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Exhibit BB.
CSX Almonaster Site
Preliminary Geotechnical Engineering Report

LOUISIANA ECONOMIC DEVELOPMENT CERTIFIED SITE

GREATER NEW ORLEANS INC
REGIONAL ECONOMIC DEVELOPMENT

EUSTIS ENGINEERING SERVICES, L.L.C.
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CSX Almonaster Site Preliminary Geotechnical Engineering Report

10 September 2012

InSite Real Estate Investment Properties, L.L.C.
Suite 300
1400 16th Street
Oak Brook, Illinois 60523

Attention Mr. Edwin J. Gebauer,P.E.

Gentlemen:

Geotechnical Services
Proposed Industrial Facility
Almonaster Boulevard
New Orleans, Louisiana
Eustis Engineering Project No. 21874

Transmitted are two copies (one bound and one unbound) of our engineering report covering geotechnical services for the subject project. A bound copy is being forwarded to Waldemar S. Nelson and Company, Incorporated, New Orleans, Louisiana, to the attention of Mr. Thomas W. Wells., P.E. Electronic copies are also being provided to you and Mr. Wells.

Thank you for asking us to perform these services.

Yours very truly,

EUSTIS ENGINEERING SERVICES, L.L.C.

John R. Eustis

JOHN R. EUSTIS, P.E.

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GEOTECHNICAL SERVICES

PROPOSED INDUSTRIAL FACILITY

ALMONASTER BOULEVARD

NEW ORLEANS, LOUISIANA

EUSTIS ENGINEERING PROJECT NO. 21874

FOR
INSITE REAL ESTATE INVESTMENT PROPERTIES, L.L.C.
OAK BROOK, ILLINOIS

By
Eustis Engineering Services, L.L.C.
Metairie, Louisiana

10 SEPTEMBER 2012

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PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA
EUSTIS ENGINEERING PROJECT NO. 21874

INTRODUCTION

1. This report contains the results of preliminary geotechnical analyses performed for the proposed industrial facility to be located on Almonaster Boulevard in New Orleans, Louisiana. The work was performed in accordance with Eustis Engineering Services, L.L.C.'s revised proposal dated 13 August 2012. Authorization to proceed with the analyses was given by Mr. Edwin Gebauer, P.E., of InSite Real Estate Investment Properties, L.L.C., on 13 August 2012.

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

3. The analyses and recommendations contained in this report are based in part on data obtained from the soil borings and cone penetrometer tests performed as part of a previous scope of work for the subject project. Results of the field exploration and laboratory tests were previously submitted by letter on 6 August 2012 entitled

"Professional Geotechnical Services, Proposed Industrial Facility, Almonaster Boulevard, New Orleans, Louisiana, Eustis Engineering Project No. 21874." The nature and extent of variations in subsoil conditions between and away from the boring/CPT locations may not become evident until construction or further geotechnical investigation. If variations then appear, it will be necessary to reevaluate the recommendations contained in this report.

4. Recommendations and conclusions contained in this report are to some degree subjective and should be used only for preliminary design and evaluation purposes.

SCOPE

5. The previous scope of work included the performance of soil test borings and CPTs to determine subsurface conditions and stratification, and to obtain samples of the various substrata. Soil mechanics laboratory tests, performed on samples obtained from the borings, were used to evaluate the physical properties of the subsoils.

6. The current scope of work includes engineering analyses to determine recommendations regarding site preparation and drainage, estimates of allowable pile load capacities, estimates of settlement, and effects of drag loads. Recommendations for surcharging the site with wick drains and deep soil mixing have also been provided.

DESCRIPTION OF EXISTING SITE CONDITIONS

Site Conditions

7. A majority of the site was heavily wooded at the time of drilling operations. A small portion of the site contains the Tchoupitoulas Beagle Club in a cleared area on the

northern side of the property. A large pile of wood debris is located near the southwestern corner of the site.

Geology

8. The site is characterized by Recent (Holocene) deposits overlying precompressed soils of the Pleistocene formation. The sequence of Recent deposits consists of highly compressible strata of swamp/marsh deposits to depths of 9 to 22 feet, followed by deltaic plain deposits ranging from 40 to 43 feet in depth, which are underlain by nearshore Gulf deposits to depths varying from 57 to 64 feet. The geologically identified Pleistocene formation continues to the final boring/CPT depths of 80 feet.

Stratigraphy

9. A graphical representation of the stratigraphy can be found on the boring/CPT logs previously provided with the letter dated 6 August 2012.

Ground Water

10. In order to determine the ground water conditions at the time of the field exploration, observations were made in each of the borings. Boring 1 was made without the addition of water to a depth of 15 feet and free water was initially encountered at 7 feet. Boring 2 was made without the addition of water to a depth of 11 feet and free water was initially encountered at 4 feet. Boring 3 was made without the addition of water to a depth of 15 feet and free water was initially encountered at 3 feet. The depth to ground water will vary with climatic conditions, drainage improvements, water levels in nearby waterways, and other factors. The depth to ground water should be determined by those persons responsible for construction immediately prior to beginning work.

FOUNDATION ANALYSIS

Furnished Information

11. The project site contains approximately 130 acres and multiple warehouses may be constructed. The warehouses would have live loads of 400 to 2,200 psf and may be constructed on 4 feet of fill to provide truck docks. A pile supported foundation is planned to support the warehouse structures. The structures could be as large as 450' x 2,220' in plan dimensions.

Foundation Recommendations

12. The buildings should be supported on a deep foundation comprising driven timber, timber composite, precast concrete, or augercast piles. All structural loads (floors, walls, and columns) should be supported on piles having approximately the same tip embedments in order to minimize differential settlement. We recommend a minimum pile embedment of 70 feet to tip into the underlying Pleistocene formation. Deeper pile lengths than those studied for this report would be recommended for the higher floor load intensities (1,300 psf+) and if fill exceeds 2 feet in height. Pile caps should be structurally integrated with grade beams. Any fill required to reach finished grade beneath the proposed structure should be placed as far in advance as possible of construction operations. However, we recommend that fill be limited to 2 feet for grading purposes and that the building be constructed with a void space or a Styrofoam form to provide the 4-ft slab height. Our recommendations assume no more than 4 feet of fill will be required to reach design grade. Should additional fill be required at the structure, Eustis Engineering should be contacted to evaluate potential settlement of this fill and its effects on the pile foundations.

13. Slab. The slab should be cast monolithically with grade beams which, in turn, are structurally connected to the pile caps to provide rigidity and minimize the potential

for differential settlement. The slab, including the reinforcing and anchor connection details, should be designed by a structural engineer.

14. Utilities. We recommend flexible type connections be specified for all piping and utilities beneath the pile supported slab. These connections should be designed to accommodate the settlement due to fill placement. Hangers should be provided and spaced such that the full weight of fill above the utilities can be supported without distress.

Ground Water Management

15. Drainage During Construction. The initial step to prepare the site for construction should be to establish adequate drainage to prevent ponding of water and ensure immediate runoff of all rainfall. This may be accomplished by grading the site to drain water away from all the paved areas and foundations. Sumps and pumps may be required to remove rainwater from low lying areas and excavations.

16. Permanent Drainage. The near surface soils supporting capacity will be reduced if they become inundated or saturated. Therefore, permanent drainage should be provided to collect rainfall away from the proposed foundations and pavement areas after completion of construction. Particular attention should be given to areas of pavements and surface founded structures. Permanent drainage should be installed to discharge rainwater away from these features. This would include collection of water from the roof of structures with gutters and downspouts, and directing this water into underground piping or channelization of the water away from the site. Saturation of base and subbase course materials for pavements will cause a reduction in the pavement service life.

Site Preparation

17. Clearing and Stripping. The existing ground surface beneath the proposed structures should be stripped of all vegetation, loose topsoil, organic matter, or other deleterious materials to the minimum depth necessary to reach firm undisturbed soil. While this depth may be substantial in localized areas (e.g., within existing ditches or stump holes), a depth of less than 8 to 12 inches should be adequate over the majority of the site. Actual requirements should be determined during construction. Excavated soils may be stockpiled for later use in landscaping, but these soils should not be used beneath the footprints of the proposed foundations or pavements.

18. Subgrade Preparation. After the stripping and clearing operations, the exposed surface should be proofrolled with a bulldozer or tracked vehicle. The vibratory system on the compactor, if present, should not be used during proofrolling. Alternative proofrolling techniques may be proposed, but these methods should be approved by Eustis Engineering prior to their use at the site. Weak zones, excavated areas, or depressions resulting from clearing and stripping operations should be thoroughly cleaned out to the surface of firm undisturbed soil and backfilled with a select fill material placed and compacted under controlled conditions.

19. Structural Fill. A select granular material, such as locally available pumped river sand, should be used as backfill and/or fill required to reach design grade. Sand fill should be non-plastic and free of roots, clay lumps, and other deleterious materials with no more than 10% by weight of material passing a U.S. Standard No. 200 mesh sieve (AASHTO A-3). The select fill should have an organic content of 5% or less.

20. Compaction. Backfill, form fill, or embankment fill beneath pavements should be placed in loose lifts of 8 to 10 inches and be compacted to at least 95% of its maximum dry density within ±2% of optimum water content in accordance with ASTM D 698. The compactive effort for backfill placed within excavations or depressions in excess of 12 inches below grade may be reduced to at least 90% of its maximum dry density in accordance with ASTM D 698. Structural fill used in association with pavement base or subbase should be placed in 6 to 8-in. thick lifts and compacted to at least 95% of its maximum dry density near optimum water content in accordance with ASTM D 1557.

21. Quality Control. Density tests should be performed on each lift of the compacted structural fill to determine if the contractor has achieved the recommended density. All clearing, proofrolling, and compaction operations should be performed only during periods of dry weather. The contractor should exercise caution during and after inclement weather to ensure subsoil support is not degraded by construction operations.

Fill Settlement

22. Analyses were made to determine the estimated settlement near the center of a 450' x 2,220' filled area. Based on a uniform dead load pressure intensity of 480 psf from 4 feet of fill (at 120 pcf), the results of the analyses indicate settlement near the center of the filled area may range from approximately 23 to 31 inches. For 2 feet of fill, our analyses indicate settlement of 12 to 16 inches near the center of the filled area. This estimate of settlement can be taken as the total settlement that would occur when the fill is placed. Time-rate analyses have been performed and based upon the presence of drainage layers and highly compressible materials approximately 85% to 90% of the ultimate settlement will occur within two years of placement. This estimate does not take into account settlement of the fill itself due to poor compaction. Settlement at the corners and midpoint of the sides is

estimated to be one-quarter and one-half of these values, respectively. We have assumed site grades will be raised no more than 4 feet. If our assumptions on fill area or height are not valid, Eustis Engineering should be contacted to reevaluate settlement potential. Fill placement will also affect piles as discussed subsequently.

23. Alternate Considerations. A preload/surcharge program should be considered if the settlement values are not considered tolerable. Other soil improvement techniques may be better suited to this site and the proposed loading. There are numerous proprietary improvement techniques that could be investigated as an alternate to traditional pile foundations. Various forms of soil mixing and/or soil-cement columns may provide suitable support of the anticipated loads.

Preload Program

24. Surcharge. The proposed 4 feet of fill will result in approximately 23 to 31 inches of settlement. An earth preload surcharge program would allow for consolidation of the underlying soils prior to the installation of piles and proposed construction.

25. We have assumed a preload surcharge will be limited to four to five months duration and must be designed to affect full consolidation of fill necessary to provide the required slab grade. To meet these requirements, we recommend the surcharge be 4.5 feet for a total fill height of 8.5 feet. In addition wick drains will be required to enhance the consolidation process of the Holocene soils. Wick drains should be spaced at 5-ft center to center spacings and configured in triangular arrays. Wick drains should extend to a depth of approximately 70 feet below the existing ground surface to the surface of the Pleistocene formation. Each wick drain should be connected to a horizontal strip drain discharging to the edge of the preload. The strip drain should be placed on a sand fill pad no more than 12 inches above existing grade.

26. Monitoring. The surcharge should be monitored by vibratory wire piezometers and vibratory wire settlement cells. Piezometers should be placed in nests of three, corresponding to depths of 10, 30, and 50 feet below the existing ground surface. The number and location of settlement cells and piezometer nests should be established once building dimensions are finalized.

27. Monitoring data should be retrieved on a biweekly basis. These data should be obtained, reduced, and evaluated by Eustis Engineering to judge the effectiveness of the surcharge.

Deep Foundations

28. Allowable Pile Load Capacities. Analyses have been made to determine the estimated allowable single pile load capacities in compression and tension for various sizes of treated ASTM D 25 quality timber piles, timber composite piles, precast concrete piles, or augercast piles for support of the proposed buildings. The allowable pile load capacities assume the top of the pile is located at existing grade and the piles are driven vertically. The results of these analyses are tabulated on Figure 1 for timber piles and shown graphically on Figures 2 and 3 for precast concrete and augercast piles. Installation requirements should be assessed during a test pile program. Selection of pile tip embedment should also consider potential settlement due to downdrag.

29. Factors of Safety. The allowable pile load capacities shown on Figures 1 through 3 contain an estimated factor of safety of 2 against failure of a single pile through the soil. To utilize the estimated capacities based on a factor of safety of 2, a pile load test should be performed.

30. Timber Piles. We recommend the treatment of timber piles meet the current American Wood Preservers Association's Standards as outlined in Section 1014 of

the Louisiana Standard Specifications for Roads and Bridges, 2000 edition (LSSRB) for both preservative and quality assurance. Treatment should also follow Section 812.06 where applicable. Furthermore, we recommend the timber piles meet the quality (clean peeled, straightness, etc.) requirements outlined in ASTM D 25 and size requirements outlined in Table X1.5 of ASTM D 25 for specified tip circumferences. The pile dimensions assumed in our analyses are tabulated on Figure 1.

31. Timber Composite Piles. Composite piles should consist of an untreated ASTM D 25 quality timber pile having a minimum 7-in. tip and 13-in. butt lower section and a 12-in. diameter concrete filled metal can upper section. The upper section should extend a minimum distance of 10 feet below the current existing ground surface to protect the untreated timber section. The upper section should be of sufficient thickness to withstand handling stresses and soil and water pressures. We recommend a maximum upper section length of 15 feet. A mandrel impacting the timber pile butt should be used to install the upper section. Prior to placing concrete, the upper section should be inspected to ensure it is free of water. Concrete placed in the upper section should have a minimum compressive strength of 2,500 psi or as dictated by structural requirements. Timber composite piles should not be used to resist lateral or tensile loads. Alternate timber composite pile configurations should be forwarded to Eustis Engineering to verify our capacity estimates. Installation requirements, such as pile refusal should be considered when selecting composite section lengths.

32. Square, Prestressed, Precast Concrete Piles. We recommend the precast concrete piles meet the requirements outlined in Section 805.14 of the LSSRB. Precast concrete piles should be designed to have a strand prestress that is structurally sufficient to facilitate handling and driving the piles without damage. Our analyses are based on a solid, square pile. Alternate precast pile configurations (pipe, triangle, etc.) should be further evaluated.

33. Augercast Piles. The allowable load capacities for the augercast piles do not include the weight of the pile. The augercast capacities also assume the actual grout volume placed is at least 30% of the theoretical volume of the pile. When computing the resultant net compressive capacity of the piles, a net load of 45 pcf may be used for the weight of the concrete (weight of the concrete minus the weight of soil removed). The full buoyant weight of the concrete of 87 pcf may be used to determine the tensile capacity provided the pile is designed to transmit these loads.

34. Structural Capacity. Analyses for pile capacities are based only on a soil-pile relationship. Therefore, the structural capacity of the piles and their connections to transmit these loads should be determined by a structural engineer. In particular, composite pile connections should be designed to handle driving stresses as well as permanent loads.

35. Pile Group Capacity and Spacing. The estimated allowable single pile load capacities described in this report will derive the majority of their compressive capacity through skin friction. Therefore, consideration of group effects is required. In this regard, the supporting value of the piles driven in groups should be investigated on the basis of the group perimeter shear by the formula shown on Figure 4. The minimum spacing between individual piles should be at least 3 feet or 5% of the embedment, whichever is greater. This relationship is also shown on Figure 4. The minimum spacing between rows or groups of piles should also meet the requirements outlined below regarding settlement.

36. Estimated Settlement due to Structural Loads. We recommend grade beams be rigidly connected to pile caps to minimize the potential for differential settlements. We understand the warehouse building may be designed for floor loads of 400, 1,500, 2,000, and 2,200 psf. Our estimates of settlement for piles with a minimum tip embedment of 70 feet below the existing ground surface are shown in Table 1.

TABLE 1: ESTIMATED PILE SETTLEMENT DUE TO STRUCTURAL LOADS

UNIFORM LOADING OVER 450' x 2,220' SLAB IN PSF	ESTIMATED SETTLEMENT OF PILES DUE TO STRUCTURAL LOADS IN INCHES
400	¼ to ½
1,500	3½ to 5
2,000	5½ to 7½
2,200	6 to 8½

These estimates assume a rigid structure. In reality, some degree of flexibility is inherent in the structure because of its large footprint and differential settlement due to uniform loads should be anticipated. Differential settlement potential for a flexible structure may be estimated by assuming 50% and 25% of the tabulated settlements occur at the edges and corners of the building. These would be differential with respect to the interior of the structure, 50 to 60 feet from its perimeter. Considering the partial rigidity of the structure and for preliminary purposes, we estimate differential settlement due to structural loads to be 15% to 20% of the tabulated values.

37. ***These settlement estimates are applicable to fill limited to 2 feet or less assuming the project incorporates a void space or Styrofoam form blocks beneath the 4-ft elevated slab or if a preload surcharge is used in association with 4 feet of fill.*** These estimates do not include elastic deformation of the piles which should be added to the settlement estimates. Elastic deformation of the piles may be estimated as 67% to 75% of the static column strain of a pile acting as a column. These estimates of settlement also do not include settlement due to filling as addressed below.

38. Our estimates of settlement are based on the assumption piles will be driven in small groups or widely spaced rows. We have assumed the center to center spacing between groups will be no closer than twice the largest group dimension

and the center to center spacing between rows of single piles will range from 4.5 to 11 feet for the furnished structural loads. In the event any of our assumptions are not met, Eustis Engineering should be contacted to evaluate the potential settlement of the pile foundations.

39. Effects of Fill Placement on Piles. Assuming 4 feet of fill beneath the slab without a preload, consolidation of the underlying subsoils will induce negative skin friction (drag loads) on the piles as the soil settles along the pile. These drag loads will result in additional pile settlement. Assuming 4 feet of fill is placed beneath the filled area (within a 450' x 2,220' area), our estimates of settlement of pile foundations are given in Table 2.

TABLE 2: ESTIMATED PILE SETTLEMENT DUE TO 4 FEET OF FILL (NO PRELOAD)

PILE TIP EMBEDMENT BELOW EXISTING GROUND SURFACE IN FEET	ESTIMATED ULTIMATE SETTLEMENT OF PILES DUE TO PLACEMENT OF 4 FEET OF FILL IN INCHES
60	3½ to 5
70	3 to 4
80	2½ to 3½

(1)Based on an estimate of the time-rate of settlement.

These estimates of settlement are only due to fill placement and should be added to settlements due to structural loads. Approximately 95% to 98% of the ultimate settlement will occur within two years of fill placement due to the presence of drainage layers and highly compressible soils. These estimates represent settlements at the center of the 450' x 2,220' filled area. Differential settlements between the interior of the filled area and edges may be taken as approximately one-half of the values tabulated above. This differential settlement will not be linear between the center of the filled area and the edges, and may occur in relatively short horizontal dimensions (50 to 60 feet) from the edge to the filled area's interior. Eustis Engineering should be further consulted if the selected pile lengths result in unacceptable differential settlements. ***Should more than 4 feet of fill be required to reach design grade at the structure or if a larger fill area is planned, Eustis Engineering should be contacted to further evaluate settlement and its effects on pile foundations.***

40. Differential Settlement. Your design should recognize the potential for differential settlement between pile supported features and grade supported features. A joint considering these movements should be provided between any grade supported and pile supported features to accommodate potential differential settlement. In addition, the structure should be designed as rigidly as possible to minimize the potential for differential settlements. Utilities beneath the pile supported structure should be supported by hangers. Flexible connections should be used at the building line.

Areal Subsidence

41. The estimated settlements previously provided in this report do not include areal subsidence. Areal subsidence is an ongoing process that is the result of ground water lowering due to area drainage, filling, biodegradation of near surface organic soils, or a combination of these factors. The amount of areal subsidence cannot be estimated from information developed for this report. However, it can result in differential movement between the grade supported features such as pavements and the pile supported structure. These settlements can be significant over short distances.

Installation of Driven Piles

42. Quality Control. All pile driving operations should be supervised by experienced personnel to ensure proper procedures are followed and accurate records are kept during all pile driving operations. The driving records should include the date, type

of pile, pile tip and butt diameters or side dimension, overall pile length, depth and diameter of predrill, hammer model, driving energy, and number of blows per foot of penetration for the full embedment of the pile. An accurate driving record is especially important to verify piles are installed to the required tip embedment and to give an indication of any unusual driving characteristics which may include pile breakage. We recommend Eustis Engineering be retained to observe, record, and evaluate all pile driving operations with respect to the recommendations presented in this report.

43. Hammers. Treated or untreated ASTM D 25 quality timber piles, having tip diameters of 7 inches or greater and butt diameters of 12 inches or more, may be driven with a single acting air hammer with a manufacturer's rated energy of 15,000 ft-lbs per blow. For these piles, the ram weight should not exceed 5,000 pounds and the maximum stroke should be limited to 3 feet. Using this driving energy, timber piles should be driven no harder than 25 blows per foot (refusal) to minimize structural damage to the piles.

44. Precast concrete piles should be driven with a single acting air hammer delivering 19,500 ft-lbs of energy. The weight of the hammer should not exceed one-third to one-half the weight of the pile and the stroke should not exceed 3 feet. Using this driving energy, concrete piles should be driven no harder than 40 blows per foot. Based on the soil conditions encountered at the borings/CPTs, we do not anticipate high driving resistances. Tensile stresses could occur in the concrete piles during low driving resistances (less than eight blows per foot) and cause damage to the piles. Driving stresses should be evaluated during the test pile program.

45. Predrilling. Predrilling may be required to a depth of 52 feet to assist the driving of timber piles through loose to very dense sand layers and to minimize the potential for heave of timber and precast concrete piles. Predrilling also has the potential of minimizing vibrations resulting from pile driving operations. The predrill bit should

be no larger than the tip diameter of timber piles or 75% of the side dimension for precast concrete piles. Predrilling should be made by wet rotary methods using a fishtail bit. Predrilling should extend no deeper than 10 feet above the final tip elevation of the piles. Actual requirements should be determined during a test pile program.

Installation of Augercast Piles

46. Quality Control. Successful installation of augercast piles will be a direct function of the quality control used at the site. As a minimum, the quality control should include the checking of the mix, fluidity, temperature, use of additives, and sampling for subsequent compressive strength testing. The quality of the grout mix used for the piles should be monitored using ASTM C 109 procedures. Fluidity may be checked by using modified U.S. Army Corps of Engineers' Specification CRD C 611 and ASTM C 939. Flow rates should be in the range of 10 to 25 seconds using a flow cone modified to have a 0.75-in. opening.

47. Prior to installation of any grout, the grout pump should be calibrated in units of volume per stroke. In addition, grout pressure and volume should be recorded for every 5 feet of installation. Finally, the theoretical volume of grout should be compared to the total grout volume used for each pile to determine the percent over theoretical volume. Eustis Engineering should be contacted if the volume of grout placed for any segment is less than 130% of the theoretical volume.

48. Equipment and Techniques. Augercast piles should be made by rotating a continuous flight hollow shaft auger into the ground to the necessary depth in order to develop the desired load capacity. The auger flighting should be continuous without gaps or breaks and should have a uniform diameter throughout its length. For auger lengths of 40 feet or more, we recommend a middle guide be used. Adjacent piles should not be installed until the pile has set for a sufficient period, as

determined by the structural engineer, to withstand earth pressures exerted by the installation process. Normally, a minimum 24-hour set time is recommended.

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

50. The grout should be designed to provide an adequate strength to support the anticipated design load. The grout head should be 5 feet higher than the fluid levels in the bore hole and/or the bottom of the auger at all times. The grout should be placed continuously from bottom to top while the auger rotates during withdrawal.

Test Piles and Load Tests

51. Eustis Engineering considers a test pile program and load test as an extension of our geotechnical investigation. Therefore, Eustis Engineering should be retained to perform these services. We recommend a test pile program be planned for the site. The number of test piles will depend on the number of piles planned for the project, selected embedments, and proposed loads.

52. The test piles should be the same type and embedment anticipated for the job piles and installed with the same equipment and techniques proposed for the job piles. The test piles can be used to evaluate installation methods. Several test piles should be selected for testing. The test piles should be allowed to set for 14 days subsequent to installation of the reaction system. The test piles should then be load tested to failure in accordance with ASTM D 1143 and the New Orleans Building Code. Results should be evaluated by Eustis Engineering to verify the estimated pile load capacities presented in this report.

53. Dynamic Pile Testing. For the initial installation of concrete piles, you should consider monitoring and evaluating by DPT using a Pile Driving Analyzer®. A PDA can monitor driving stresses during installation and evaluate pile integrity during or after installation. A PDA can also monitor energy transferred to the pile by the hammer to evaluate pile installation efficiency. In order to evaluate pile capacity, a "restrike" DPT should be performed a minimum of 28 days after its initial installation. Shorter restrike set times may be considered, but a test may not indicate the full ultimate capacity. In any case, a restrike set time less than seven days to evaluate capacity of friction piles should not be considered for the evaluation of capacity. Data from this restrike should be further evaluated by CAPWAP® analyses.

Post Construction Quality Control

54. In order to ensure successful installation of augercast piles, we recommend post construction inspection be performed on test piles. Post construction inspection can be performed by low strain integrity methods including Pile Integrity Testing (PIT), cross-hole sonic logging (CSL), or single-hole sonic logging (SSL). Based on the diameters and lengths of augercast piles proposed for this project, we recommend SSL be performed. If these tests or installation records reveal structural defects or anomalies, additional quality control tests, consisting of further SSL or PIT, may be recommended. SSL testing should also conform with Section 814.19 of the LSSRB. The SSL test method requires the installation of an access hole during pile installation. Therefore, the project schedule should allow for load test evaluation prior to job pile installations should this testing be warranted.

Vibrations

55. Pile installation, as well as other construction activities, has the potential to generate vibrations that may affect nearby structures, pavements, and underground utilities.

Eustis Engineering recommends vibrations be monitored during the test pile program and subsequent construction activities of concern. This monitoring should evaluate peak particle velocities during pile installation at critical structures with a seismograph, as well as other construction activities generating vibrations (hauling fill, moving heavy equipment, etc.). The record of peak particle velocities will provide information in assessing potential damage and the need for changes in construction operations.

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

RECOMMENDATIONS FOR SITE SPECIFIC GEOTECHNICAL EXPLORATION

57. General. The recommendations contained in this report are preliminary and are based on a limited geotechnical exploration performed at the site. In order to implement these recommendations into your design, the subsurface conditions and stratifications must be substantiated by an additional geotechnical exploration. We understand the proposed facility will generally include large warehouse buildings. Pavements and a rail spur could be the other project features. The proposed project features will likely require both shallow and deep foundations for support of the structures depending on the anticipated loadings of the individual structures.

Based on the furnished information, we recommend a combination of soil borings and CPTs be made for the proposed industrial facility should this site be selected for development. Depending on fill heights and floor loading intensities, borings deeper than those made for this preliminary investigation may be required. Eustis Engineering should be contacted to determine the number and depth of soil borings and CPTs once a preliminary site layout is determined.

GEOTECHNICAL SERVICES DURING CONSTRUCTION

58. In order to provide continuity among the preliminary and final investigation, design, and construction phases, Eustis Engineering should be retained to review plans and specifications developed for the project and all contractor submittals related to geotechnical issues and foundations. Eustis Engineering can also provide additional geotechnical services which may include consultation during design and construction. We can also provide steel, concrete, and asphalt inspection services, and compaction and inplace density determinations on fill materials. We can perform appropriate laboratory tests to determine the gradation and quality of material proposed as structural fill or backfill. Eustis Engineering can also log the installation of test piles and job piles, provide DPT services, perform and evaluate pile load tests, and monitor vibrations.

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

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA
EUSTIS ENGINEERING PROJECT NO. 21874

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES
TREATED ASTM D 25 QUALITY TIMBER PILES
AND TIMBER COMPOSITE PILES

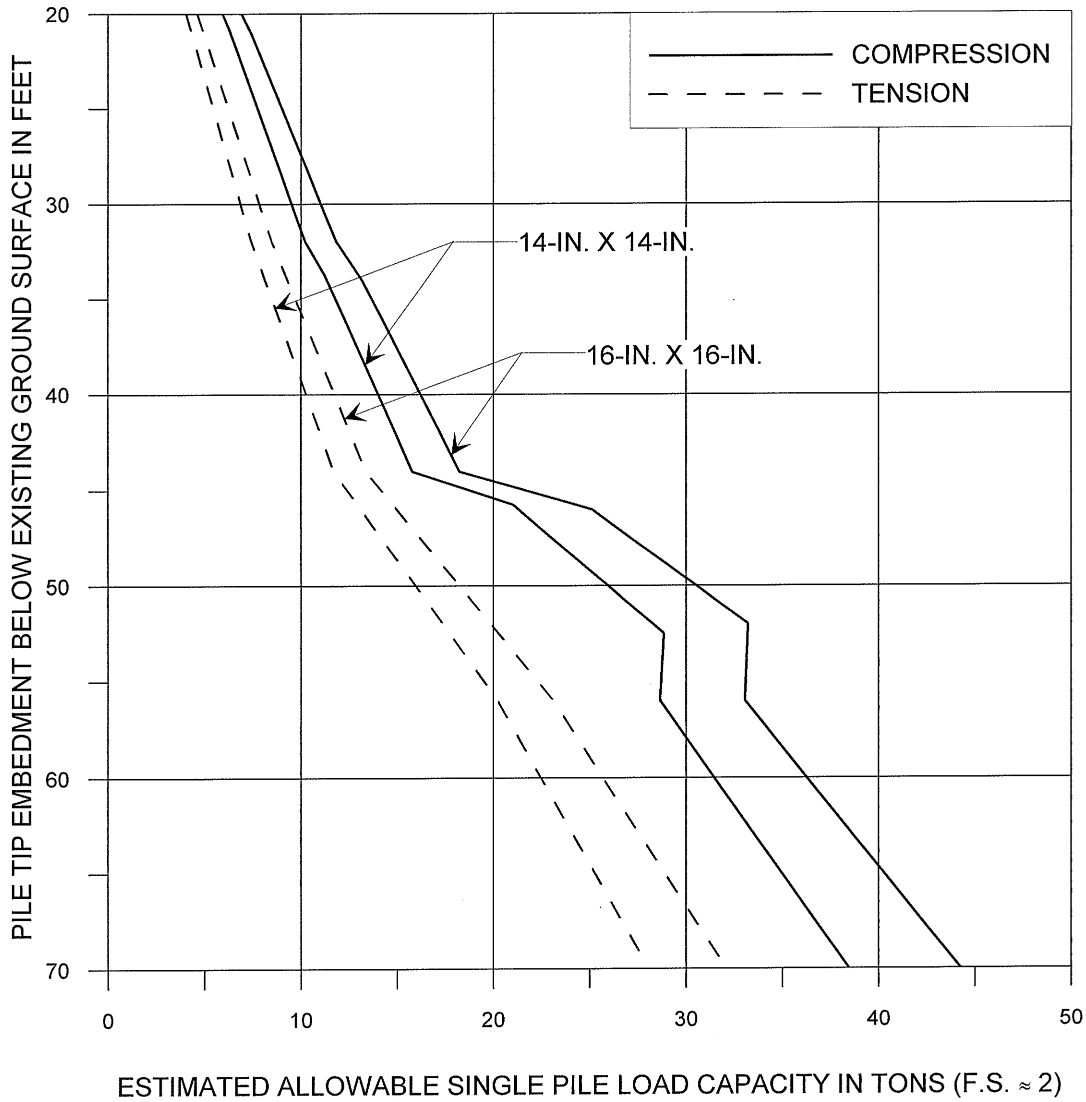
PILE DIAMETER	ESTIMATED PILE TIP EMBEDMENT BELOW EXISTING GROUND SURFACE	ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES IN TONS[1] FACTOR OF SAFETY ≈ 2[2]	
		COMPRESSION	TENSION
7-In. Tip, 12-In. Butt	40 45 50 55	7 10 13½ 16½	5 6½ 8½ 11
7-In. Tip, 13-In. Butt or Composite	60 65 70	19 21 23	13[3] 14½[3] 16[3]

[1] These estimated capacities do not include limitations on structural capacity or imposed by some building codes.

[2] Use of a factor of safety of 2 assumes a static pile load test is performed.

[3] Timber composite piles should not be used to resist tensile or lateral loads. Timber composite piles should meet the size and installation requirements given in the report.

SQUARE PRECAST CONCRETE PILES



NOTE:

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

(2) USE OF A FACTOR OF SAFETY OF 2 ASSUMES CAPACITIES ARE VERIFIED BY STATIC LOAD TESTS.

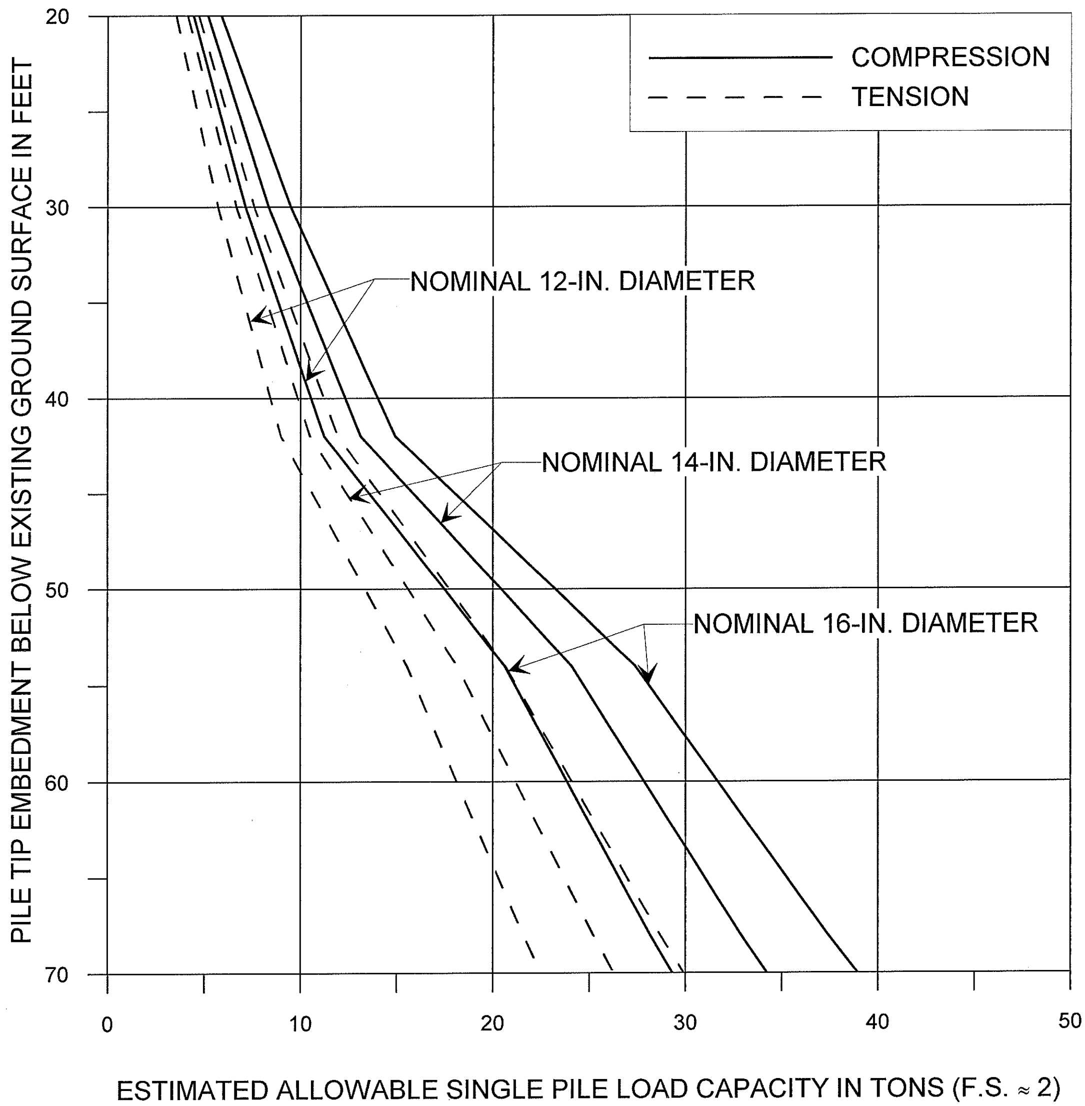
EUSTIS ENGINEERING SERVICES, L.L.C.
GEOTECHNICAL ENGINEERS
3011 28TH STREET METAIRIE, LOUISIANA

ESTIMATED ALLOWABLE
SINGLE PILE LOAD CAPACITIES
SQUARE PRECAST CONCRETE PILES

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS (ORLEANS PARISH), LOUISIANA

DRAWN BY: J.M.G.	22 AUG 2012	FILE: SPC.GRF
CHECKED BY: W.W.G.	JOB : 21874	FIGURE 2

AUGERCAST PILES



NOTE:

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

(2) USE OF A FACTOR OF SAFETY OF 2 ASSUMES CAPACITIES ARE VERIFIED BY STATIC LOAD TESTS.

(3) CAPACITIES ASSUME ACTUAL VOLUME PLACED IS AT LEAST 130% OF THE THEORETICAL VOLUME COMPUTED FOR THE NOMINAL DIAMETER SHOWN.

EUSTIS ENGINEERING SERVICES, L.L.C.
GEOTECHNICAL ENGINEERS
3011 28TH STREET METAIRIE, LOUISIANA

ESTIMATED ALLOWABLE
SINGLE PILE LOAD CAPACITIES
AUGERCAST PILES

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS (ORLEANS PARISH), LOUISIANA

DRAWN BY: J.M.G.	22 AUG 2012	FILE: AC.GRF
CHECKED BY: W.W.G.	JOB : 21874	FIGURE 3

CAPACITY OF PILE GROUPS

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

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

In Which:

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

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

SPACING WITHIN PILE GROUPS

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

In Which:

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

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

EUSTIS ENGINEERING SERVICES, L.L.C.
3011 28TH STREET
METAIRIE, LOUISIANA 70002-6019
PN 504-834-0157 I FN 504-834-0354
EMAIL: INFO@EUSTISENG.COM I SITE: WWW.EUSTISENG.COM

6 August 2012

InSite Real Estate, LLC
Suite 300
1400 16th Street
Oak Brook, Illinois 60523-8854

Attention Mr. Eric Pedersen
PN 1-630-617-9135
Email epedersen@insiterealestate.com

Gentlemen:

Professional Geotechnical Services
Proposed Industrial Facility
Almonaster Boulevard
New Orleans, Louisiana
Eustis Engineering Project No. 21874

In accordance with our proposal dated 14 June 2012, Eustis Engineering Services, L.L.C., performed professional geotechnical exploration and laboratory services for the subject project. InSite Real Estate, LLC, accepted this proposal for a preliminary geotechnical data report by an "Agreement between InSite and Consultant" dated 17 July 2012.

This exploration has been performed in accordance with generally accepted geotechnical engineering practice for the exclusive use of InSite for specific application to the subject site. Should these data be used by anyone other than InSite, they should contact Eustis Engineering for interpretation of data and to secure other information pertinent to this project.

The results of the soil borings, laboratory tests, and cone penetrometer tests contained in Appendices I and II of this letter may be included in the plans and specifications.

The individual boring logs and CPT records are considered representative of subsurface conditions at their respective locations on the dates completed. No warranty is given that the boring logs and CPT records are representative of subsurface conditions at other locations or times. The nature and extent of variations in subsurface conditions between and away from the boring and CPT locations may not become evident until construction or an additional geotechnical investigation.

SCOPE

The investigation included the drilling of soil test borings and performance of CPTs to evaluate subsurface conditions and stratification. Samples were recovered from the various substrata. Soil mechanics laboratory tests, performed on samples obtained from the undisturbed borings, were used to evaluate the physical properties of the various substrata. Engineering analyses were not included in the current scope of work.

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

FIELD EXPLORATION

General

The scope of the field services outlined in the referenced proposal includes the performance of three soil borings and five CPTs, all extending to a depth of 80 feet below the existing ground surface. The soil borings are designated as B-1 through B-3 and the CPTs as CPT-1 through CPT-5. The approximate locations of the borings and CPTs performed for this investigation are included on Enclosure 1.

Boring and CPT locations were generally selected using a drawing furnished by InSite. Several of the borings and CPTs were relocated in the field due to heavily wooded site conditions. The GPS coordinates of the actual locations, as determined by a handheld unit, are provided on the boring logs in Appendix I and CPT records in Appendix II. The borings were drilled 18 through 23 July 2012 and the CPTs were performed 23 through 25 July 2012.

Soil Borings

The undisturbed borings were drilled using a rotary type drill rig mounted on a rubber-tired all-terrain carrier. Upon completion of drilling the borings, the holes were grouted in accordance with current regulatory requirements.

Cohesive or semi-cohesive subsoils were sampled at close intervals or changes in strata using a 3-in. diameter thinwall Shelby tube sampling barrel. The undisturbed samples were immediately extruded from the sampling barrel in the field. Pocket penetrometer tests were performed on the soil samples to provide a general indication of their shear strength or consistency. The results of these tests are shown on the boring logs in Appendix I under the column heading "PP." All samples were inspected and visually classified by Eustis Engineering's soil technician. Representative portions of the samples were placed in moisture proof containers and returned to Eustis Engineering's laboratory for additional testing.

Cohesionless and semi-cohesive soils were obtained during the performance of in situ Standard Penetration Tests. This test consists of driving a 2-in. diameter splitspoon sampler 1 foot into the soil after first seating the sampler 6 inches. A 140-lb weight dropped 30 inches is used to advance the sampler. The number of blows required to drive the sampler through the final 1-ft increment is indicative of the relative density or approximate consistency of the subsoils tested. The results of the Standard Penetration Tests are shown on the boring logs in Appendix I under the column heading "SPT." Representative samples were placed in moisture proof containers for preservation of their natural moisture content.

Cone Penetrometer Tests

The static CPTs were performed using an electronic piezocone penetrometer having a 5-ton capacity on track mounted equipment. The CPTs were performed using a 10-cm^2 cross-sectional area cone with a 60° apex angled tip and 150-cm^2 sleeve area. The penetrometer was hydraulically advanced into the ground at the rate of 2 cm/sec. During cone testing, CPT parameters (tip resistance, friction resistance, and pore pressure) were recorded at 5-cm depth intervals. The results of the CPTs were plotted graphically with depth. These plots are provided in Appendix II. The plots provide tip resistance (q_t), sleeve friction (f_s), and pore pressure (u_2). The estimated undrained shear strength (S_u), equivalent standard penetration resistance (N_{60}), and interpreted soil behavior are also shown on the CPT records. These values are interpreted from correlations developed by Lunne, Robertson and Powell (1997 and 1986) and our engineering experience in southern Louisiana. Testing was performed in accordance with methods and procedures outlined in ASTM D 5778-07. Upon completion of the CPTs, the holes were grouted in accordance with current regulatory requirements.

LABORATORY TESTS

Soil mechanics laboratory tests, consisting of natural water content, unit weight, and either unconfined compression shear (UC) or unconsolidated undrained triaxial compression shear (OB), were performed on undisturbed samples obtained from the borings. Atterberg limits determinations were also performed on representative samples to further classify the subsoils. The results of these laboratory tests are tabulated on the boring logs in Appendix I.

DESCRIPTION OF EXISTING SITE CONDITIONS

Site Conditions

A majority of the site was heavily wooded at the time of drilling operations. A small portion of the site contains the Tchoupitoulas Beagle Club in a cleared area on the northern side of the property. A large pile of wood debris is located near the southwestern corner of the site.

Stratigraphy

Our interpretation of the stratigraphy at the boring and CPT locations is shown on the logs in Appendices I and II.

Ground Water

In order to determine the ground water conditions at the time of the field investigation, observations were made in each of the borings. Boring 1 was made without the addition of water to a depth of 15 feet and free water was initially encountered at 7 feet. Boring 2 was made without the addition of water to a depth of 11 feet and free water was initially encountered at 4 feet. Boring 3 was made without the addition of water to a depth of 15 feet and free water was initially encountered at 3 feet. The depth to ground water will vary with climatic conditions, drainage improvements, water levels in nearby waterways, and other factors. The depth to ground water should be determined by those persons responsible for construction immediately prior to beginning work.

InSite Real Estate, LLC
6 August 2012

Thank you for asking us to perform these services. If you have any questions or require further information, please contact us.

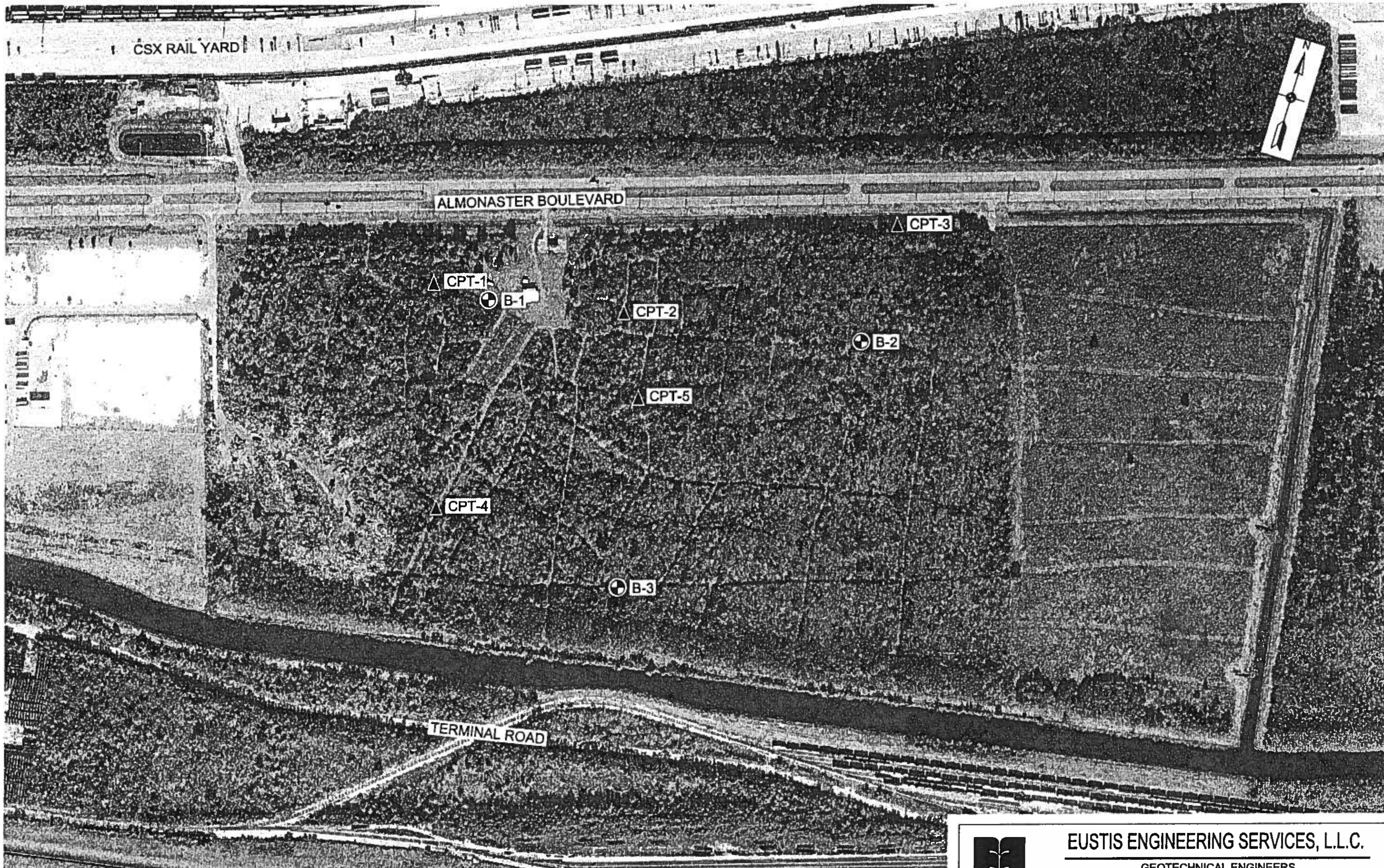
Yours very truly,

EUSTIS ENGINEERING SERVICES, L.L.C.

JOHN R. EUSTIS, P.E.

J. M. Gisclair:aln/jdd

Enclosure 1
Appendices I and II



⊕ DENOTES LOCATION OF UNDISTURBED SOIL BORINGS DRILLED:
18 THROUGH 23 JULY 2012

▲ DENOTES LOCATION OF CONE PENETROMETER TESTS PERFORMED:
23 THROUGH 25 JULY 2012

NOT TO SCALE

EUSTIS ENGINEERING SERVICES, L.L.C.
GEOTECHNICAL ENGINEERS
3011 28TH STREET METAIRIE, LOUISIANA

BORING AND CONE PENETROMETER TEST
LOCATION PLAN

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA

DRAWN BY: J.L.S.	PLOT DATE: 3 AUG 12	CADD FILE: LOCATION PLAN.DGN
CHECKED BY: J.M.G.	JOB NO.: 21874	ENCLOSURE 1

APPENDIX I

LEGEND AND NOTES FOR LOG OF BORING AND TEST RESULTS

PP — Pocket penetrometer: Resistance in tons per square foot

SPT — Standard Penetration Test: Number of blows of a 140-lb hammer dropped 30 inches required to drive 2-in. O.D., 1.4-in. I.D. sampler a distance of 1 foot into the soil after first seating it 6 inches

SPLR — Type of Sampling: Shelby; SPT; Auger; No sample

SYMBOL — Clay; Silt; Sand; Peat/Humus; Shells; Stone/Gravel

Predominant type shown heavy; Modifying type shown light

USC — Unified Soil Classification

DENSITY — Unit weight in pounds per cubic foot

SHEAR TESTS

TYPE

- UC — Unconfined compression shear
- OB — Unconsolidated undrained triaxial compression shear on one specimen confined at the approximate overburden pressure
- UU — Unconsolidated undrained triaxial compression shear
- CU — Consolidated undrained triaxial compression shear
- DS — Direct shear

Ø — Angle of internal friction in degrees

c — Cohesion in pounds per square foot

ATTERBERG LIMITS

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

OTHER TESTS

- CON — Consolidation
- PD — Particle size distribution (sieve and/or hydrometer)
- k — Coefficient of permeability in centimeters per second
- SP — Swelling pressure in pounds per square foot

Other laboratory test results reported on separate figures

GENERAL NOTES

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

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

EUSTIS ENGINEERING

LOG OF BORING AND TEST RESULTS

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 21874 **Date Drilled:** 7/18-19/12 **Boring:** 1

Refer to "Legends & Notes"

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
0					Medium stiff brown clay w/silt pockets & roots	CH	1	0-1	28										
	1.75				Medium stiff to stiff tan & gray clay w/silt pockets & roots	CH	2	2-3	35	85	115	UC	--	1117					
	0.50						3	4-5	47	71	104	UC	--	590	75	25	50		
					Very soft gray & dark gray organic clay w/roots, decayed wood, & humus layers	OH	4	5.5-7	209										
							5	7.5-9	169										
10					Very soft gray clay w/silt pockets, roots, & decayed wood	CH	6	9.5-11	108										
	0.50						7	12-13	61	62	100	UC	--	241					
							8	14-15	64	60	98	UC	--	220					
20	0.25				Loose gray silty sand w/decayed wood	SM	9	18-20	28	94	121	OB	0	476					
		2			Very soft gray silty clay w/fine sand pockets & layers	CL	10	23.5-25	42										
30					Very soft to soft gray clay w/trace of decayed wood, trace of organic matter, & fine sand pockets	CH	11	28-30											
							12	33-35	77	54	96	UC	--	322	93	23	70		
40							13	38-40											
					Very loose gray fine sand w/clay layers & shell fragments	SP	14	43-45	29	91	118	OB	0	202					
50		50=6"			Very dense gray fine sand w/clay layers	SP	15	48.5-50	23										

Comments: Latitude: 30° 00.286' N
Longitude: 90° 00.358' W

EUSTIS ENGINEERING

LOG OF BORING AND TEST RESULTS

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 21874 **Date Drilled:** 7/18-19/12 **Boring:** 1

Refer to "Legends & Notes"

Scale In Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Shear Tests ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50			Very dense gray fine sand w/clay layers	SP													
			Soft gray sandy clay w/shell fragments	CL	16	53-55	31	88	115	UC	--	226					
	0.25		Soft gray clay w/organic matter, fine sand lenses & pockets, & shell fragments	CH	17	58-60											
60					18	63-65	43	75	107	OB	0	308					
		69	Very dense gray fine sand w/shells	SP	19	68.5-70.5	26										
70	0.75		Soft to medium stiff gray clay w/trace of organic matter, sand lenses & layers, & shell fragments	CH	20	73-75	47	75	110	UC	--	921					
		59	Very dense gray fine sand	SP	21 22	78.5-79.5 79.5-80											
80																	
90																	
100																	

Comments: Latitude: 30° 00.286' N
Longitude: 90° 00.358' W

EUSTIS ENGINEERING

LOG OF BORING AND TEST RESULTS

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 21874 **Date Drilled:** 7/19/12 **Boring:** 2

Refer to "Legends & Notes"

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Shear Tests ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0	0.50				Stiff gray & brown clay w/silt pockets, decayed wood, & roots	CH	1	0-1	36										
	1.50						2	1-3	34	87	116	UC	--	1572					
	1.25				Very soft to soft brown humus w/roots	Pt	3	3-5	363	14	63	UC	--	281					
							4	5-7	285										
					Very soft gray clay w/roots & silt pockets	CH	5	7-9	112										
10					Very soft gray organic clay w/roots & wood	OH	6	9-11	167										
							7	11-13	221										
					Very soft gray clay w/roots & decayed wood	CH	8	13-15	122						116	31	85		
					Very soft to soft gray clay w/silty sand pockets & lenses	CH	9	18-20	57	67	105	UC	--	154					
20																			
							10	23-25											
							11	28-30	70	57	98	UC	--	215					
30																			
							12	33-35											
							13	38-40	79	53	94	UC	--	341					
40					Very loose gray clayey sand w/shell fragments	SC													
		6					14	43.5-45	27										
50							15	48-50	26										

Comments: Latitude: 30° 00.309' N
Longitude: 90° 00.126' W

EUSTIS ENGINEERING

LOG OF BORING AND TEST RESULTS

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 21874 **Date Drilled:** 7/19/12 **Boring:** 2 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
50					Very loose gray clayey sand w/shell fragments	SC													
		4			Soft to medium stiff gray clay w/silt lenses & pockets, fine sand pockets, & shells	CH	16	53.5-55											
60	0.25						17	58-60	52	70	106	UC	--	693					
							18	63-65											
70	1.50						19	68-70	43	77	110	UC	--	619	67	22	45		
	0.75						20	73-75											
80	0.25						21	78-80	38	81	112	UC	--	815					
90																			
100																			

Comments: Latitude: 30° 00.309' N
Longitude: 90° 00.126' W

EUSTIS ENGINEERING

LOG OF BORING AND TEST RESULTS

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 21874 **Date Drilled:** 7/20 & 23/12 **Boring:** 3

Refer to "Legends & Notes"

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Shear Tests ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0	1.50				Medium stiff brown & gray clay w/silt pockets & shell fragments	CH	1	0-1											
	1.25						2	1-3	45	76	110	UC	--	960					
					Very soft brown organic clay	OH	3	3-5	192										
					Very soft dark brown humus	Pt	4	5-7	396										
					Extremely soft gray & brown organic clay w/roots	OH	5	7-9	164	31	82	UC	--	94					
10							6	9-11	122	39	87	UC	--	66					
					Very soft gray organic clay w/wood & roots	OH	7	11-13	157										
							8	13-15	147										
20		3					9	18.5-20											
	0.25				Very soft gray clay w/silt lenses & pockets, decayed wood, & silty sand pockets	CH	10	23-25	50	72	108	UC	--	180					
30							11	28-30											
							12	33-35	84	51	94	UC	--	129					
40							13	38-40											
	0.25				Loose gray clayey sand w/shell fragments	SC	14	43-45	28	94	119	OB	0	207					
50		3					15	48.5-50.5											

Comments: Latitude: 30° 00.150' N
Longitude: 90° 00.237' W

EUSTIS ENGINEERING

LOG OF BORING AND TEST RESULTS

PROPOSED INDUSTRIAL FACILITY
ALMONASTER BOULEVARD
NEW ORLEANS, LOUISIANA

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 21874 **Date Drilled:** 7/20 & 23/12 **Boring:** 3

Refer to "Legends & Notes"

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Shear Tests ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50					Loose gray clayey sand w/shell fragments	SC													
	1.50				Medium stiff greenish-gray & tan clay w/silt pockets	CH	16	53-55	51	71	107	OB	0	550					
60	1.25				Stiff light gray clay w/organic matter & silt pockets	CH	17	58-60	49	71	106	UC	--	1073					
	1.00				Medium stiff gray & tan silty clay	CL	18	63-65	35	87	117	UC	--	985					
70	0.75				Medium stiff gray clay w/silt layers & pockets, organic matter, decayed wood, & concretions	CH	19	68-70	54	66	101	UC	--	530					
		56			Dense to very dense gray silty sand w/clay layers, shell fragments, & organic matter	SM	20	73.5-75.5											
80		31					21	78.5-80											
90																			
100																			

Comments: Latitude: 30° 00.150' N
Longitude: 90° 00.237' W

APPENDIX II

CONE PENETROMETER
SOIL BEHAVIOR TYPE LEGEND

SOIL BEHAVIOR TYPE LEGEND

1 - SENSITIVE FINE GRAINED

2 - ORGANIC MATERIAL

3 - CLAY

4 - SILTY CLAY TO CLAY

5 - CLAYEY SILT TO SILTY CLAY

6 - SANDY SILT TO CLAYEY SILT

7 - SILTY SAND TO SANDY SILT

8 - SAND TO SILTY SAND

9 - SAND

10 - GRAVELLY SAND TO SAND

11 - VERY STIFF FINE GRAINED (*)

12 - SAND TO CLAYEY SAND (*)

*OVERCONSOLIDATED OR CEMENTED

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

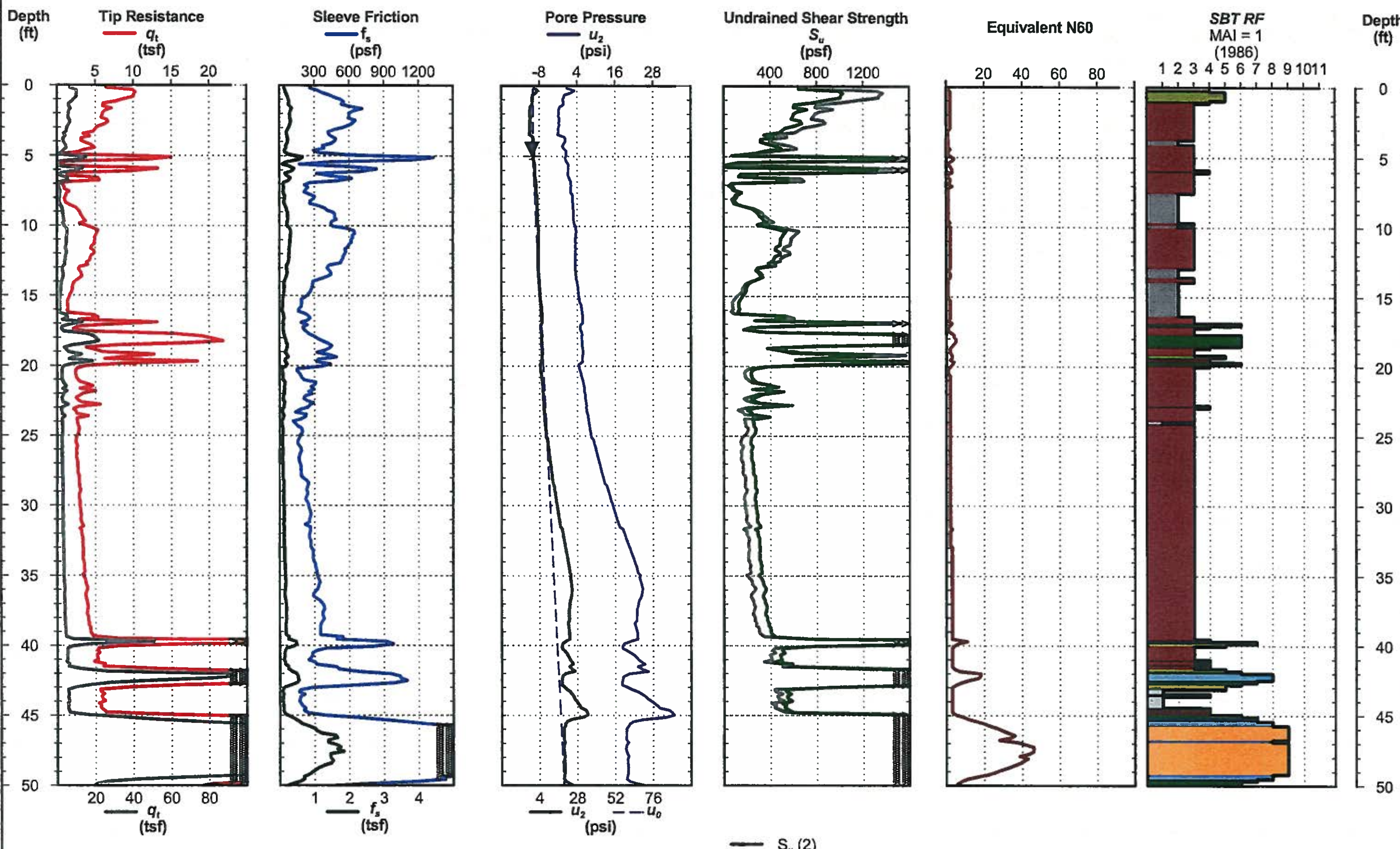
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-1

Latitude: 30.004767
Longitude: 90.006533
Date: 07/24/12
Operator: G. Reitmeyer

Water Depth: 5.0ft
Total Depth: 81.0 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-1.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

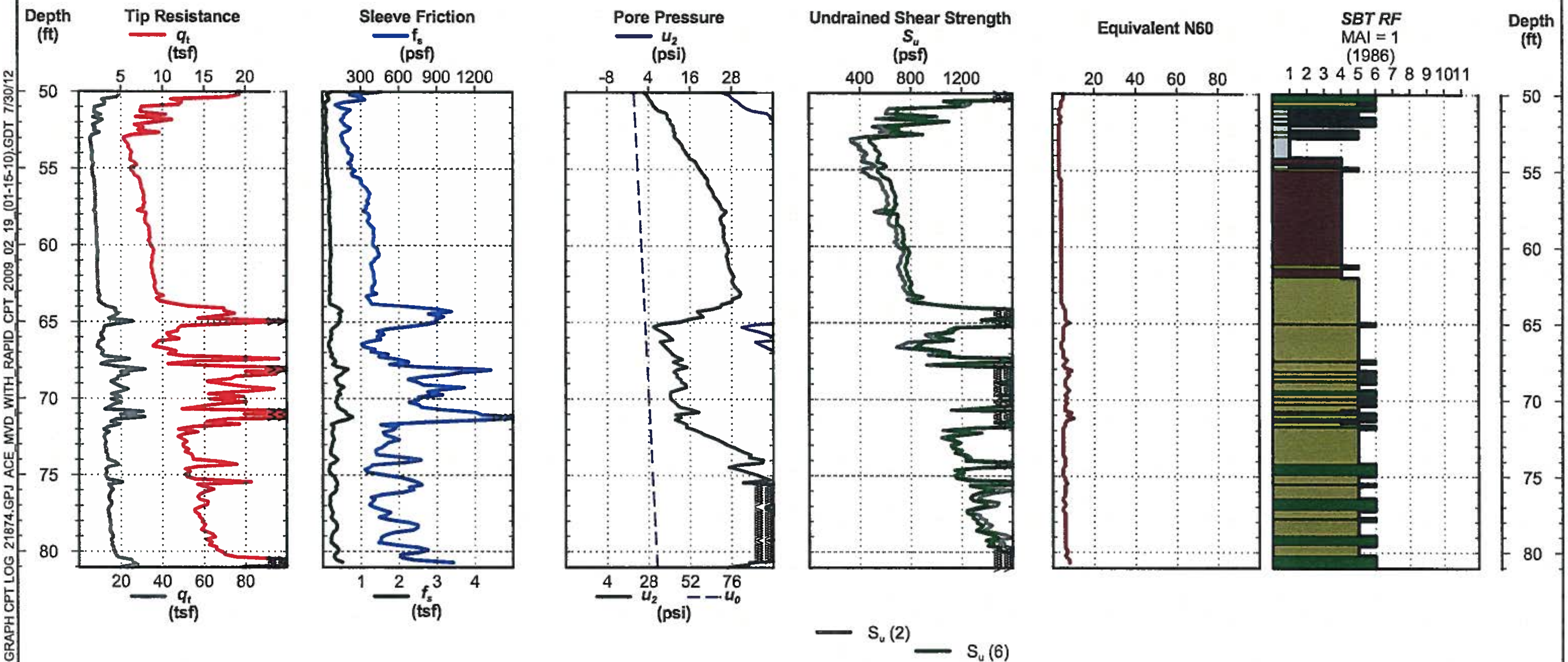
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-1

Latitude: 30.004767
Longitude: 90.006533
Date: 07/24/12
Operator: G. Reitmeyer

Water Depth: 5.0ft
Total Depth: 81.0 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-1.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

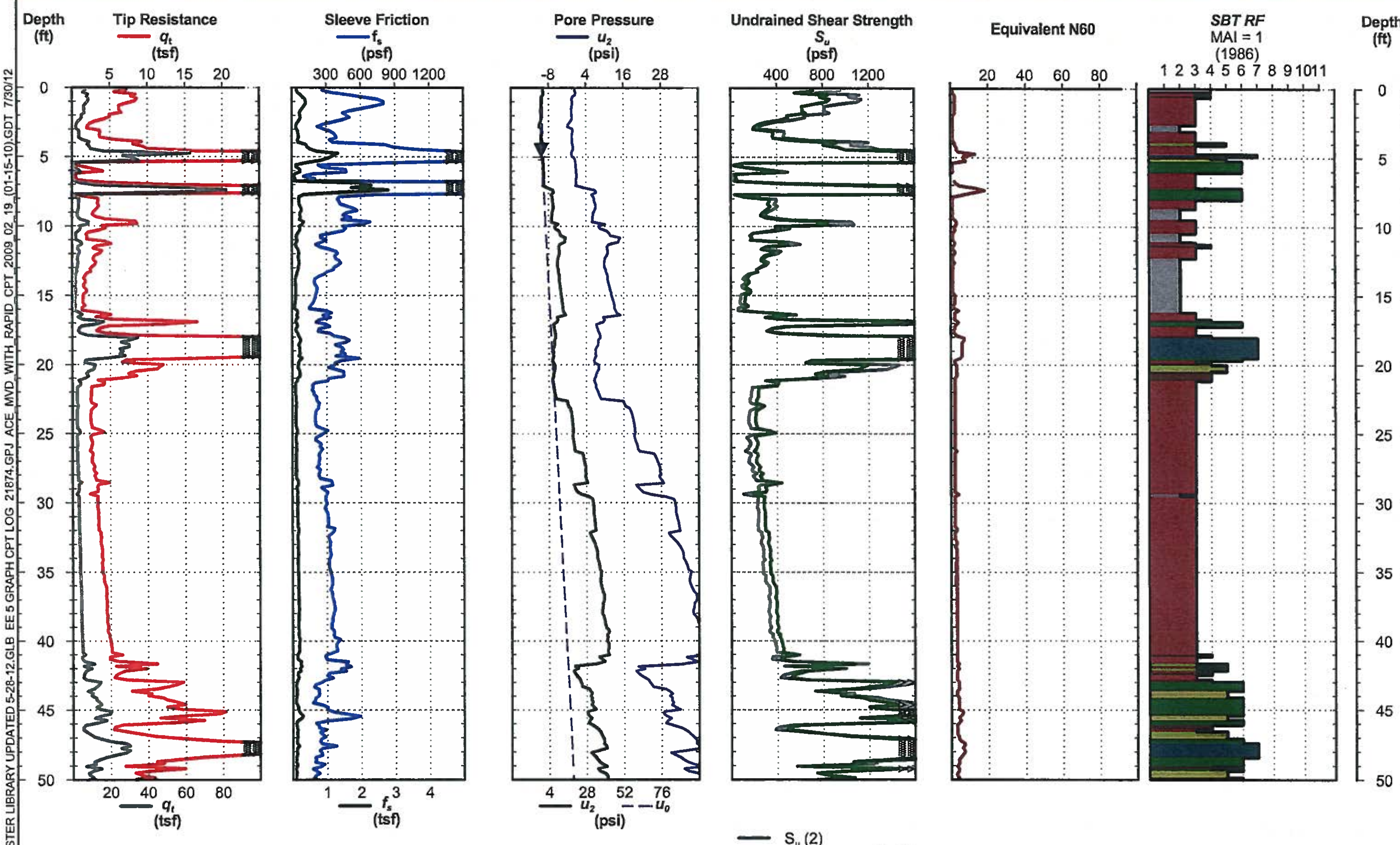
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-2

Latitude: 30.0049
Longitude: 90.00455
Date: 07/23/12
Operator: G. Reitmeyer

Water Depth: 5.0ft
Total Depth: 79.7 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-2.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

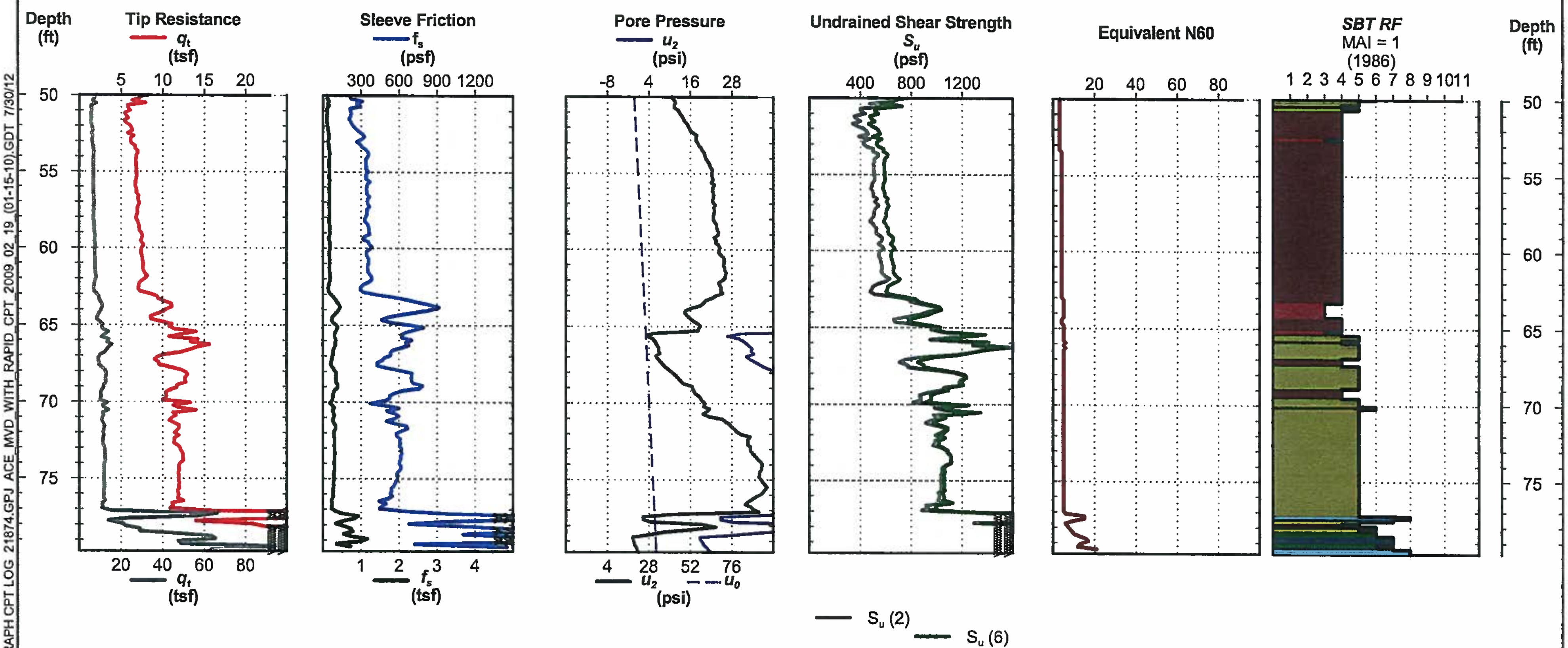
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-2

Latitude: 30.0049
Longitude: 90.00455
Date: 07/23/12
Operator: G. Reitmeyer

Water Depth: 5.0ft
Total Depth: 79.7 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-2.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

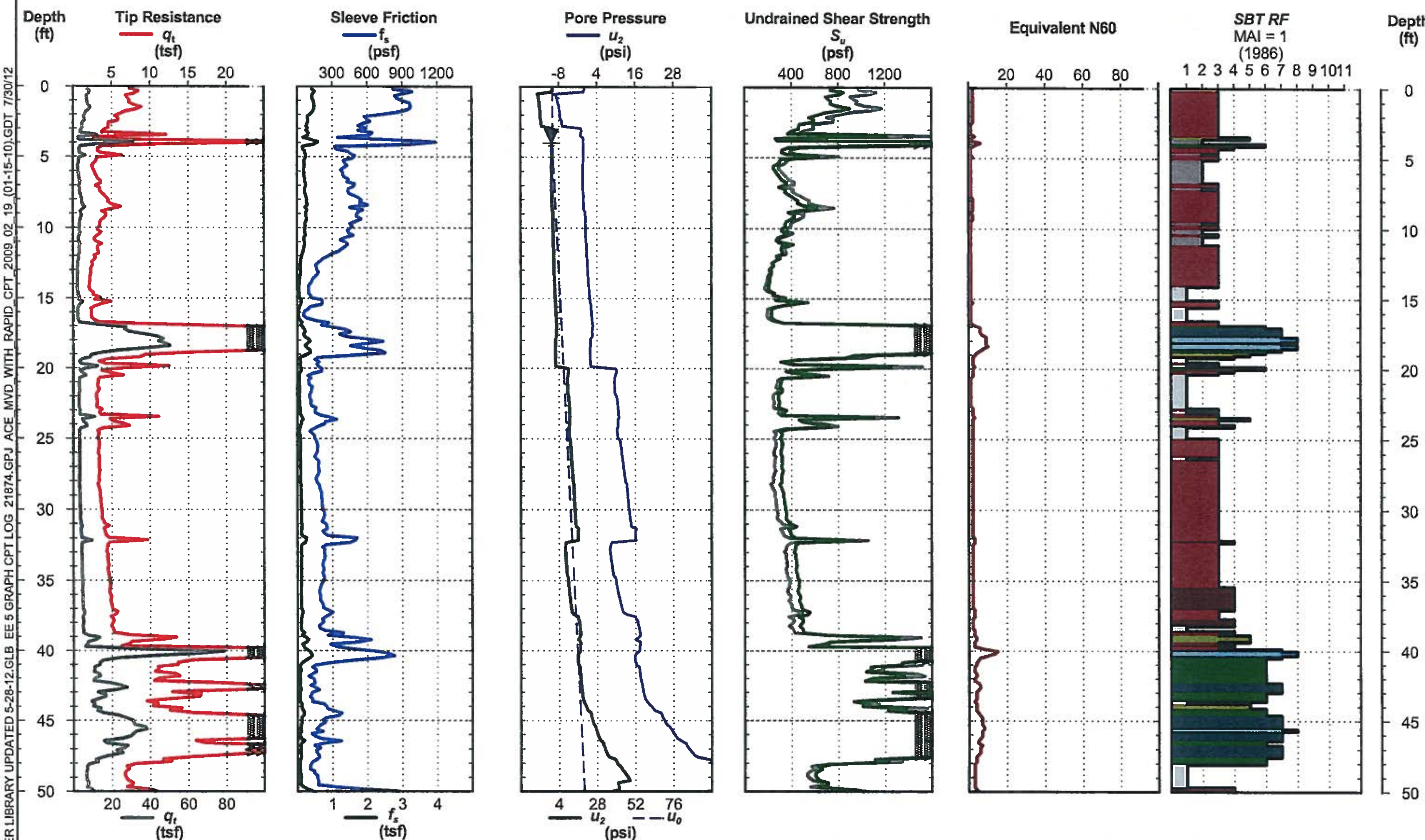
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-3

Latitude: 30.006233
Longitude: 90.002017
Date: 07/25/12
Operator: G. Reitmeyer

Water Depth: 4.0ft
Total Depth: 80.9 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-3x.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

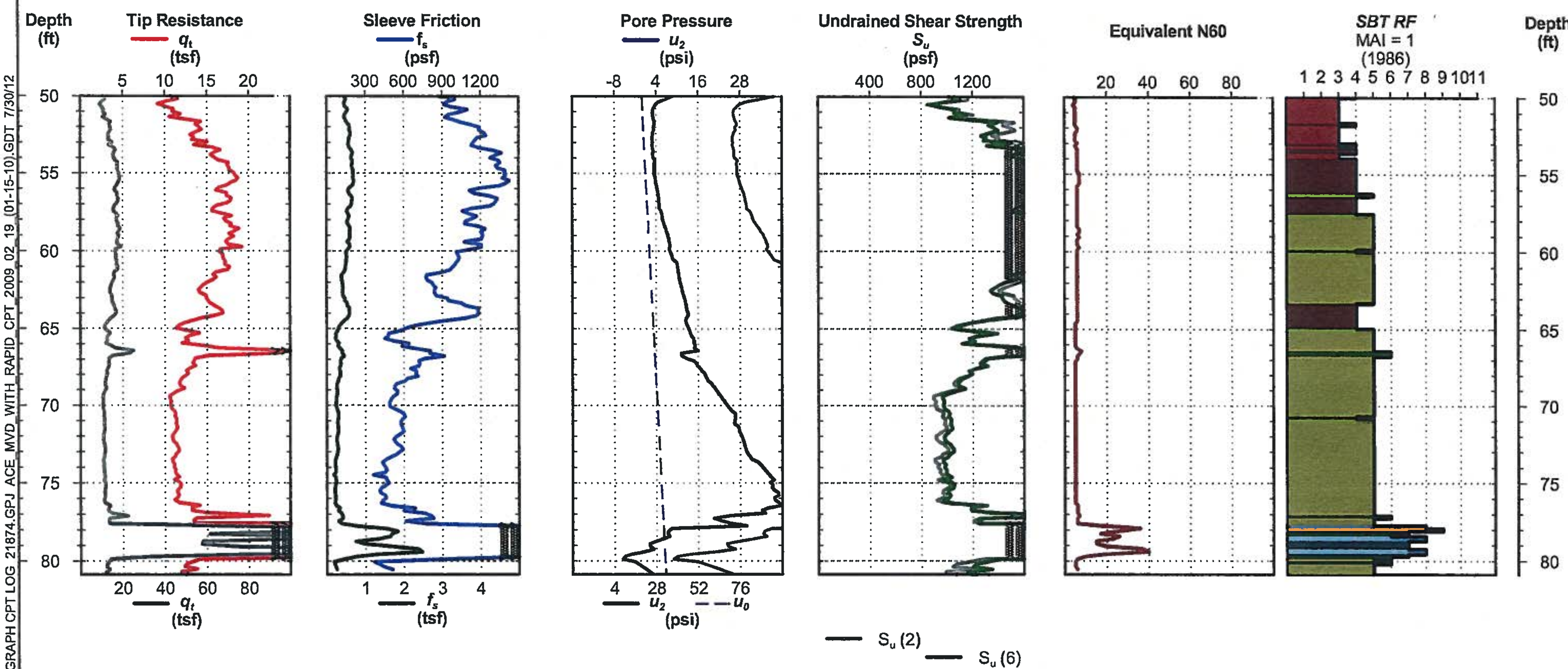
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-3

Latitude: 30.006233
Longitude: 90.002017
Date: 07/25/12
Operator: G. Reitmeyer

Water Depth: 4.0ft
Total Depth: 80.9 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-3x.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

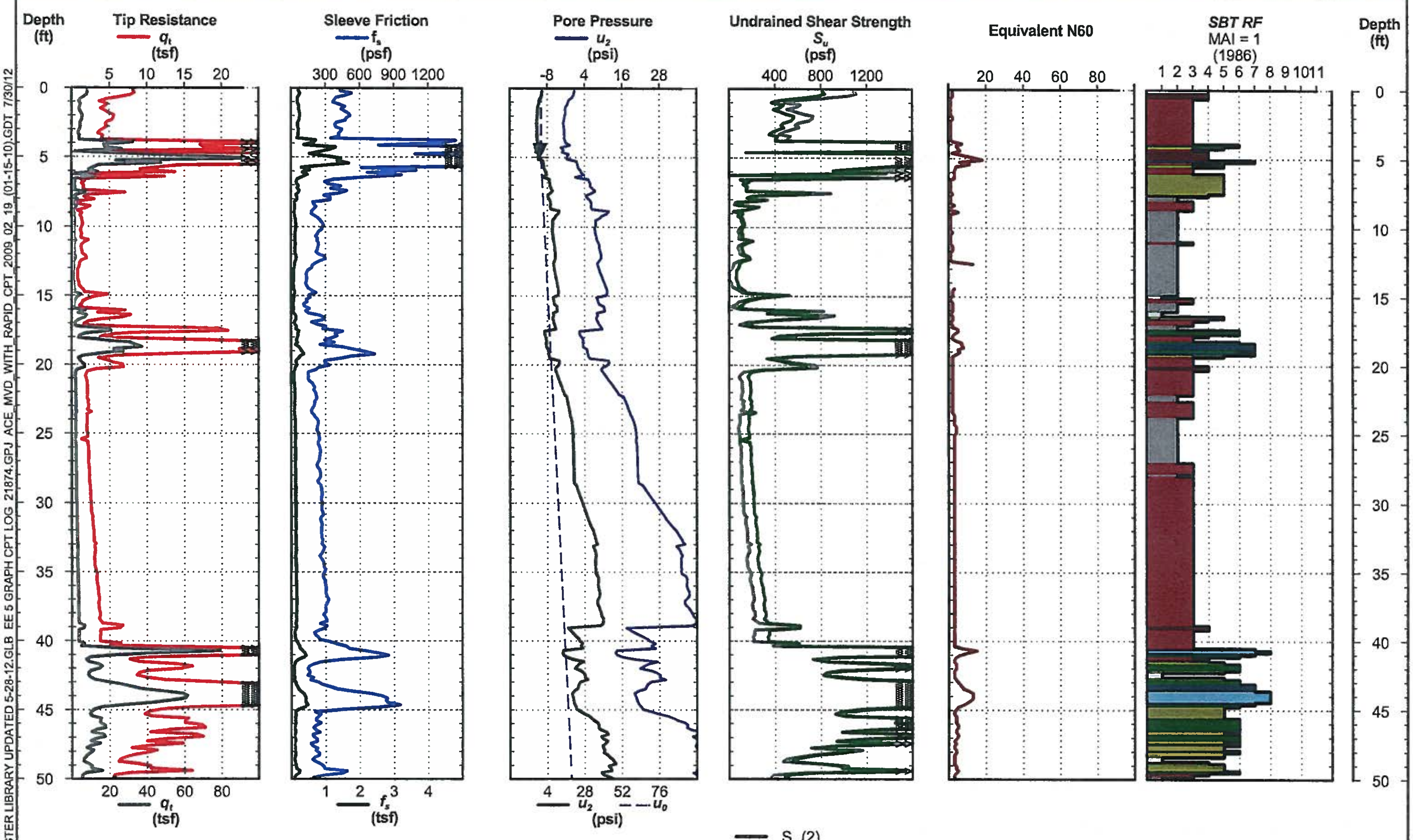
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-4

Latitude: 30.0028
Longitude: 90.005967
Date: 07/24/12
Operator: G. Reitmeyer

Water Depth: 5.0ft
Total Depth: 81.0 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-4.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

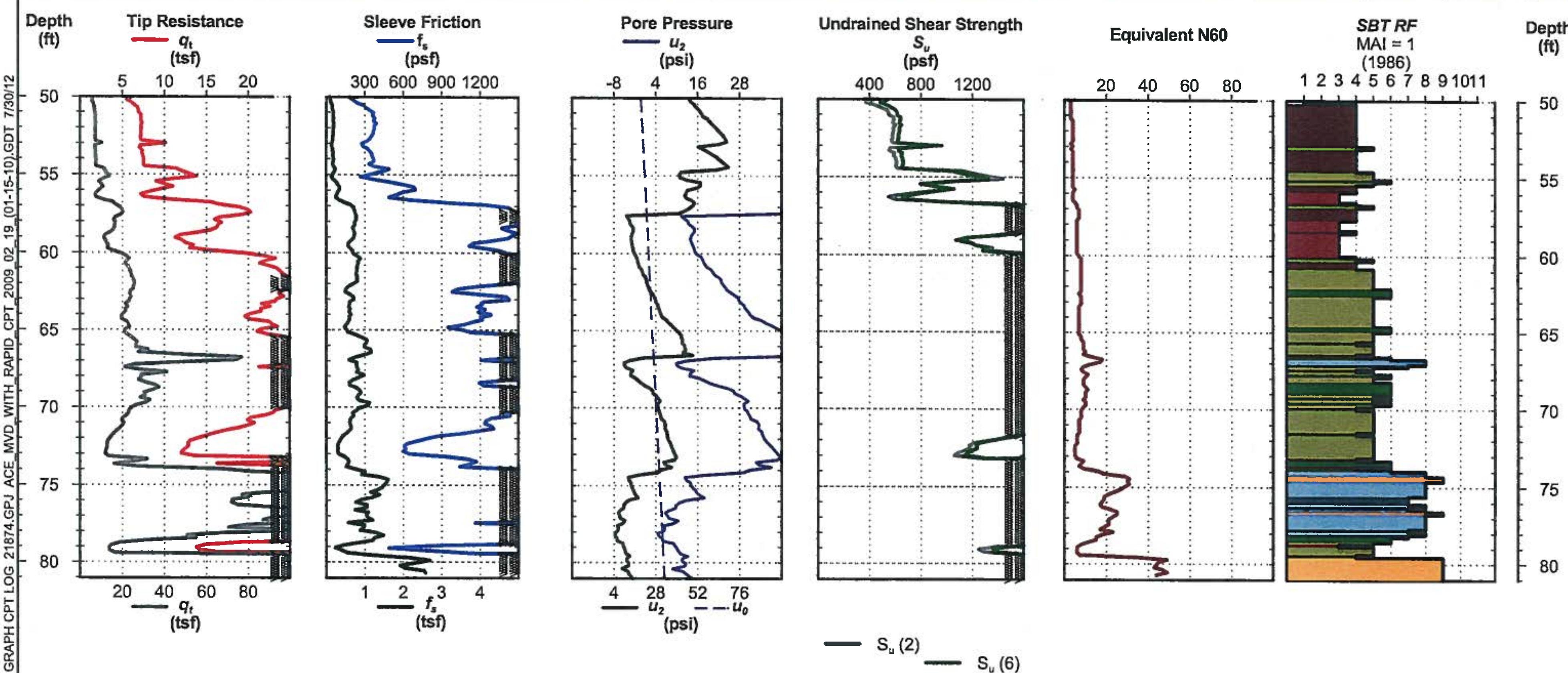
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-4

Latitude: 30.0028
Longitude: 90.005967
Date: 07/24/12
Operator: G. Reitmeyer

Water Depth: 5.0ft
Total Depth: 81.0 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-4.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

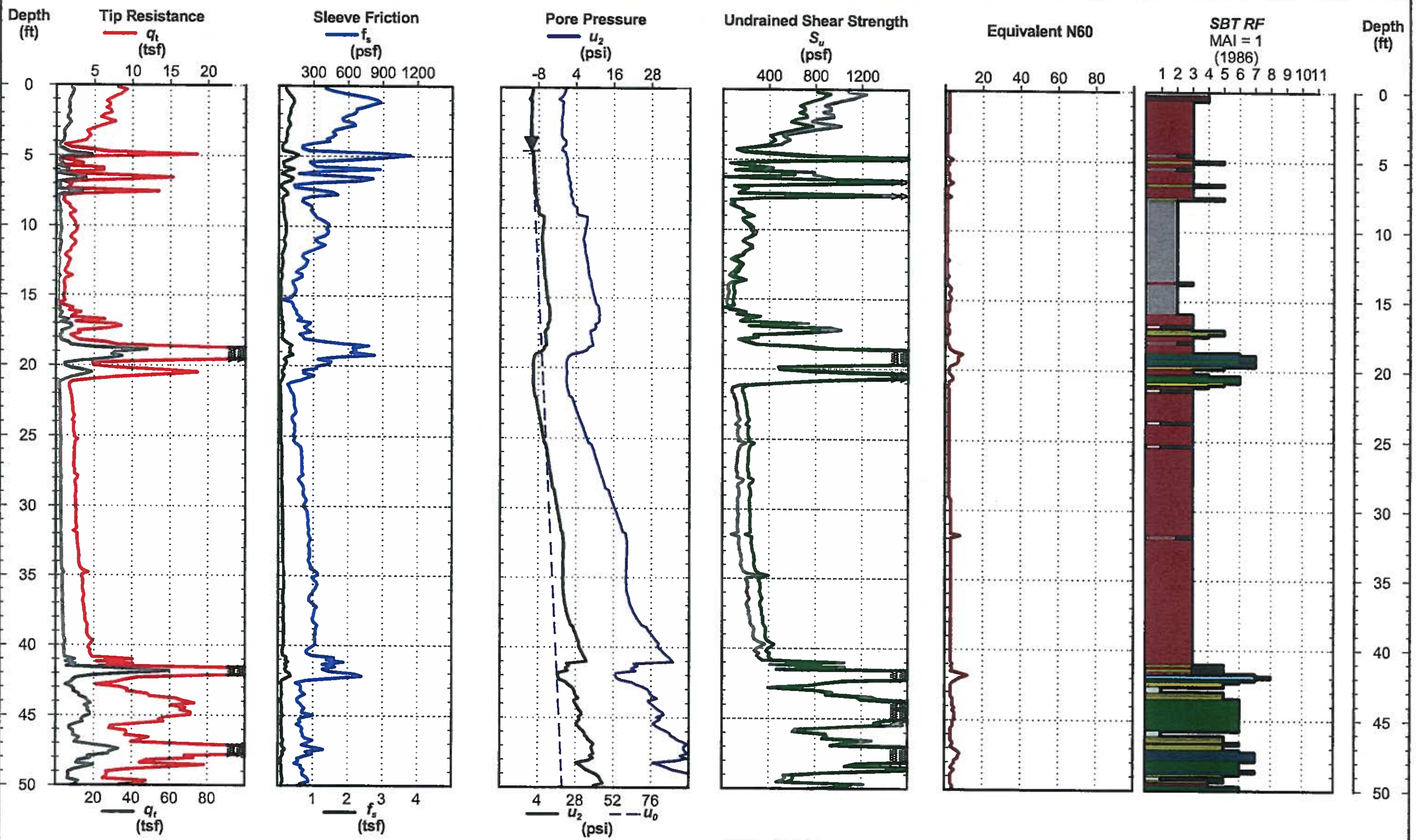
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

Latitude: 30.004183
Longitude: 90.0042
Date: 07/24/12
Operator: G. Reitmeyer

CPT-5

Water Depth: 4.5ft
Total Depth: 81.0 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-5.cpt

EUSTIS ENGINEERING

Proposed Industrial Facility
Almonaster Boulevard

New Orleans, Louisiana
Project No: 21874

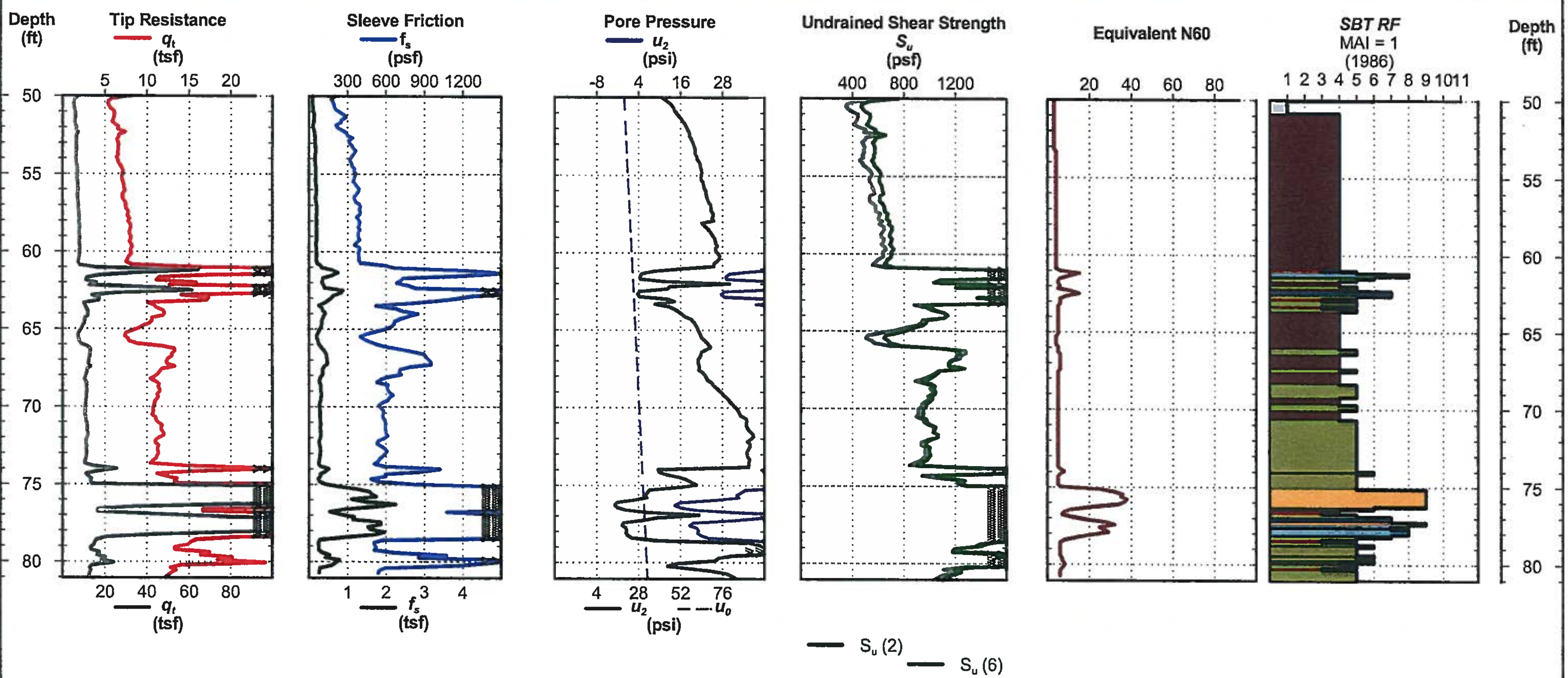
CPT ID/Net Area Ratio: DTA1061 / 0.8

Cone Penetration Test

CPT-5

Latitude: 30.004183
Longitude: 90.0042
Date: 07/24/12
Operator: G. Reitmeyer

Water Depth: 4.5ft
Total Depth: 81.0 ft



Notes: Soil behavior type was determined using friction ratio classification chart (after Robertson *et al.*, 1986).
Test performed in general accordance with ASTM D5778-07.

Electronic Filename: 21874IF CPT-5.cpt

CPT Correlations

References are in parenthesis next to the appropriate equation.

General

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

$B_q = (u_2-u_0)/(q_t-\sigma_{vo})$
$Q_t=(q_t-\sigma_{vo})/\sigma'_{vo}$
$F_r=100\% \cdot f_s/(q_t-\sigma_{vo})$
$I_c = ((3.47-\log Q_t)^2+(\log F_r+1.22)^2)^{0.5}$

K_o

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

Stress History

$OCR = \sigma_p'/\sigma_{vo}$

(OCR 1)	$\sigma_p' = 0.33(q_t - \sigma_{vo})$	8
(OCR 2)	$\sigma_p' = 0.53(u_2 - u_o)$	9
(OCR 3)	$\sigma_p' = 0.60(q_t - u_2)$	9

N-Value

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

Undrained Shear Strength

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

Drained Friction Angle

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

Unit Weight

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

Relative Density and Void Ratio

D_R (1)	$D_R = 100(q_{c1}/305)^{1/2}$	where, $q_{c1} = q_c/(\sigma_{vo}')^{1/2}$	1
D_R (2)	$D_R = -1.292 + 0.268\ln(q_c \cdot (\sigma_{vo}'^{-0.5}))$	18	
D_R (3)	$D_R = (1/2.41)\cdot\ln(q_{c1}/15.7)$	3	

D_R (4) $D_R = 1/2.91 * \ln((q_c/(61* \sigma'_{vo}{}^{0.7\,t}))*100$ 20

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

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

Compressibility

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

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

M (2) $M = q_c \cdot 10^{(1.09-0.0075D_R)}$ *sands*

M (3) $M = 8.25\,(q_t - \sigma_{vo})$ *clays*

Sensitivity

S_t (1) $S_t = 7.5/R_f$ 2

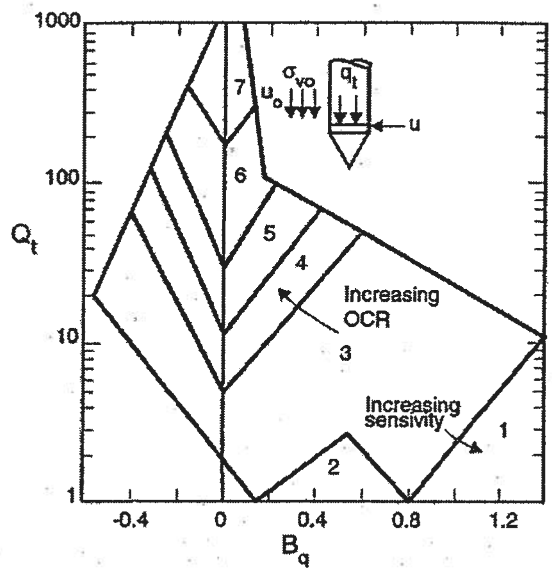
S_t (2) $S_t = (q_t - \sigma_{vo})/(15 \cdot f_s)$ 2

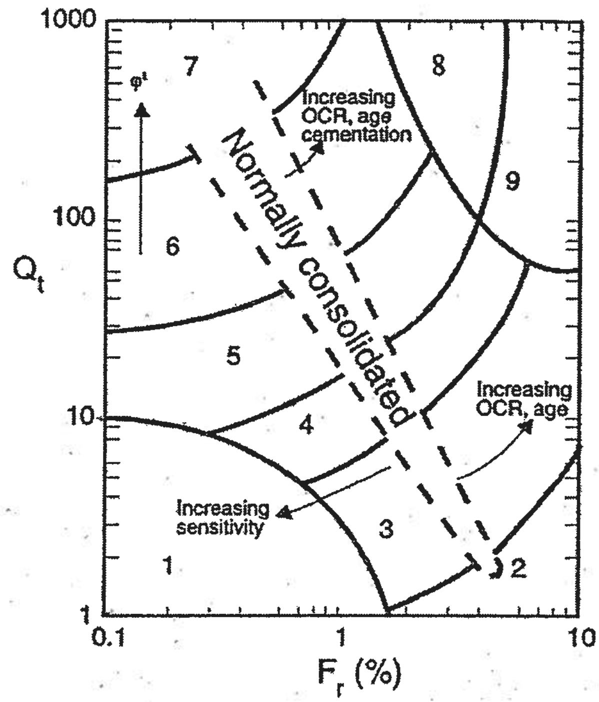
Fines Content

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

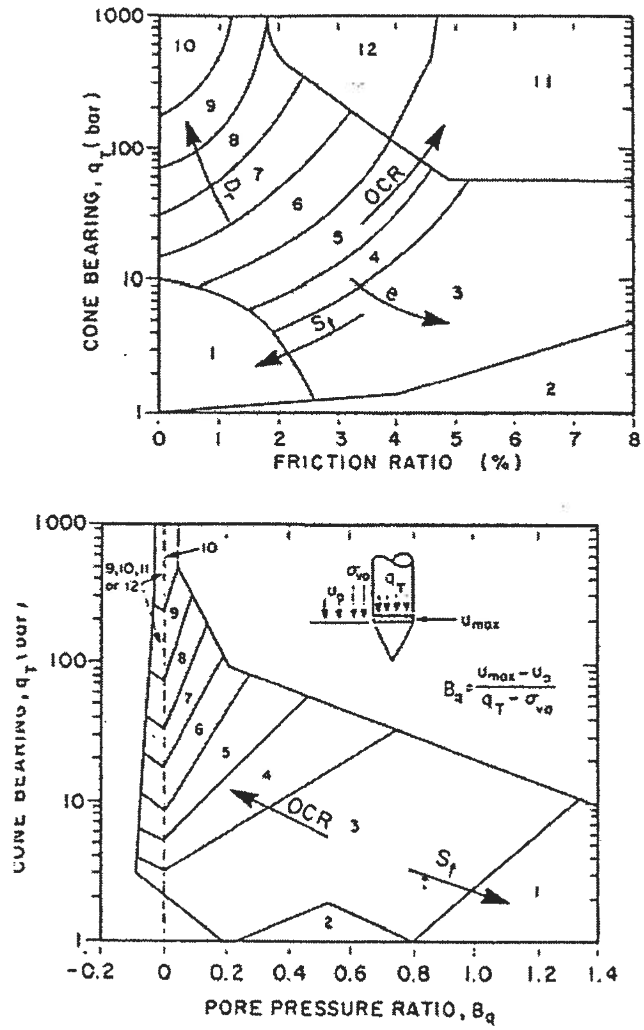
$FC = [5.31(I_{cfs})^{2.31}]+9.61$, where $I_{cfs} = [(1.95-\mathrm{Log}Q_t)^2+(\log F_r+1.78)^2]^{0.5}$ 4

Normalized Soil Behavior Types - Robertson & Campanella (1990)





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



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