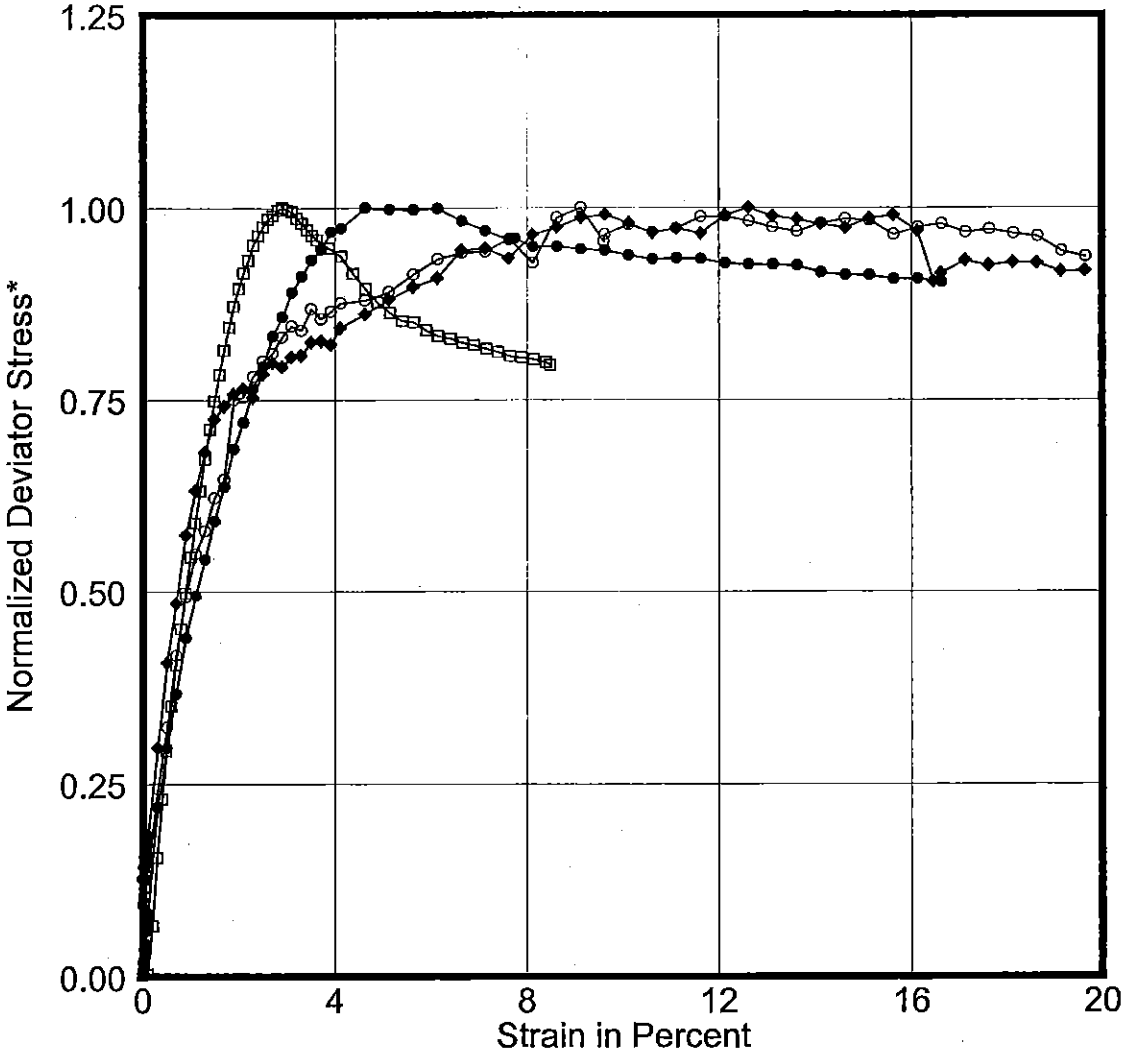
Curve	Sample No.	Depth [ft]	Test Type	Confining Pressure [psi]	Maximum Deviator Stress [ksf]	ε50 [%]
○—○	30	44.50	UU	80.3	2.01	1.2
□—□	35	53.50	UU	121.2	2.47	1.0
◆—◆	35	53.50	UUr	121.2	0.76	2.2
●—●	42	65.00	UU	120.8	2.38	1.2

* Normalized with respect to maximum deviator stress.

STRESS-STRAIN CURVES

Unconsolidated-Undrained Triaxial Compression Test

Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area



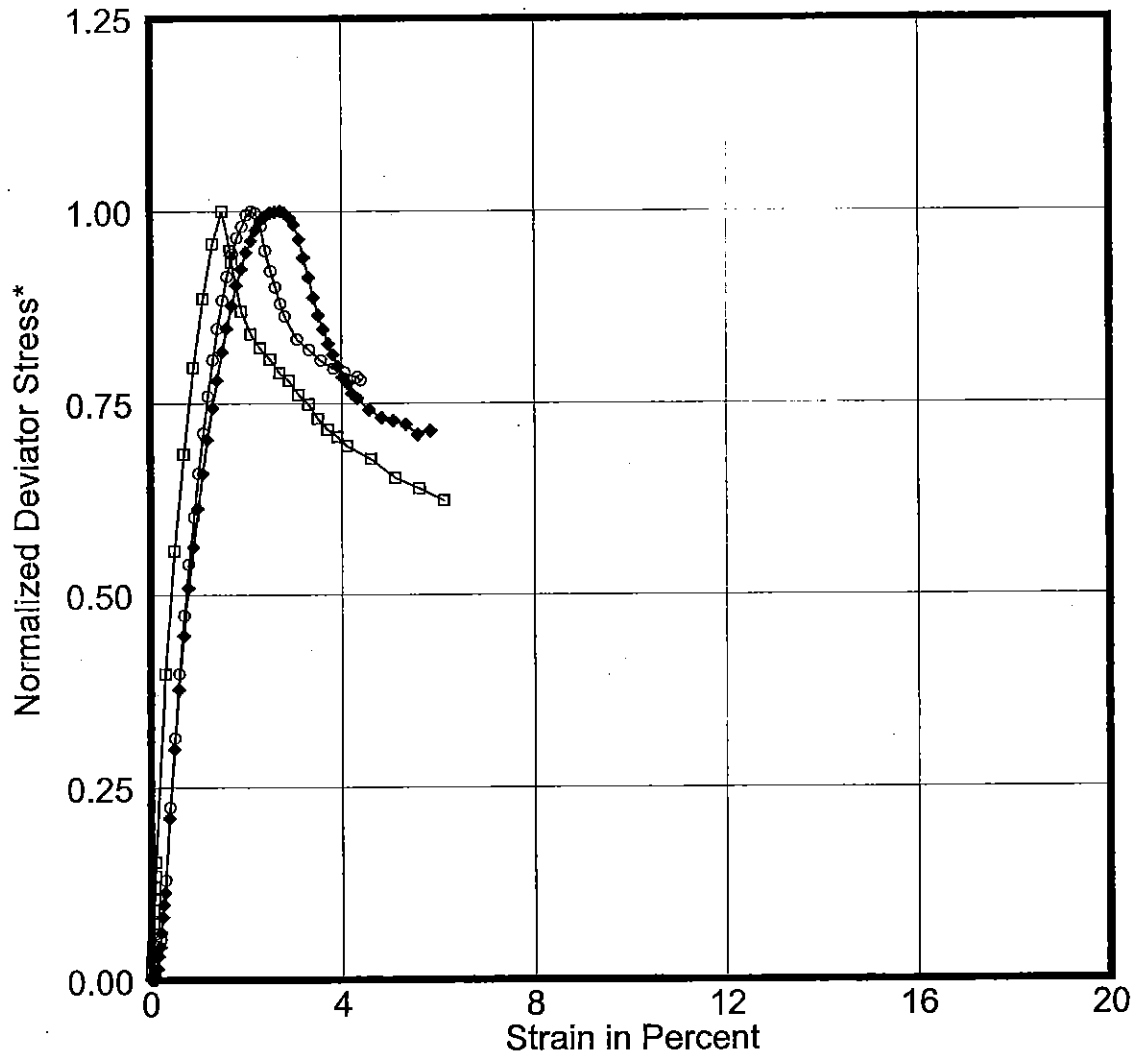
Curve	Sample No.	Depth [ft]	Test Type	Confining Pressure [psi]	Maximum Deviator Stress [ksf]	ε50 [%]
o———o	42	65.00	UUr	121.2	0.86	0.9
□———□	47	77.50	UU	120.5	2.58	0.9
◆———◆	47	77.50	UUr	120.1	0.70	0.7
●———●	49	86.50	UU	120.1	2.30	1.1

* Normalized with respect to maximum deviator stress.

STRESS-STRAIN CURVES
Unconsolidated-Undrained Triaxial Test

Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

Drawn By: TTomol Date: 9/4/08
Checked By: [illegible] Date: 9/17/08
Approved By: [illegible] Date: 9/5/08



Curve	Sample No.	Depth [ft]	Test Type	Confining Pressure [psi]	Maximum Deviator Stress [ksf]	ε_{50} [%]
o———o	52	97.50	UU	120.6	4.49	0.7
□———□	55	107.50	UU	120.4	4.12	0.4
◆———◆	58	117.50	UU	121.3	4.30	0.8

* Normalized with respect to maximum deviator stress.

STRESS-STRAIN CURVES
Unconsolidated-Undrained Triaxial Compression Test

Texas Offshore Port System, SPM #2 Anchor Leg #6
Block A-56, Galveston Area

Drawn By: Tomal Date: 9/4/08
Checked By: HW Date: 9/4/08
Approved By: JH Date: 9/5/08

Report No. 0201-6503

PLATE A-16d

ANALYTICAL PROCEDURES
CONTENTS

Page

ILLUSTRATIONS

Plate

CRITERIA FOR AXIAL PILE LOAD ANALYSIS

In this report, the word "pile" is used as a generic term for foundation piles, caissons and conductors. The installation of caissons and conductors is the same as that of foundation piles, except that the soil plug is later removed or disturbed, thus reducing the end bearing component. For this reason, the end bearing of caissons and conductors is neglected in capacity computations.

Method of Analysis

The static method of computing axial pile capacity described in API RP 2A (2000) is used to compute ultimate compressive and tensile capacities of pipe piles installed to a given penetration. In this method, the ultimate compressive capacity, Q, for a given penetration is taken as the sum of the skin friction on the pile wall, Q_s, and the end bearing on 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 pile end area; f and q represent, respectively, the unit skin friction and unit end bearing. Procedures used to compute values of f and q are discussed in the following paragraphs. When computing ultimate tensile capacity or compressive capacity of conductors and caissons, the end bearing term in the above equation is neglected.

Unit Skin Friction

Cohesive Soils. Computation of Q_s for pipe piles embedded in cohesive soils is done in accordance with the API RP 2A recommendations. According to API RP 2A-WSD (2000), Sec. 6.4.2 or API RP 2A-LRFD (1993), Sec. G.4.2, the unit skin friction may be expressed as:

$$f = \alpha S_u$$

where: α = a dimensionless factor; and

S_u = undrained shear strength of the soil at the point in question.

The factor α is computed by:

$$\alpha = 0.5\,\psi^{-0.5} \quad \text{for } \psi \le 1.0\text{, or}$$

$$\alpha = 0.5\,\psi^{-0.25} \quad \text{for } \psi > 1.0$$

with the constraint that $\alpha \le 1.0$,

where: ψ = S_u/σ'_v for the point in question, and

σ'_v = effective vertical stress at the point in question.

The undrained shear strength used in our computations and the values of submerged unit weight used to compute effective vertical stress are presented in the main report along with the resulting skin friction values.

Granular Soils. The procedure recommended by API RP 2A-WSD (2000), Sec. 6.4.3 or API RP 2A-LRFD (1993), Sec. G.4.3 is used to determine unit skin friction in granular soils. Unit skin friction, f, for granular soils is computed from the expression:

$$f = K \sigma'_v \tan \delta$$

where: K = coefficient of lateral earth pressure,

σ'_v = effective vertical stress, and

δ = angle of friction between soil and pile.

API RP 2A recommends values for K of 0.8 for open-ended pipe piles driven unplugged, and 1.0 for full displacement piles (plugged or close-ended).

API RP 2A presents recommended values for δ, the angle of friction acting between the soil and pile and specifies limiting values of skin friction. The recommended values for granular deposits composed primarily of silica are related to the density and composition of the granular deposits, and are presented on Plate B-1.

Unit End Bearing

Cohesive Soils. The procedure recommended by API RP 2A-WSD (2000), Sec. 6.4.2 or API RP 2A-LRFD (1993), Sec. G.4.2 is used to determine unit end bearing, q, in clays. Unit end bearing in clays can be estimated by the following equation:

$$q = 9\ S_u$$

where: S_u = undrained shear strength.

Granular Soils. Unit end bearing in granular soils is computed by API RP 2A-WSD (2000), Sec. 6.4.3 or API RP 2A-LRFD (1993), Sec. G.4.3 using the expression:

$$q = \sigma'_v N_q$$

where: σ'_v = effective vertical stress, and

N_q = a dimensionless bearing capacity factor that is a function of ϕ, the angle of internal friction of the material.

Recommended bearing capacity factors, N_q, for granular soils composed primarily of silica are given in API RP 2A and are presented on Plate B-1. Also shown on Plate B-1 are limiting unit end bearing values.

Equivalent Unit End Bearing. For open-ended driven pipe piles, the end bearing is limited to the frictional resistance of a soil plug developed inside the pile. The total skin friction on the inside of the pile is assumed equal to the total skin friction on the outside of the pile. Any influence of the driving shoe on the internal skin friction is neglected. The end bearing on the steel end area of the pile is also neglected. The assumptions made in the analyses make no difference in the unit end bearing below the point where the pile plugs (i.e., equivalent unit end bearing becomes equal to unit end bearing). Above this point, the unit end bearing is limited by the frictional resistance of the soil plug.

CRITERIA FOR AXIAL LOAD TRANSFER DATA

An axial load-pile movement analysis requires load transfer data on the skin friction along the side of the pile (t-z data) and the end bearing on the pile tip (Q-z data). Recommended procedures are given in API RP 2A-WSD (2000), Sec. 6.7 or API RP 2A-LRFD (1993), Sec. G.7.

Side Friction Versus Pile Movement Data

Axial side load transfer curves are different for cohesive soils (clay) and granular soils (sand). Typical axial side load transfer-displacement (t-z) curves for both material types are illustrated on Plate B-2 and discussed below.

Cohesive Soils. The side friction versus pile movement (t-z) curve for cohesive soils is given in API RP 2A-WSD (2000), Sec. 6.7.2 or API RP 2A-LRFD (1993), Sec. G.7.2, and is the same for compressive and tensile loading. The maximum side friction, t_{max}, at the pile-soil interface is taken as the ultimate skin friction, f, as determined by API RP 2A-WSD (2000), Sec. 6.4.2 or API RP 2A-LRFD (1993), Sec. G.4.2.

The post peak adhesion ratio for clays can range from 0.90 to 0.70 for highly plastic, normally consolidated clays, to as low as 0.50 for low plasticity, highly overconsolidated clays. The recommended adhesion ratios beyond peak values for static loading conditions are given in the report text.

Granular Soils. The side friction versus pile movement (t-z) curve for granular soils is presented in API RP 2A-WSD (2000), Sec. 6.7.2 or API RP 2A-LRFD (1993), Sec. G.7.2. The maximum side friction, t_{max}, at the pile-soil interface is the ultimate unit skin friction, f, determined by API RP 2A-WSD (2000), Sec. 6.4.3 or API RP 2A-LRFD (1993), Sec. G.4.3.

Tip Load Versus Tip Movement Data

Relatively large axial movements may be required to mobilize full end bearing resistance. End bearing or tip load increases with displacement of the pile tip. The development of full end bearing occurs at a displacement equal to 10 percent of the pile diameter according to API RP 2A. The tip load versus tip movement curve is given in API RP 2A-WSD (2000), Sec. 6.7.3 or API RP 2A-LRFD (1993), Sec. G.7.3. The end bearing component should not be considered when tensile loads are applied to a pile. Typical pile tip-load-displacement (Q-z) curves are presented in Plate B-3.

CRITERIA FOR LATERAL SOIL RESISTANCE-PILE DEFLECTION DATA

API RP 2A recommends that pile foundations be designed for lateral loading conditions. The lateral soil structure interaction is complex and the soil response to lateral loading is generally nonlinear. To analyze this complex interaction, a computer program based on the finite difference or finite element method is normally used. The nonlinear soil response is input into these methods with lateral soil resistance-pile deflection (p-y) curves. The methods for constructing p-y curves follow.

Cohesive Soils

Soil resistance-pile deflection (p-y) data for cohesive soils are developed using the procedure outlined by Matlock (1970) for soft clays subjected to cyclic loads and adopted by API RP 2A-WSD (2000), Sections 6.8.2 and 6.8.3. Interpreted shear strengths, submerged unit weights, and strain values at onehalf the maximum deviator stress (ε_{50}) used in our computations are presented in the text illustrations. These strain values were selected based on data from unconsolidated-undrained triaxial compression tests.

The ultimate lateral soil resistance (p_{us}) increases from $3S_uD$ to $9S_uD$ as X increases from 0 to X_R according to:

$$p_{us} = 3S_uD + \gamma XD + JS_uX$$

and

$$p_{ud} = 9S_uD \text{ for } X \geq X_R$$

where:

p_u = ultimate resistance (s = shallow, d = deep),

S_u = undrained shear strength for undisturbed clay soil samples,

D = pile diameter,

γ = effective unit weight of soil,

J = dimensionless empirical constant with values ranging from 0.25 to 0.5 having been determined by field testing. A value of 0.5 is appropriate for Gulf of Mexico clays,

X = depth below soil surface, and

X_R = depth below soil surface to bottom of reduced resistance zone.

The deflection values (y) are a function of the pile diameter and ε_{50}. Typical curve shapes are shown on Plate B-4.

Granular Soils

Soil resistance-pile deflection (p-y) data for granular soils are developed using the procedure outlined by O'Neill and Murchison (1983) for sands subjected to cyclic loading and adopted by API RP 2A-WSD (2000) in Sections 6.8.6 and 6.8.7. Input parameters include submerged unit weight, angle of internal friction, and the initial modulus of horizontal subgrade reaction. These values are presented in the text illustrations. Values of initial modulus of subgrade reaction are selected from the recommendations in API RP 2A based on our interpretation of the soil relative density from sampler driving resistance records and grain size analyses.

At a given depth, the following equation giving the smallest value of p_u should be used as the ultimate lateral bearing capacity in granular soils.

$$p_{us} = (C_1 H + C_2 D) \gamma H$$

and

$$p_{ud} = C_3 D \gamma H$$

where: p_u = ultimate resistance (s = shallow, d = deep),

γ = effective unit weight of soil,

H = depth,

C_1, C_2, C_3 = coefficients, and

D = average pile diameter from surface to depth.

The shape of the p-y curve in granular soil is defined by the following equation:

$$P = A\, p_u \tanh \left[\frac{k\,H}{A\, p_u}\, y \right]$$

where: A = factor to account for cyclic or static loading condition,

p_u = ultimate bearing capacity at depth H,

k = initial modulus of subgrade reaction,

y = lateral deflection, and

H = depth.

The shape of typical granular p-y curves is illustrated on Plate B-4.

SCOUR EFFECTS

Whenever the near-surface soils are comprised of granular material, they may be susceptible to scour. Scour effects are considered insignificant to axial capacity but can have a large influence on lateral capacity. When scour is considered likely, the p-y data are reduced to reflect the potential loss of lateral support from the material scoured away near the seafloor around the pile. General scour indicates that installation of the structure may cause a layer of material to be removed throughout the area of the platform. Local scour indicates that scour is likely to occur only in the near vicinity of the piles.

SERVICE WARRANTY

The "Service Warranty" outlines the limitations and constraints of this report in terms of a range of considerations including, but not limited to, its purpose, its scope, the data on which it is based, its use by third parties, possible future changes in design procedures and possible changes in the conditions at the site with time. This section represents a clear description of the constraints which apply to all reports issued by FMMG. It should be noted that the Service Warranty does not in any way supersede the terms and conditions of the contract between FMMG and the Client.

1. This report and the assessment carried out in connection with the report (together the "Services") were compiled and carried out by Fugro-McClelland Marine Geosciences, Inc. (FMMG) for the Client in accordance with the terms of the Contract. Further, and in particular, the Services were performed by FMMG taking into account the limits of the scope of works required by the Client, the time scale involved, and the resources, including financial and manpower resources, agreed between FMMG and the Client. FMMG has not performed any observations, investigations, studies or testing not specifically set out or required by the Contract between the Client and FMMG.

2. The Services were performed by FMMG exclusively for the purposes of the Client. Should this report or any part of this report, or details of the Services or any part of the Services be made known to any third party, such third party shall not rely on the report unless FMMG provides guidance required to interpret the report, i.e., respond to non-operational questions. If such third party does rely on the report without obtaining FMMG's guidance, it does so wholly at its sole risk and FMMG disclaims all liability resulting from third party use of the report.

3. It is FMMG's understanding that this report is to be used for the purpose described in the report. That purpose was a significant factor in determining the scope and level of the Services. Should the purpose for which the report is used, and/or should the Client's proposed development or use of the site change (including in particular any change in any design and/or specification relating to the proposed use or development of the site), this report may no longer be valid or appropriate and any further use of, or reliance upon, the report in those circumstances by the Client without FMMG's review and advice shall be at the Client's sole and own risk. Should FMMG be requested, and FMMG agree, to review the report after the date hereof, FMMG shall be entitled to additional payment at the then existing rates or such other terms as may be agreed between FMMG and Client.

4. The passage of time may result in changes (whether man-made or otherwise) in site conditions and changes in regulatory or other legal provisions, technology, methods of analysis, or economic conditions, which could render the information and results presented in the report inaccurate or unreliable. The information, recommendations and conclusions contained in this report should not be relied upon if any such changes have taken place, without the written agreement of FMMG. In the absence of such written agreement of FMMG, reliance on the report after any such changes have occurred shall be at the Client's own and sole risk. Should FMMG agree to review the report after such changes have taken place, FMMG shall be entitled to additional payment at the then existing rates or such other terms as may be agreed between FMMG and the Client.

5. Where the Services have involved FMMG's interpretation and/or other use of any information (including documentation or materials, analyses, recommendations and conclusions) provided by third parties (including independent testing and/or information, services or laboratories) or the Client and upon which FMMG was reasonably entitled to rely or involved FMMG's observations of existing physical conditions of any site involved in the Services, then the Services clearly are limited by the accuracy of such information and the observations which were reasonably possible of the said site. Unless otherwise stated, FMMG was not authorized and did not attempt to independently verify the accuracy or completeness of such information, received from the Client or third parties during the performance of

the Services. FMMG is not liable for any inaccuracies (including any incompleteness) in the said information, save as otherwise provided in the terms of the contract between the Client and FMMG.

6. The soil and ground conditions information provided in the Services are based solely on evaluations of the soil and ground condition samples (and in situ tests) at determined sample test locations and elevations. That information cannot be extrapolated to any area or elevation outside those locations and elevations unless specifically so stated in the report. In the light of the information available to FMMG, the soil and ground conditions information is considered appropriate for use in relation to the geotechnical design and installation aspects of the structures addressed in the report, but they may not be appropriate for the design of other structures.

APPENDIX B

ILLUSTRATIONS

Density	Soil Description	Soil-Pile Friction Angle, δ Degrees	Limiting Skin Friction Values kips/ft^2(kPa)	N_q	Limiting Unit End Bearing Values kips/ft^2(MPa)
Very Loose Loose Medium	Sand Sand-Silt* Silt	15	1.0 (47.8)	8	40 (1.9)
Loose Medium Dense	Sand Sand-Silt* Silt	20	1.4 (67.0)	12	60 (2.9)
Medium Dense	Sand Sand-Silt*	25	1.7 (81.3)	20	100 (4.8)
Dense Very Dense	Sand Sand-Silt*	30	2.0 (95.7)	40	200 (9.6)
Dense Very Dense	Gravel Sand	35	2.4 (114.8)	50	250 (12.0)

* Sand-Silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

Note: API RP 2A notes that the parameters listed above are intended as guidelines only. Where detailed information, such as in situ cone tests, strength tests on high quality samples, model tests, or pile driving performance is available, other values may be justified.

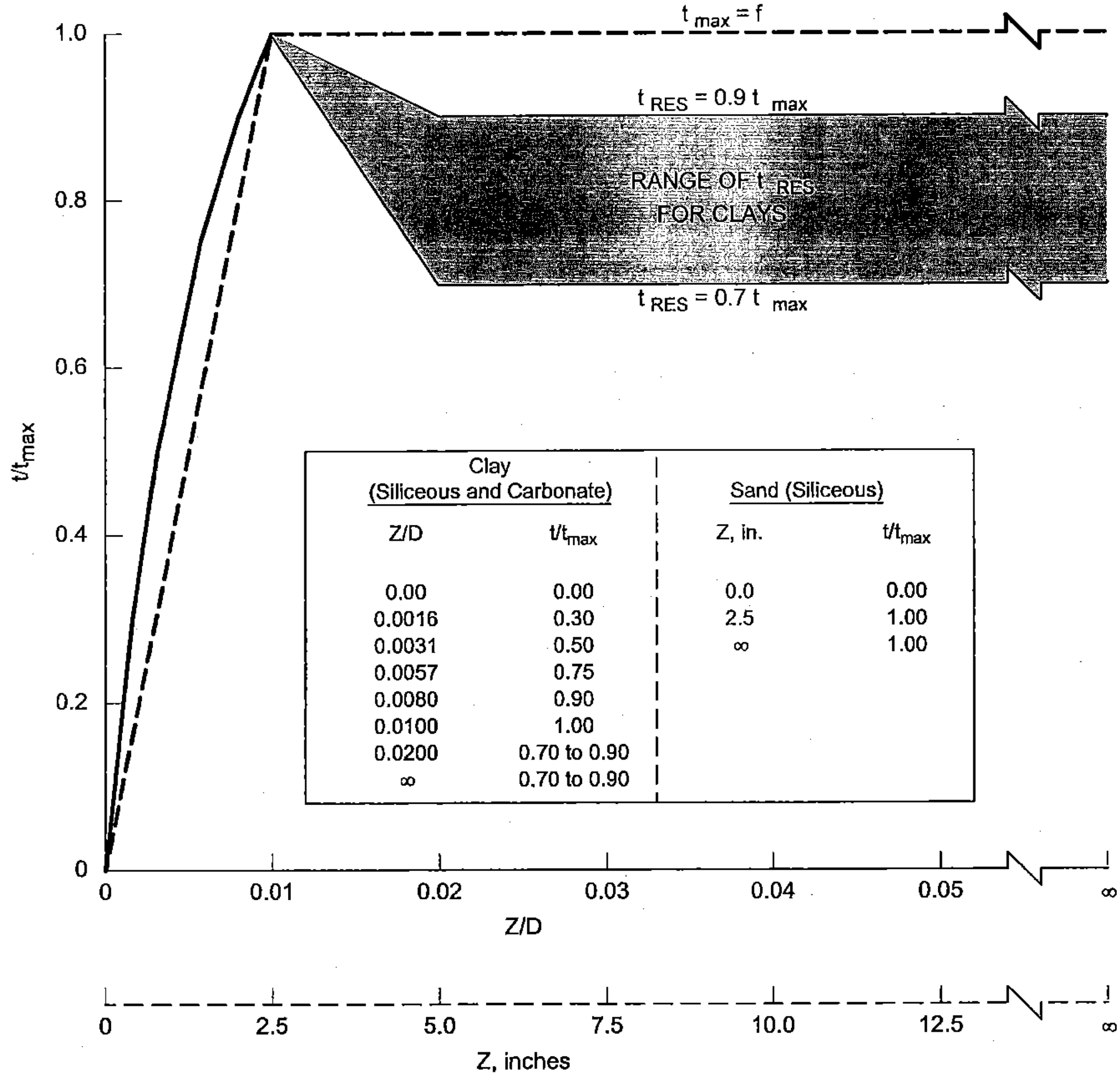
SUMMARY OF RECOMMENDED DESIGN PARAMETERS (API RP 2A, 2000) FOR COHESIONLESS SILICEOUS SOILS

Drawn By: Tomas Date: 9/14/08

Checked By: [illegible] Date: 8/1/00

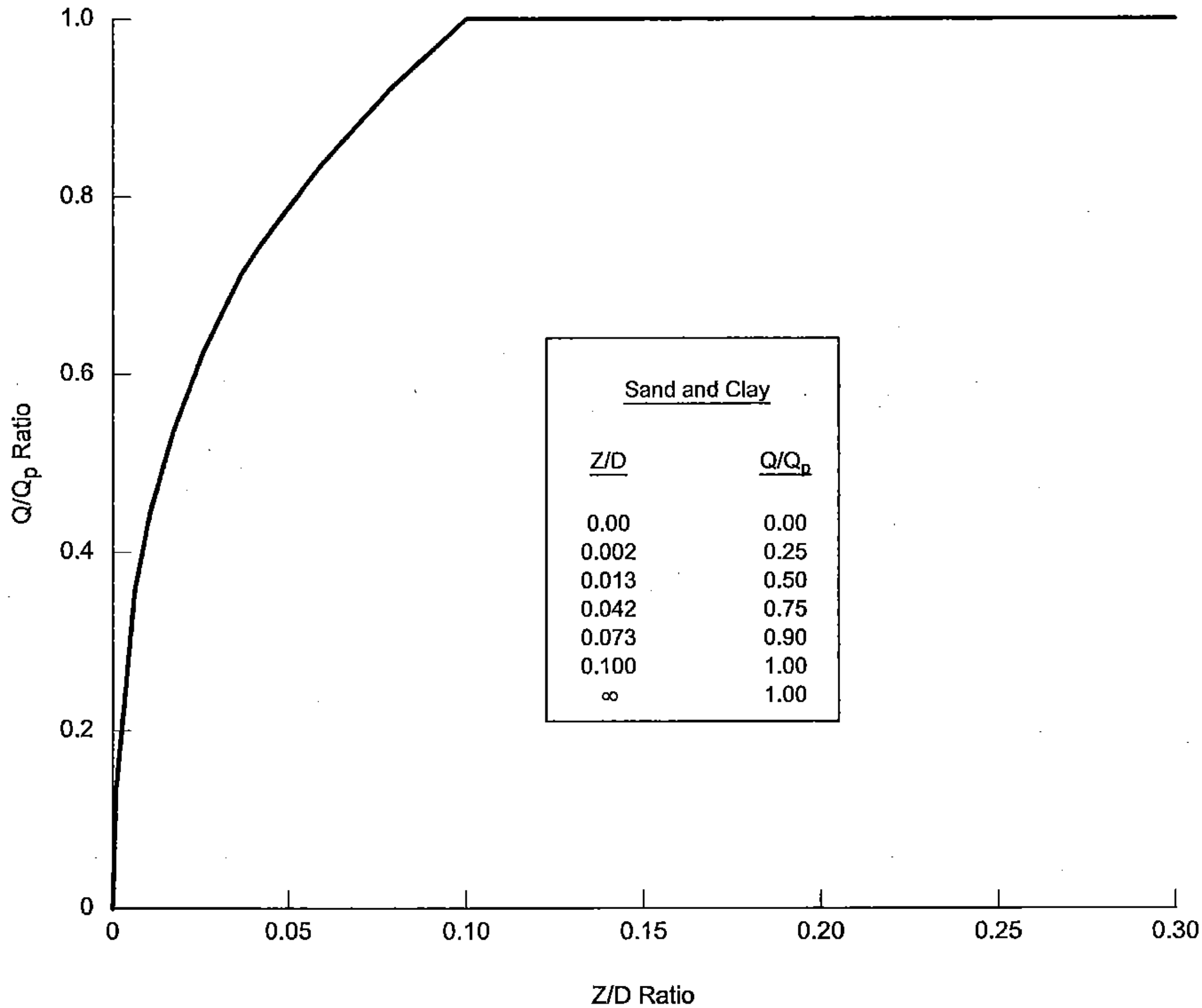
Approved By: [illegible] Date: 7/1/00

Drawn By: Date: 9/9/08
Checked By: Date:
Approved By: Date:



Clay (Siliceous and Carbonate)		Sand (Siliceous)	
Z/D	t/t_{max}	Z, in.	t/t_{max}
0.00	0.00	0.0	0.00
0.0016	0.30	2.5	1.00
0.0031	0.50	∞	1.00
0.0057	0.75		
0.0080	0.90		
0.0100	1.00		
0.0200	0.70 to 0.90		
∞	0.70 to 0.90		

TYPICAL SIDE LOAD TRANSFER (t-z) CURVES

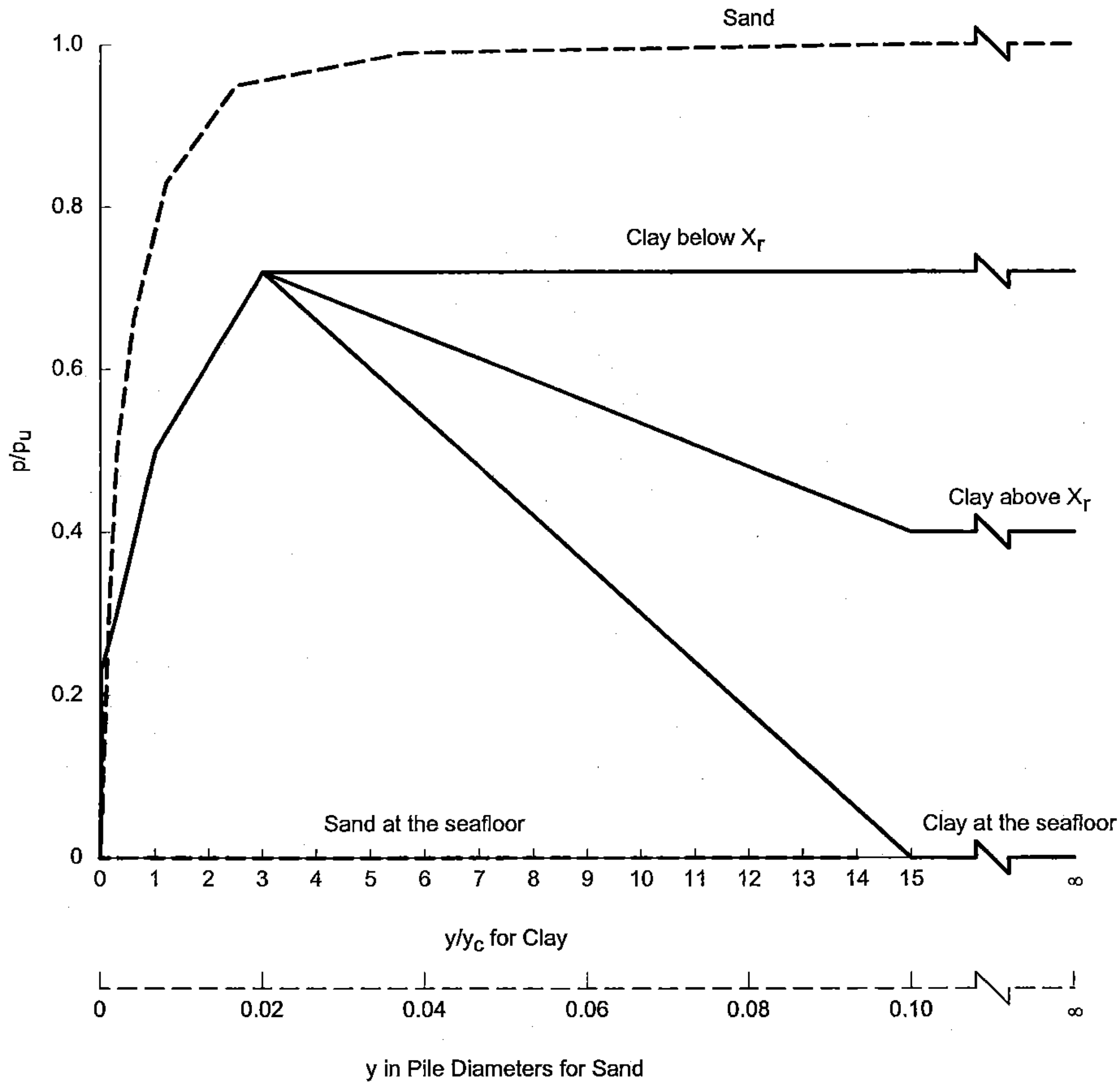


TYPICAL PILE TIP LOAD TRANSFER (Q-z) CURVES

Drawn By: Date: 9/9/08

Checked By: Date:

Approved By: Date:



TYPICAL LATERAL LOAD - PILE DEFLECTION (p-y) CURVES

Drawn By: Date: 9/4/08

Checked By: Date: 9/4/08

Approved By: Date: 9/5/08

ENTERPRISE FIELD SERVICES, LLC

SOIL BORINGS
TEXAS OFFSHORE PORT SYSTEM
SPM #1 and SPM #2
GALVESTON AREA BLOCK A56

1. INTRODUCTION:

Fugro Chance Inc. (CHANCE) was contracted by Fugro-McClelland Marine Geosciences, Inc. to position the *M/V "Seaprobe"* for soil borings in Galveston Area, Block A56, Offshore Texas.

2. REQUIREMENTS:

Positioning requirements were transmitted, via e-mail, to Mr. Joel Jones of ***CHANCE*** by Mr. Frank Ortiz of Fugro-McClelland Marine Geosciences, Inc. A "Survey Request" form dated June 25, 2008 e-mailed to Mr. Manuel Lopez of Enterprise Field Services, LLC, confirmed these requirements. A copy of this form was also e-mailed to Mr. Frank Ortiz.

Requirements were as follows:

A) Proposed Locations - SPM #1 and SPM #2

The Texas South Central Zone Coordinates are:

CORE 1(Leg #2 - West)	CORE 2 (Leg #6 - East)
Y = 251,897.66'	Y = 257,148.73'
X = 3,257,224.19	X = 3,266,735.50'

CORE 3 (PLET #1)	CORE 4 (PLET #2)
Y = 252,334.60'	Y = 256,177.08'
X = 3,258,627.75'	X = 3,265,632.42'

3. CHANCE PERSONNEL:

Party Chief - C. Evans

4. EQUIPMENT AND METHOD:

A) Primary Positioning System - **STARFIX® Satellite Positioning System**

Continuous dynamic positioning through the use of Navstar GPS with differential signals from multiple reference stations corrected for ionospheric and tropospheric effects transmitted via the **STARFIX®** equatorial geosynchronous satellite.

B) Secondary Positioning System - **Differential Global Positioning System (DGPS)**

DGPS utilizes the Navstar Satellite Constellation with data from selected reference sites transmitted via LF radio-link for enhanced accuracy through differential techniques.

C) **STARFIX.NAV®**

STARFIX.NAV® is an on board computer graphic system interfaced to the primary positioning system capable of displaying real time position of a vessel in relation to known hazards, fairways, proposed location, etc. **DRONE®** units when used on anchor handling vessels utilize Differential GPS transmitted to the master station via radio telemetry link to display in real time the position of that vessel.

D) Vessel orientation by Sperry SR 50 Mod 1 North Seeking Gyro or a S. G. Brown Meridian North Seeking Gyro

North seeking Gyro compass. Accuracy $\pm$ 2° after 4 hours initial spin up.

E) Scanning Sonar

Simrad MS1000 High Resolution Sonar

5. RESULTS:

Geographic positions are based on Clarke 1866 Spheroid, North American Datum 1927. Grid coordinates are based on Texas South Central Zone Lambert, NAD 27.

Field operations were conducted from June 30, 2008 to July 3, 2008 with the following results:

A) **STARFIX®** position derived by averaging readings over a one hour period at an update rate of 750 ms. per reading.

CORE 1 (Leg #2 - West)

Y = 251,890.17'
X = 3,257,199.22'

Latitude: 28° 28' 05.657" N
Longitude: 95° 05' 11.190" W

This location being 6,370.17' FSL and 3,163.41' FWL of Block A56, Galveston Area

CORE 2 (Leg #6 - East)

Y = 257,168.98'
X = 3,266,759.28'

Latitude: 28° 28' 54.716" N
Longitude: 95° 03' 22.142" W

This location being 4,191.02' FNL and 3,116.53' FEL of Block A56, Galveston Area

CORE 3 (PLET #1)

Y = 252,312.21'
X = 3,258,638.76'

Latitude: 28° 28' 09.356" N
Longitude: 95° 04' 54.911" W

This location being 6,792.21' FSL and 4,602.95' FWL of Block A56, Galveston Area

CORE 4 (PLET #2)

Y = 256,155.28'
X = 3,265,649.72'

Latitude: 28° 28' 45.055" N
Longitude: 95° 03' 34.951" W

This location being 5,204.72' FNL and 4,226.09' FEL of Block A56, Galveston Area

6. CONFIRMATION:

DGPS was used for confirmation.

The results were as follows:

CORE 1 (Leg #2 - West)

Y = 251,890'
X = 3,257,199'

CORE 2 (Leg #6 - East)

Y = 257,169'
X = 3,266,759'

CORE 3 (PLET #1)

Y = 252,312'
X = 3,258,639'

CORE 4 (PLET #2)

Y = 256,156'
X = 3,265,651'

7. HSE INCIDENTS:

No incidents.

8. CHRONOLOGY:

June 30, 2008

0001 Conducted job change in field; aboard "Seaprobe" at location GA. A56; preparing to set out anchors; start time for job
0124 PB #1 Anchor on bottom

0135 SB #2 Anchor on bottom
0146 PS #4 Anchor on bottom
0158 SS #3 Anchor on bottom; moving onto location
0210 Vessel on location; deploying Sonar to conduct site investigation
0219 Archiving Sonar data at proposed Core #2 location
0224 Starting final tie; drill crew performing soil boring
0324 Final tie at Core #2 complete
0338 Emailing Lafayette office for final tie confirmation
0343 Sonar on deck
0357 Received final tie confirmation from office
1030 Core #2 complete; starting anchor recovery
1035 PS #4 Anchor off bottom
1044 SS #3 Anchor off bottom
1052 PB #1 Anchor off bottom
1102 SB #2 Anchor off bottom
1104 PB #1 Anchor on bottom; SB #2 Anchor being repaired; anchor line twisted
1105 PB #1 Anchor off bottom
1106 PB #1 Anchor on bottom; anchor line twisted
1107 PB #1 Anchor off bottom
1130 PB #1 Anchor on bottom; repairing SB #2 Anchor
1413 PB #1 Anchor off bottom; en route to Galveston dock
2100 Arrived at Galveston dock; standing by for HSE meeting, crew change, and re-supply
2400 Continuing to standby at dock

<u>July 1, 2008</u>

0001 Standing by at Galveston dock
1200 HSE meetings occurring
2400 Continuing to standby at dock

<u>July 2, 2008</u>

0001 Standing by at Galveston dock
0230 Departed dock; en route to proposed Core #4 location
0925 Arrived on location; preparing to set out anchors
0930 PB #1 Anchor on bottom
0941 SB #2 Anchor on bottom
0954 SS #3 Anchor on bottom
1005 PS #4 Anchor on bottom
1028 Vessel on location; deploying Sonar to conduct site investigation at proposed Core #4 location
1033 Sonar on deck
1115 Starting final tie; drill crew performing soil boring
1215 Final tie at Core #4 complete
1300 Emailing Lafayette office for final tie confirmation
1400 Received final tie confirmation from office
1820 Core #4 complete
1830 Preparing to start anchor recovery
1834 SS #3 Anchor off bottom

1844 PS #4 Anchor off bottom
1854 SB #2 Anchor off bottom
1905 PB #1 Anchor off bottom; en route to proposed Core #3 location
1927 Setting out anchors at proposed Core #3 location; PB #1 Anchor on bottom
1939 SB #2 Anchor on bottom
1953 PS #4 Anchor on bottom
2005 SS #3 Anchor on bottom
2031 Vessel on location; deploying Sonar to conduct site investigation at core site
2034 Archiving Sonar data on proposed core site
2055 Sonar on deck; starting final tie; drill crew performing soil boring
2157 Final tie at Core #3 complete; emailing Lafayette office for final tie confirmation
2257 Received final tie confirmation from office
2400 Soil boring continues

July 3, 2008

0001 Standing by for completion of soil boring
0330 Core #3 complete; preparing to start anchor recovery
0337 SS #3 Anchor off bottom
0351 PS #4 Anchor off bottom
0401 SB #2 Anchor off bottom
0410 PB #1 Anchor off bottom; en route to proposed Core #1 location
0420 Preparing to set out anchors
0429 PB #1 Anchor on bottom
0443 SB #2 Anchor on bottom
0455 SS #3 Anchor on bottom
0509 PS #4 Anchor on bottom
0540 Vessel on location; deploying Sonar to conduct site investigation at core site
0555 Sonar on deck; starting final tie; drill crew performing soil boring
0655 Final tie at Core #1 complete; emailing Lafayette office for final tie confirmation
0720 Received final tie confirmation from office
0900 Called Lafayette office to discuss end of job details
1140 Core #1 complete; preparing to start anchor recovery
1146 SS #3 Anchor off bottom
1156 PS #4 Anchor off bottom
1203 SB #2 Anchor off bottom
1206 SB #2 Anchor on bottom; anchor line twisted
1207 SB #2 Anchor off bottom
1216 PB #1 Anchor on bottom
1230 En route to next location; end time for job

Sincerely,

FUGRO CHANCE INC.

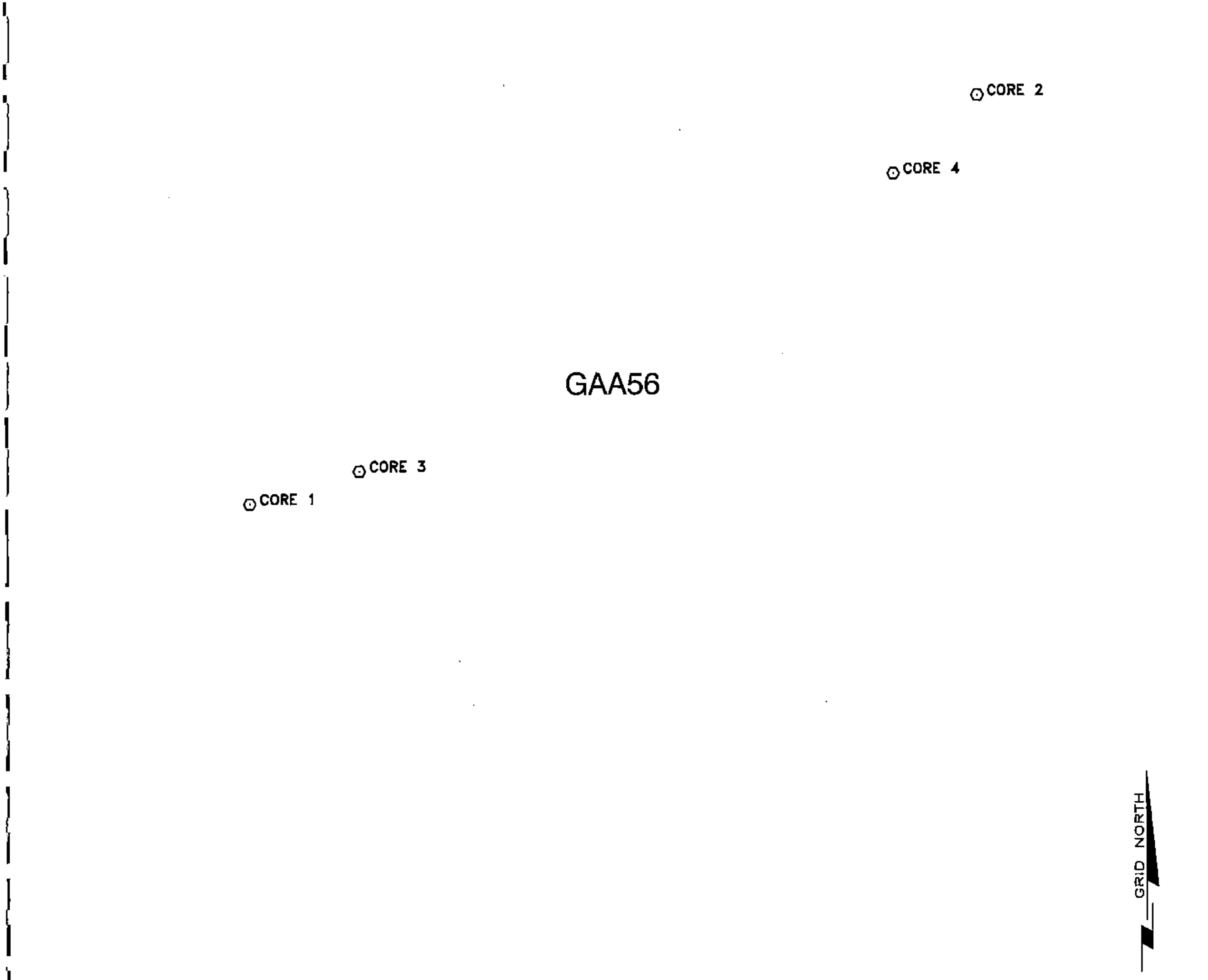
James P. O'Neal, P.L.S.
Vice-President, Marine Operations

JPO: mmg

Attachment

FINAL SOIL BORINGS

LOCATION	CALLNS	CALLEW	X COORDINATE	Y COORDINATE	LATITUDE	LONGITUDE
CORE 1	6,370.17' FSL	3,163.41' FWL	3,257,199.22'	251,890.17'	28° 28' 05.657"N	95° 05' 11.190"W
CORE 2	4,191.02' FNL	3,116.53' FEL	3,266,759.28'	257,168.98'	28° 28' 54.716"N	95° 03' 22.142"W
CORE 3	6,792.21' FSL	4,602.95' FWL	3,258,638.76'	252,312.21'	28° 28' 09.356"N	95° 04' 54.911"W
CORE 4	5,204.72' FNL	4,226.09' FEL	3,265,649.72'	256,155.28'	28° 28' 45.055"N	95° 03' 34.951"W



I HEREBY CERTIFY THAT THE ABOVE FINAL SOIL BORING POSITIONS ARE CORRECT.

STATE OF LOUISIANA
STEPHEN R. HENRY
REG. No. 4903
REGISTERED PROFESSIONAL LAND SURVEYOR

Stephen Henry
REG. PROFESSIONAL LAND SURVEYOR NO. 4903
STATE OF LOUISIANA 7-7-08

NOTES:

1) SURVEYED COORDINATES TRANSFORMED FROM NAD83 (GPS DATUM) TO NAD27 (CHART DATUM) USING NADCON VERSION 2.1.

Printed: 7/7/08

ENTERPRISE FIELD SERVICES, LLC

FINAL SOIL BORINGS
NO LEASE NUMBER (PROP. ANC & PLET)

BLOCK A56
GALVESTON AREA
GULF OF MEXICO

FUGRO CHANCE INC.

200 Dulles Dr. Lafayette, Louisiana 70506-3001 (337) 237-1300

GEODETIC DATUM: NAD27
PROJECTION: TEXAS SOUTH CENTRAL
GRID UNITS: US SURVEY FEET

SCALE IN FEET 0 2,000'

Job No.: 08-01933	Date: 7/7/08	Drwn: TCG	Chart: 1 Of: 1
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