Exhibit Y. Port of Vinton Site Preliminary Geotechnical Engineering Report

SWLA ECONOMIC DEVELOPMENT ALLIANCE

CSRS
IMAGINE SHAPE DELIVER

Port of Vinton Site
Preliminary Geotechnical
Engineering Report

Preliminary Geotechnical Engineering Report

Port of Vinton Site
Vinton, Louisiana
May 16, 2018
Terracon Project No. EH175369

Prepared for:
SWLA Economic Development Alliance
Lake Charles, Louisiana

Prepared by:
Terracon Consultants, Inc.
Baton Rouge, Louisiana

May 16, 2018

SWLA Economic Development Alliance
4310 Ryan Street
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Attn: Mr. Gus Fontenot
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Re: Preliminary Geotechnical Engineering Report
Port of Vinton Site
Gray Road and Johnny Breaux Road
Vinton, Louisiana
Terracon Project No. EH175369

Dear Mr. Fontenot:

We have completed the Preliminary Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with our proposal number PEH175369 dated December 22, 2017. This report presents the findings of the subsurface exploration and provides preliminary geotechnical recommendations concerning earthwork and the design and construction of foundations and floor slabs for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

STATE OF LOUISIANA
STEPHEN E. GREABER
License No. 26107
PROFESSIONAL ENGINEER

John M. Voelker, EI
Engineer-in-Training

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REPORT TOPICS

Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

Preliminary Geotechnical Engineering Report

Port of Vinton Site
Gray Road and Johnny Breaux Road
Vinton, Louisiana
Terracon Project No. EH175369
May 16, 2018

INTRODUCTION

This report presents the results of our subsurface exploration and preliminary geotechnical engineering services performed for the potential development to be located at Gray Road and Johnny Breaux Road in Vinton, Louisiana. The purpose of these services is to provide information and preliminary geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Site preparation and earthwork
- Foundation design and construction
- Floor slab design and construction
- Seismic site classification per IBC
- Pavement design and construction

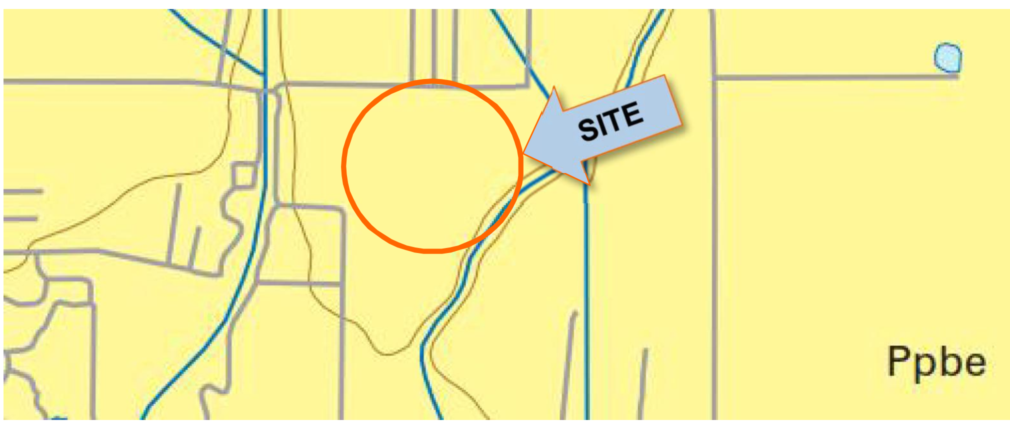
The geotechnical engineering scope of services for this project included the advancement of 4 test borings to depths ranging from approximately 20 to 100 feet below existing site grades and 3 CPT soundings to depths of approximately 50 feet below existing site grades.

Maps showing the site and boring locations are shown in the **Site Location** and **Exploration Plan** sections, respectively. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring logs in the **Exploration Results** section of this report.

SITE CONDITIONS

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project is located at southeast quadrant of Gray Road and Johnny Breaux Road in Vinton, Louisiana. Approximately 157 Acres. Latitude: 30.1581, Longitude: 93.5578 (approximate center of parcel). See **Site Location**.

Item	Description
Existing Improvements	Undeveloped pasture land with apparent pipeline easement on the north end of the site.
Current Ground Cover	Pasture grass and sparse trees.
Existing Topography	Relatively flat with 1 to 3 feet of grade change. Drainage ditch located on north side of site and along east property boundary.
Geology	The property is located within an area of Beaumont Alloformation (Ppbe) of Prairie Terrace deposits of Pleistocene Age. Beaumont Alloformation consists of plain deposits of late to middle Pleistocene streams: the oldest alloformation and topographically highest surface of the Prairie Allogroup units of southwestern Louisiana. It exhibits the relict channels of the Red and Calcasieu rivers, and includes deposits of the Ingleside barrier trend within the Lake Charles quadrangle. These Pleistocene Age deposits typically consist of medium stiff to very stiff tan and light gray silty clays and clays with silt and sand layering. The soils within the Prairie Terrace deposits typically provide good foundation support for relatively light to moderately loaded structures, fair pavement subgrades, and are overconsolidated, and normally only marginally compressible. In some areas that are very dry and desiccated, the potential for expansive properties exists, but these conditions are not typical of the Prairie Terrace deposits.  **Lake Charles 30x60 Minute Geologic Quadrangle (Louisiana Geological Survey, 2002)**

We also collected photographs at the time of our field exploration program. Representative photos are provided in our **Photography Log**.

PHOTOGRAPHY LOG

PROJECT DESCRIPTION

Our initial understanding of the project was provided in our proposal and was discussed in the project planning stage. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	Information about the project site was provided by CSRS on behalf of the SWLA Economic Development Alliance. The information included an aerial photograph and USGS topographic map showing the outline of the property boundary.
Project Description	The LED Certified Site Application requires we obtain a preliminary geotechnical investigation of the site generally characterizing the site's soil and groundwater conditions.
Proposed Structures	No information regarding structures was available at this time. It is our understanding that the site is a potential candidate for industrial development. The narrative of the geotechnical report should clearly state the approximate load bearing capacity of a 14" concrete or pipe pile or other similar, commonly used geotechnical support structures used in a major petrochemical plant. It should also estimate the approximate size of spread footings for 2-3 types of industrial structures (tanks, pipe racks, etc.).
Pavements	It is anticipated that paved driveways may be constructed on the industrial capacities. We recommend rigid (concrete) pavement sections be constructed for this application. Anticipated traffic is as follows: ■ Autos/light trucks: 1,000 vehicles per week ■ Dump truck vehicles: 50 vehicles per week ■ Tractor-trailer trucks: 10 vehicles per week The pavement design period is 20 years.
Estimated Start of Construction	Unknown

GEOTECHNICAL CHARACTERIZATION

Subsurface Profile

Subsurface conditions at the boring locations can be generalized as follows:

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Description	Consistency/Density
Surface	3 to 4 inches	Topsoil: brown, friable and contained significant organic matter	N/A
1	11 to 18	Lean Clay	Medium Stiff to Stiff
2	30	Fat Clay with some Lean Clay	Stiff to Very Stiff
3	78	Fat Clay	Very Stiff to Hard
4	88	Silt with Sand	Very Dense
5	98	Silty Sand	Very Dense
6	100	Silty Clay	Hard
Notable Variations	1. Medium Dense clayey sand layers were encountered in borings B-01 and B-03 from 6-16 feet.		

Conditions encountered at each boring location are indicated on the individual boring logs shown in the **Exploration Results** section and are attached to this report. Stratification boundaries on the boring logs represent the approximate location of changes in native soil types; in situ, the transition between materials may be gradual.

Groundwater Conditions

The boreholes were observed while drilling for the presence and level of groundwater. For borings that encountered groundwater, the drilling operations were suspended for about 15 minutes to observe the change in water level over that period. The water levels observed in the boreholes can be found on the boring logs in **Exploration Results**, and are summarized below.

Boring Number	Approximate Depth to Groundwater while Drilling (feet) [1]	Approximate Depth to Groundwater after about 15 Minutes (feet) [1]
B-01	13	5
B-01-A	18	13.5
B-02	10	5
B-03	10	4.25

1. Below ground surface

Due to the low permeability of some of the soils encountered in the borings, a relatively long period may be necessary for a groundwater level to develop and stabilize in a borehole. Long term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

GEOTECHNICAL OVERVIEW

The near surface, medium stiff to stiff, medium plasticity lean clay and low plasticity silty clay and silt appeared to be stable while accessing the borings, but could become unstable with typical earthwork and construction traffic, especially after precipitation events. Effective drainage should be completed early in the construction sequence and maintained after construction to limit instability issues. If possible, the grading should be performed during the warmer and drier time of the year. If grading is performed during the winter months, an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional general site preparation recommendations including clearing, grubbing and proof-rolling are provided in the **Earthwork** section.

The **Shallow Foundations** section addresses support of moderately loaded industrial structures bearing on native stiff lean clay or structural fill. Shallow foundations are a viable option for this site for support of tanks, pipe racks, warehouses and other auxiliary structures. It is expected that settlement from these structures and up to 5 feet of fill will be within typical allowable ranges. We have calculated bearing capacities assuming 2 feet of fill above existing grades. Pipe rack foundations with a compression load of around 8-10 kips/support can be supported on minimum 3-ft x 3-ft footing. However, these types of foundation are more likely controlled by overturning moments; resulting in a slightly larger footing to resist eccentric loads. It is not uncommon in these soil condition to support pipe rack columns on shallow drilled shafts; which can be efficient in resisting lateral and overturning loads. Shafts that are on the order of 2 to 2-1/2-foot diameter with depths on the order of 15 to 20 feet are common.

We estimate that tanks up to 20 feet in diameter and 20 feet tall can be supported on ground supported mat or ring wall foundations with settlement of around 1 inch. Larger diameter steel tanks (e.g., API Steel Tanks) up to approximately 40 feet tall supported on concrete ring walls are also possible provided total settlement on the order of 3 to 4 inches is tolerable.

The **Floor Slabs** section addresses design considerations of slab-on-grade support of a typical warehouse building. For a typical warehouse type project, we do not expect that significant treatment or replacement of soils will be necessary based on the soil conditions encountered. In some cases, placing crushed stone below the slab to increase the modulus of subgrade reaction can be specified to aid in minimizing concrete thickness due to fork lifts or similar point load.

The heavily loaded industrial structures may be supported on a deep foundation consisting of driven pre-stressed concrete or open ended steel pipe. The **Deep Foundations** section addresses deep foundation support of the structures and provides preliminary pile capacity information.

A rigid pavement system is recommended for industrial developments and the **Pavements** section addresses the design considerations of pavement systems. Alternatively, unpaved gravel roads over geotextile fabric are commonly used.

The **General Comments** section provides an understanding of the report limitations.

EARTHWORK

It is anticipated that earthwork will include clearing and grubbing, proof-rolling, excavations and fill placement. The following sections provide preliminary recommendations for use in the preparation of specifications for the work. Preliminary recommendations include quality criteria as important to render the site in the state considered in our geotechnical engineering evaluation for foundations, floor slabs, and pavements.

Site Preparation

The site should be stripped of existing vegetation, trees, stumps, roots, grass, topsoil, organic laden soil, organic matter, and any rubble or debris encountered to prepare for construction of structures and pavements. Stripped materials consisting of vegetation and organic materials should be wasted off site or used to vegetate landscaped areas. Topsoil measurements were made at the boring locations; however, stripping depths between our boring locations and across the site could vary considerably. If roots are encountered, the entire root ball should be excavated such that the remaining roots measure 1 inch in diameter or less.

To observe for possible unstable areas, the subgrade should be proof-rolled, after stripping, with an adequately loaded vehicle such as a loaded scraper or fully loaded tandem axle dump truck. The vehicle should weigh between 20 to 25 Tons (total vehicle weight). The proof-rolling should be performed under the direction of the Geotechnical Engineer. Proof-rolling should be performed after a suitable period of dry weather to avoid degrading an otherwise acceptable subgrade and to reduce the amount of undercutting/remedial work required. Areas excessively deflecting under the proof-roll should be delineated and subsequently addressed by the Geotechnical Engineer. Such areas could be undercut, replaced with engineered fill and compacted. Widespread instability may require chemical treatment as specified by the Geotechnical Engineer at the time of construction. Excessively wet or dry material should either be removed or moisture conditioned and recompacted.

Grading and Drainage

All grades must provide effective drainage away from buildings during and after construction and should be maintained throughout the life of the structure. Water retained next to the building can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential floor slab and/or foundation movements, cracked slabs and walls, and roof leaks.

Earthwork Construction Considerations

Shallow excavations, for the proposed structure, are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of floor slabs. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over, or adjacent to, construction areas should be removed. If the subgrade desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted, prior to floor slab construction.

The groundwater table was encountered at about 5 feet below existing grade. Groundwater could affect excavation efforts, especially for over-excavation and replacement of lower strength soils. A temporary dewatering system consisting of sumps with pumps could be necessary to achieve the recommended depth of over-excavation.

Construction Observation and Testing

The earthwork efforts should be monitored under the direction of the Geotechnical Engineer. Monitoring should include documentation of adequate removal of vegetation and top soil, proof-rolling and mitigation of areas delineated by the proof-roll to require mitigation.

Each lift of compacted fill should be tested, evaluated, and reworked as necessary until approved by the Geotechnical Engineer prior to placement of additional lifts.

In areas of foundation excavations, the bearing subgrade should be evaluated under the direction of the Geotechnical Engineer. In the event unanticipated conditions are encountered, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer’s evaluation of subsurface conditions, including assessing variations and associated design changes.

SHALLOW FOUNDATIONS

The following design parameters are estimated for shallow foundations.

Preliminary Design Parameters – Spread Footing Compressive Loads

Item	Description
Preliminary Maximum Net Allowable Bearing pressure [1, 2]	
Pipe Racks and Warehouse Buildings - Isolated Columns up to 10-ft by 10-ft and Continuous Footings up to 3 feet wide	2,000 psf
Mat Foundations up to 15-ft by 15-ft	1,250 psf
Mat Foundations up to 20-ft by 20-ft	900 psf
Required Bearing Stratum [3]	Stiff lean clay, fat clay or structural fill. Bearing stratum should be verified by the Geotechnical Engineer
Estimated Total Settlement from Structural Loads [2]	About 1 inch
Estimated Differential Settlement [2, 4]	About 1/2 of total settlement

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. An appropriate factor of safety has been applied.
2. Values provided are for a fill height of 2 feet and assumed typical loads for the anticipated structures.
3. Unsuitable or soft soils should be over-excavated and replaced per the recommendations presented in the Earthwork section.
4. Differential settlements are as measured over a span of 40 feet.

DEEP FOUNDATIONS

Driven Pre-Stressed Concrete and Pipe Piles

Preliminary design recommendations and construction considerations for driven pre-stressed square concrete piles and steel open ended pipe piles based on the average soil conditions obtained from the widely spaced borings are provided. The length of piles is typically evaluated based upon the required resistance, but also on considerations of pile delivery access, pile handling, drivability, and group settlement considerations.

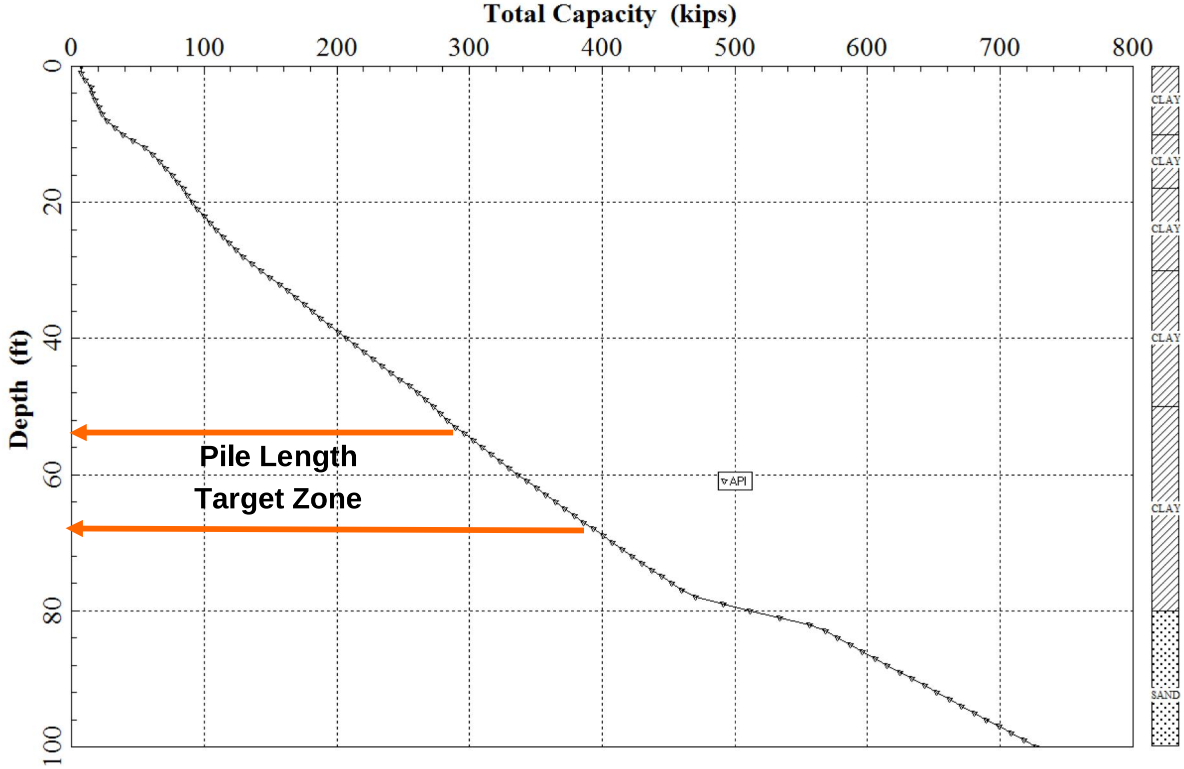
We have set a target zone for pile lengths based on anticipated loading and pile order length considerations. For example, a 14” concrete pile less than 66 feet in length can be lifted using a

1-point pick-up to maintain the piles within allowable tension stresses as shown on standard detail CS-216 by DOTD Bridge and Structural Design department. Typically, concrete piles longer than 90 feet require a two-point pick-up and an escort for delivery, which adds additional cost to the project. Other options for long concrete piles include design of the piles with cast-in mechanical or compression couplers. Pipe piles are typically delivered in 40-ft lengths and coupled using compression fittings that may or may not be welded depending on whether tension loading is present. The presence of very stiff to hard clays may limit the depth to which concrete piles can be efficiently installed without high stresses, so drivability may control.

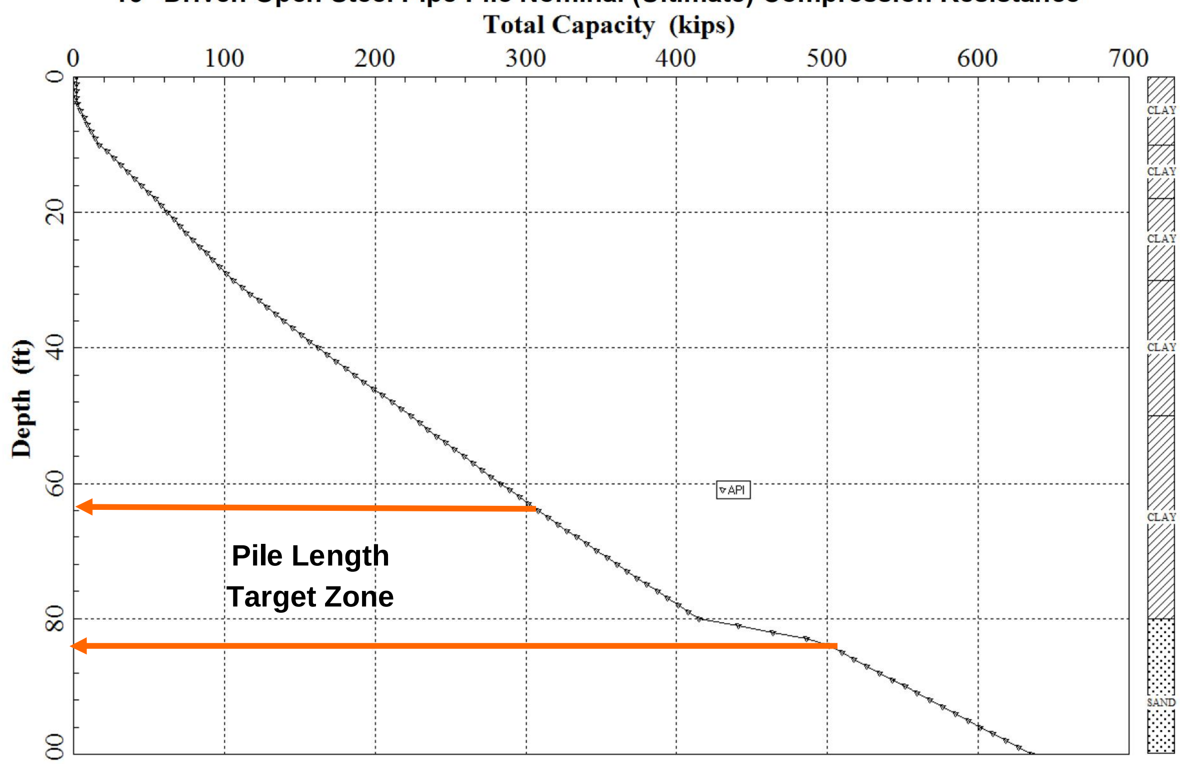
Axial Resistance

As requested, we have predicted the nominal (ultimate) geotechnical axial compression resistances for a 14” driven square pre-stressed concrete pile and a 16” open ended steel pipe pile under static load conditions using contributions from skin friction and end bearing assuming the average soil conditions observed at the limited widely spaced borings/CPTs. The ultimate side friction resistance of the piles was predicted using published design approaches for calculation of skin friction including the American Petroleum Institute (API) RP2AA Method with maximum adhesion limits applied based on experience in similar soil conditions. The program used estimated remolded strength of the clays in determining whether the soil in the pipe pile interior will develop sufficient friction resistance for the end bearing plug to form. The skin friction resistance from the upper 4 feet of the pile was neglected. The ultimate end bearing resistance for the piles was estimated using classic bearing capacity theory for cohesive soils and empirical correlations for estimated angle of friction and using the API RP2AA method for cohesionless soils, again with maximum end bearing resistance applied from our experience with similar conditions.

14" Driven Pre-Stressed Concrete Pile Nominal (Ultimate) Compression Resistance



16" Driven Open Steel Pipe Pile Nominal (Ultimate) Compression Resistance



The pile resistances presented in the graphs above are nominal (ultimate) geotechnical resistances and appropriate ASD factors of safety for the design allowable load should be established considering control methods specified to verify capacity at the time of driving. Provided below are the recommended factors of safety that can be considered for a typical industrial project:

Field Capacity Verification Method [1]	ASD Factor of Safety
Static Load Test on minimum 1 test pile per structure (after minimum 14-day set-up time).	2
Installation of minimum 1 test piles per structure including PDA testing at the time of installation. Dynamic Testing w/Signal Matching (PDA + CAPWAP) on a 14-day restrike of test piles, and Dynamic Testing of not less than 1% of production piles.	2
Wave Equation Analysis (WEAP), without pile dynamic measurements or load test but with field confirmation of hammer performance.	3

1. Field load verification procedures should be completed prior to finalizing order lengths for concrete and pipe piles.

The allowable tension capacity should be determined by taking 80% of the compression capacity and applying a factor of safety of 3 unless a static tension load test is performed. Note either tension or compression allowable capacity calculated with an appropriate factor of safety can be increased by 33% for maximum wind gust or other very transient load conditions, unless these transient loads have been included in the factored design load (subject to verification of allowable structural capacity).

Since these piles will derive some of their capacity from skin friction in cohesive soils, the static load testing and/or restrike for dynamic testing should be performed after allowing a minimum 14-day set-up time. The resistance obtained at end-of-driving and upon restrike may be less than the nominal predicted herein depending on the time required for set-up to occur in the cohesive soils. This consideration should be incorporated into selection of the allowable resistance and the analysis of the static load test or PDA+ CAPWAP results.

Pile top spacing is normally set to allow for typical construction tolerances in placement and vertical alignment. General practice is to have the minimum center-to-center pile top spacing either be three (3) pile diameters, or as determined by the following expression, whichever is greater:

$$SPAC = 0.05 (L1) + 0.025 (L2) + 0.0125 (L3)$$

where SPAC = Center-to-center spacing of piles, ft.
L1 = Pile penetration up to 100 ft.
L2 = Pile penetration from 101 to 200 ft.
L3 = Pile penetration beyond 201 ft.

Greater spacing than the minimum value may be required due to construction limitations, satisfy group effects, and to assure that the piles do not interfere with or intersect each other during installation. Piles installed into the very stiff to hard clays or sands below about 30 feet are not considered sensitive to group effect settlement. For large pile groups, the final design should be checked to evaluate potential for group settlement.

It should be noted that tension resistance does not account for the weight of the pile and the resistances provided herein are the geotechnical capacities of the foundation elements. The structural capacity of the piles should be checked to assure that they can safely accommodate the combined stresses that may be induced by axial and lateral loads, drag loads and overturning moments. Provisions for structural design of the foundation units have not been made and should be performed by a licensed structural engineer.

Lateral Capacity

The response of deep foundations to lateral loads is not only dependent upon the soil material's horizontal subgrade reaction, but also on the pile actual cross sectional features, effective length, stiffness, arrangement in the pile cap with respect to direction of loading, and fix-head or free-head cap interaction conditions. The analysis is usually performed to provide a lateral load that result in some limiting amount of deflection or to a specified maximum yield moment resistance of the pile. Piles subjected to lateral and moment loading should be analyzed as part of the structural detailing. Tensile and lateral load resistance of deep foundation elements should be neglected unless the piles are adequately reinforced.

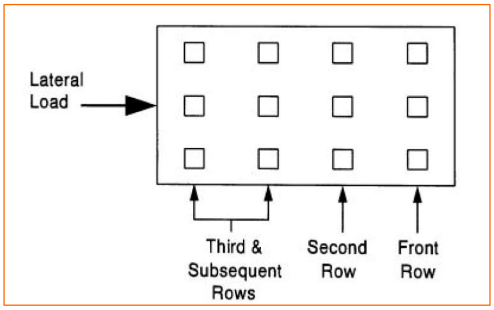
We have not performed a lateral resistance analyses as part of this scope. However, we have included soil parameters below for a lateral analysis using LPILE™ software. A detailed analysis of lateral load resistance should be performed after the actual loading conditions and pile group configurations have been determined taking into account reductions for shadowing in a pile group.

The following table lists preliminary input values for use in LPILE™ analyses. LPILE™ will estimate values of k_h and E_{50} based on undrained strength; however, non-default values of k_h should be used where provided. Since deflection or a service limit criterion will likely control lateral capacity design, no safety/resistance factor is included with the parameters.

Approximate Depth Below the Existing Grade (ft) [1]	LPILE™ Soil p-y Model	Effective Unit Weight (lb/ft^3)	Cohesion (lb/ft^2)	Internal Angle of Friction (Degrees)	Strain [2] e_{50}	Static Lateral Subgrade Modulus [2] k (lb/in^3)
Lean Clay [2] 0 – 4	Stiff Clay w/o free water	125	1,000	--	Default	Default
Lean Clay 4 – 16	Stiff Clay w/o free water	62	2,500	--	Default	Default
Lean Clay 16 – 30	Stiff Clay w/o free water	62	2,000	--	Default	Default
Fat Clay 30 – 50	Stiff Clay w/o free water	62	3,000	--	Default	Default
Fat Clay 50 – 80	Stiff Clay w/o free water	58	3,000	--	Default	Default
Silty Sand 80 – 100	Sand (Reese)	68	0	38	Default	--

1. Minimum foundation depth of 16-ft. If the foundation length is less than 16 feet, analysis for fixity is warranted.
2. The upper 4-feet should not be considered to provide full passive resistance due to potential for disturbance and desiccation effects.

When piles are used in groups, the lateral capacities of the piles in the second, third, and subsequent rows of the group should be reduced as compared to the capacity of a single, independent pile. Guidance for applying p-multiplier factors to the p values in the p-y curves for each row of pile foundations within a pile group are as follows:



- Front row: $P_m = 0.75$;
- Second row: $P_m = 0.4$
- Third and subsequent row: $P_m = 0.3$.

For the case of a single row of piles supporting a laterally loaded grade beam, group action for lateral resistance of piles would need to be considered when spacing is less than three pile diameters (measured center-to-center). However, spacing closer than 3D (where D is the diameter of the pile) is not recommended, due to potential for the installation of a new pile disturbing an adjacent installed pile, likely resulting in axial capacity reduction. It may be appropriate in some cases to design the foundations with deeper foundation caps or grade beams or utilize other means of lateral support where high lateral loads occur.

Pile Settlement, Drag Load and Down Drag

Piles installed into the stiff to very stiff overconsolidated clays below about 30 feet at the site should experience minimal settlements. Top of pile movements of about 1/2 inch are expected for the anticipated allowable design loads. These movements are associated with the loading from the structure and would be in addition to any fill-induced or down-drag settlement, where applicable. The final foundation design for large pile groups should be evaluated for group effect settlement.

Driven Pile Construction Considerations

The pre-stressed concrete piles or open steel pipe piles should be installed using a conventional external combustion or diesel hammer. The contractor should select a hammer with an energy rating capable of efficiently installing the pile but without damage. The contractor should select a driving hammer and cushion combination which can install the selected piling without overstressing the pile material.

Pile driving may become difficult below approximately 30 feet when very stiff to hard clays are encountered. Pre-boring into the upper very stiff clays may be required to achieve some depths of penetration for the piles; however, this should be evaluated during the initial test pile installations. Pre-boring diameters should be limited to not more than 80% of the pile diameter. Sand and silt layering may also locally increase driving resistance.

The driving criteria should be established at the time of construction using FHWA WEAP87 or newer version based on the characteristics of the pile driving hammer cushion assembly, the required pile capacity, the load test results, and the allowable tension and compression forces in the piles. Pile driving conditions, hammer efficiency, stress on the pile during driving and verification of the field pile capacity could be better evaluated during installation using a Pile Driving Analyzer (PDA).

Proper site preparation, construction techniques, and quality control are important for the integrity of the deep foundation system. These construction efforts should be monitored and documented by the geotechnical engineer’s representative. Each pile should be observed and checked for buckling, cracking, and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations.

SEISMIC CONSIDERATIONS

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7-10.

Description	Value
International Building Code Site Classification (IBC) [1]	D [2]
Site Latitude	30.1577 ° N
Site Longitude	-93.5575 ° W

1. Seismic site classification in general accordance with the *2015 International Building Code*, which refers to ASCE 7-10.
2. The 2015 International Building Code (IBC) uses a site profile extending to a depth of 100 feet for seismic site classification. Borings at this site were extended to a maximum depth of 100 feet.

FLOOR SLABS

Design parameters for floor slabs assume adequate earthwork procedures have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the aggregate base beneath the floor slab.

Floor Slab Design Parameters

Item	Description
Floor Slab Support [1]	A course of 4-6 inches of free-draining (less than 5% passing the U.S. No. 200 sieve) sand compacted to at least 95% of ASTM D 698 [2] over compacted engineering fill.
Estimated Modulus of Subgrade Reaction [2]	100 pounds per square inch per inch (psi/in) for point loads.

1. Free-draining granular material should have less than 5 percent fines (material passing the #200 sieve). Other design considerations such as cold temperatures and condensation development could warrant more extensive design provisions.
2. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in **Earthwork**, and the floor slab support as noted in this table. It is

Item	Description
	provided for point loads. For large area loads the modulus of subgrade reaction would be substantially lower.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

Floor Slab Construction Considerations

Finished subgrade within and for at least 10 feet beyond the floor slab should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of floor slabs, the affected material should be removed and structural fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course.

The Geotechnical Engineer should approve the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

PAVEMENTS

General Pavement Comments

Pavement designs are provided for the traffic conditions and pavement life conditions as noted in **Project Description** and in the following sections of this report. A critical aspect of pavement performance is site preparation. Pavement designs, noted in this section, must be applied to the site, which has been prepared as recommended in the **Earthwork** section.

Support characteristics of subgrade for pavement design do not account for shrink/swell movements of an expansive clay subgrade, such as soils encountered on this project. Thus, the pavement may be adequate from a structural standpoint, yet still experience cracking and deformation due to shrink/swell related movement of the subgrade.

Pavement Design Parameters

Designs for minimum thicknesses for new pavement sections for this project have been based on the procedures outlined in the 1993 Guideline for Design of Pavement Structures by the American Association of State Highway and Transportation Officials (AASHTO-1993).

A subgrade CBR of 4 and a modulus of subgrade reaction of 200 pci was used for the PCC pavement designs. The values were empirically derived based upon our experience with the lean clay subgrade soils and our understanding of the quality of the subgrade as prescribed by the **Site Preparation** conditions as outlined in **Earthwork**.

Pavement Section Thicknesses

The following table provides options for PCC Sections:

Portland Cement Concrete Design		
Layer	Minimum Thickness (inches)	
	Industrial Driveways	Industrial Entrances, Exits
PCC [1]	8	10
Aggregate Base [2,3]	4	4

1. 4,000 psi at 28 days, 4-inch maximum slump and 5 to 7 percent air entrained. PCC pavements are recommended for trash container pads and in any other areas subjected to heavy wheel loads and/or turning traffic.
2. Aggregate base course should be a No. 610 limestone or similarly graded recycled concrete compacted to 100% of its max dry density as determined by ASTM D-698, Standard Proctor Test with stability present.
3. The aggregate base will serve to protect the subgrade, reduce pumping of fines, and reduce shrink/swell affects for the concrete pavement applications.

Pavement Drainage

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase.

Based on the possibility of shallow and/or perched groundwater, we recommend installing a pavement subdrain system to control groundwater, improve stability, and improve long term pavement performance.

The pavement surfacing and adjacent sidewalks should be sloped to provide rapid drainage of surface water. Water should not be allowed to pond on or adjacent to these grade supported slabs, since this could saturate the subgrade and contribute to premature pavement or slab deterioration.

Pavement Maintenance

The pavement sections represent minimum recommended thicknesses and, as such, periodic maintenance should be anticipated. Therefore, preventive maintenance should be planned and provided for through an on-going pavement management program. Maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Maintenance consists of both localized maintenance (e.g. crack and joint sealing and patching) and global maintenance (e.g. surface sealing). Preventive maintenance is usually the priority when implementing a pavement maintenance program. Additional engineering observation is recommended to determine the type and extent of a cost-effective program. Even with periodic maintenance, some movements and related cracking may still occur and repairs may be required.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the geotechnical conditions in the area, and data obtained from widely spaced borings from our site exploration and from our limited understanding of the project. Variations will occur between exploration point locations, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer to develop a scope of a final geotechnical evaluation once the project becomes more defined. Furthermore, given the limitations described above based on the preliminary nature of this report, all parties are advised that any decisions or actions taken by any party based on the information contained herein, including decisions with financial implications are done solely at the risk of that party. By providing this information in this preliminary form, Terracon expressly disclaims any duties or obligations associated with the usage of this information for decision-making purposes.

Our scope of services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third party beneficiaries intended. Any third party access to services or correspondence is solely for information purposes only.

Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

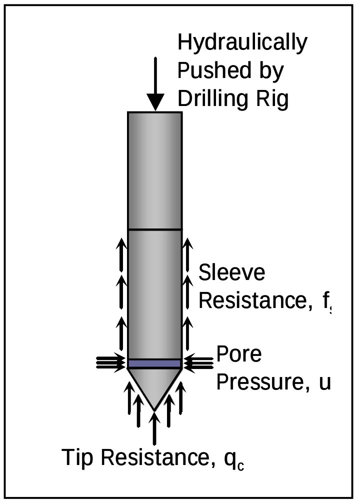
Number of Locations	Type of Exploration	Planned Depth (feet) [1, 2]	Planned Location
3	Borings	20 feet	Adjacent to CPTs
2	CPTs	50 feet	Site
1	CPT	100 feet or refusal	Site
1	Boring	100 feet	Site

1. Below ground surface
2. 100-foot boring (B-01-A) was added due to 100-foot CPT refusal.

Boring Layout and Elevations: Unless otherwise noted, Terracon personnel provide the boring layout. Coordinates are obtained with a handheld GPS unit (estimated horizontal accuracy of about ±10 feet) and approximate elevations are obtained by interpolation from Google Earth™ imagery. If elevations and a more precise boring layout are desired, we recommend borings be surveyed following completion of fieldwork.

Subsurface Exploration Procedures: We advanced the borings with track-mounted and ATV-mounted rotary drill rigs using continuous flight augers (solid stem). Five samples are continuously obtained in the upper 10 feet of each boring and at maximum intervals of 5 feet thereafter. In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge is pushed hydraulically into the soil to obtain a relatively undisturbed sample. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon is driven into the ground by a 140-pound safety hammer falling 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. We observe and record groundwater levels during drilling and sampling. We observe and record groundwater levels during drilling and sampling. For safety purposes, all borings are backfilled with auger cuttings or cement-bentonite grout, consistent with state regulations after their completion.

At each designated location, a CPT test was performed by pushing a 10-square centimeter electric cone penetrometer at an approximate rate of 20 millimeters/second using the hydraulic cylinders of the drilling rig. The cone penetrometer is equipped with electronic load cells to measure tip resistance and sleeve resistance, and a pressure transducer is measure the generated ambient pore pressure, as illustrated in the insert diagram.



Digital data representing the tip resistance, the sleeve penetration, the pore pressure and the CPT sounding inclination are typically measured at 50 mm intervals during penetration using a CPT data acquisition system or logger. These data are transferred to an on-site computer using a cable transmission system. This process allowed continuous monitoring of the data as the cone is advanced in a real-time fashion.

Upon completion of the test, the data collected were downloaded directly from the CPT data logger to an on-site computer. The collected data were then interpreted using a software package provided by the cone manufacture to provide the cone and sleeve resistance, pore pressure and inclination. The software also allows interpretation of soil types (clay, silt, sand, etc.), soil unit weight, and selected soil parameters, such as undrained shear strength, overconsolidation ratio, and equivalent standard penetration resistance. The conventional field data from the soil boring and the available laboratory test results can also correlate with the interpreted CPT data for a particular site. The testing and calibration of the CPT device was conducted in general conformance with ASTM D 5778.

The sampling depths, penetration distances, and other sampling information are recorded on the field boring logs. The samples are placed in appropriate containers and taken to our soil laboratory for testing and classification by a geotechnical engineer. Our exploration team prepares field boring logs as part of the drilling operations. These field logs include visual classifications of the materials encountered during drilling and our interpretation of the subsurface conditions between samples. Final boring logs are prepared from the field logs. The final boring logs represent the geotechnical engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

Laboratory Testing

The project engineer reviews the field data and assigns various laboratory tests to better understand the engineering properties of the various soil strata as necessary for this project. Procedural standards noted below are for reference to methodology in general. In some cases, variations to methods are applied because of local practice or professional judgment. Standards noted below include reference to other, related standards. Such references are not necessarily applicable to describe the specific test performed.

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D422 Standard Test Method for Particle-Size Analysis of Soils
- ASTM D2166/D2166M Standard Test Method for Unconfined Compressive Strength of Cohesive Soil

The laboratory testing program often includes examination of soil samples by an engineer. Based on the material's texture and plasticity, we describe and classify the soil samples in accordance with the Unified Soil Classification System.

SITE LOCATION AND EXPLORATION PLANS

SITE LOCATION

Port of Vinton ■ Vinton, LA

April 9, 2018 ■ Terracon Project No. EH175369

Terracon GeoReport

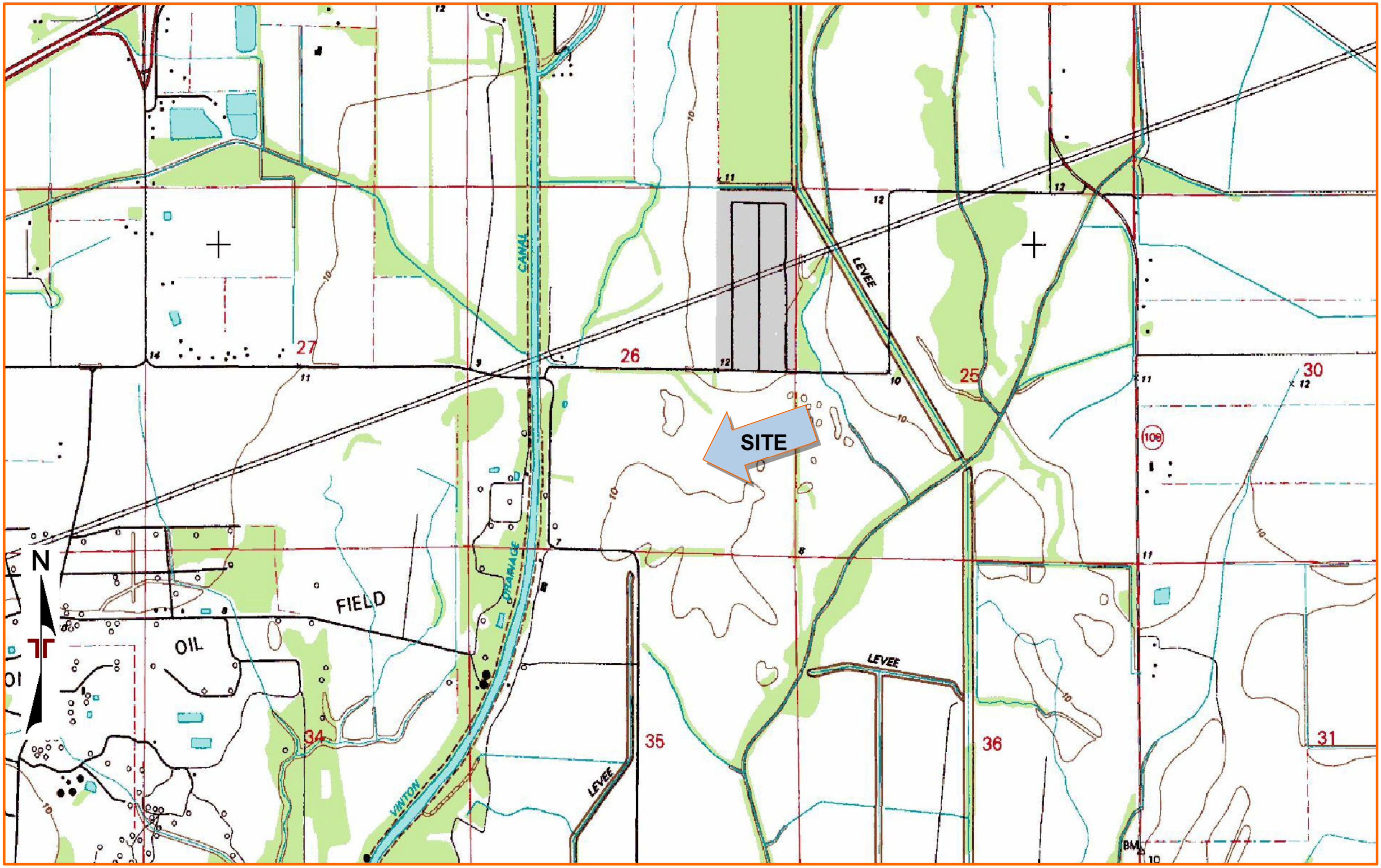


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

TOPOGRAPHIC MAP IMAGE COURTESY OF THE U.S. GEOLOGICAL SURVEY QUADRANGLES INCLUDE: VINTON, LA (1/1/1994).

EXPLORATION PLAN

Port of Vinton ■ Vinton, LA

May 2, 2018 ■ Terracon Project No. EH175369

Terracon GeoReport

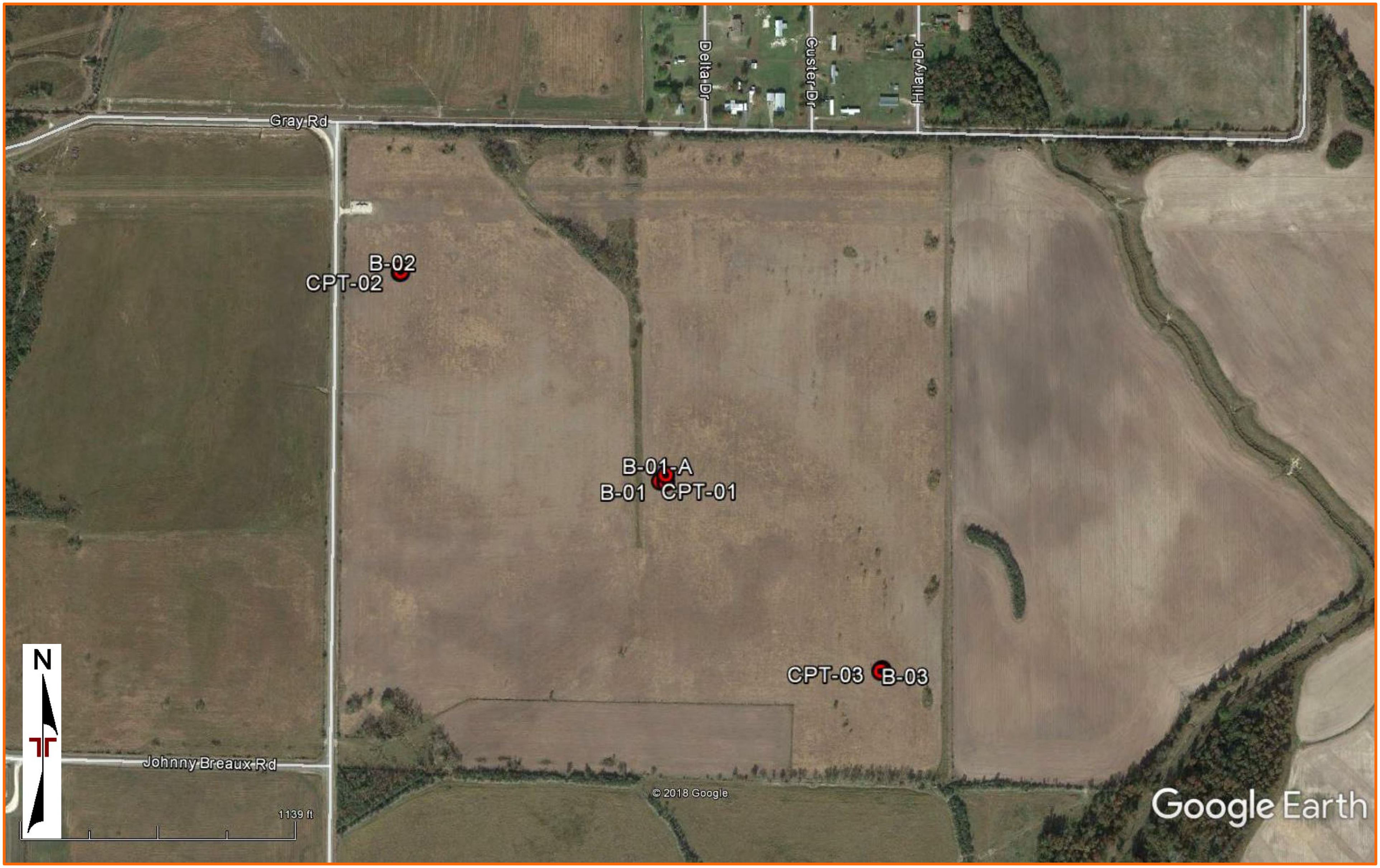


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

AERIAL PHOTOGRAPHY PROVIDED BY GOOGLE EARTH

EXPLORATION RESULTS

BORING LOG NO. B-01-A

PROJECT: Port of Vinton

**CLIENT: SWLA Economic Development Alliance
Lake Charles, LA**

**SITE: Gray Road and Johnny Breaux Road
Vinton, LA**

LOCATION See Exploration Plan

Latitude: 30.1577° Longitude: -93.5575°

Approximate Surface Elev: 9 (Ft.) +/-

DEPTH	Material Description	ELEVATION (Ft.)
0.3	**3" TOPSOIL**	8.5+/-
8.0	**SILTY CLAY (CL-ML)**, dark gray and brown, medium stiff to stiff, with sand and roots	1+/-
18.0	**LEAN CLAY (CL)**, gray and tan, medium stiff, with sand and ferrous nodules	-9+/-
30.0	**FAT CLAY (CH)**, gray and brown, stiff to very stiff, slickensided, with ferrous nodules, calcareous nodules, and silt seams - failure at low strain at 18'	-21+/-
	FAT CLAY (CH), tan and gray, very stiff to hard, slickensided, with silty sand pockets and calcareous nodules - failure at low strain at 38'	

DEPTH (Ft.)	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0–2	2.00 (HP)				17	108	21-17-4	
8–10	1.00 (HP)	UC	0.63	11.5	27	98		
18–20	2.50 (HP)	UC	1.30	3.8	33	88	61-22-39	
28–30	2.00 (HP)							
38–40	4.25 (HP)	UC	0.64	1.5	25	101	58-20-38	
48–50	3.25 (HP)							

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
0'-20' continuous flight auger, 20'-99.5' rotary wash

Abandonment Method:
Boring backfilled with auger cuttings and cement-bentonite grout upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation based on Google Earth imagery.

Notes:

WATER LEVEL OBSERVATIONS

- ▽ *Groundwater first encountered.*
- ▽ *Water level after 15 minutes*

Terracon
2822 Oneal Ln Bldg B
Baton Rouge, LA

Boring Started: 04-18-2018	Boring Completed: 04-18-2018
Drill Rig: GP #891	Driller: G. Whitmire
Project No.: EH175369	

BORING LOG NO. B-01-A

PROJECT: Port of Vinton

CLIENT: SWLA Economic Development Alliance Lake Charles, LA

SITE: Gray Road and Johnny Breaux Road Vinton, LA

LOCATION See Exploration Plan

Latitude: 30.1577° Longitude: -93.5575°

Approximate Surface Elev: 9 (Ft.) +/-

DEPTH	Description	ELEVATION (Ft.)	DEPTH (Ft.)	SAMPLE TYPE	FIELD TEST RESULTS	TEST TYPE	COMPRESSIVE STRENGTH (tsf)	STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	FAT CLAY (CH), tan and gray, very stiff to hard, slickensided, with silty sand pockets and calcareous nodules *(continued)*											
58.0		-49+/-										
	FAT CLAY (CH), gray, stiff to very stiff, slickensided, with silty sand pockets, trace organics - failure at low strain at 58'		58-60	Shelby tube	2.50 (HP)	UC	1.32	2.6	37	84	72-24-48	
68.0		-59+/-										
	FAT CLAY (CH), black and gray, stiff to very stiff, with silty sand pockets		68-70	Shelby tube	3.00 (HP)				25	100		
78.0		-69+/-										
	SILT WITH SAND (ML), gray, hard		78-79.5	Split spoon	22-38-50 N=88				19			79
88.0		-79+/-										
	SILTY SAND (SM), gray, very dense		88-89.5	Split spoon	15-38-50 N=88				19			47
98.0		-89+/-										
	SILTY CLAY (CL-ML), tan, black, and gray, hard, with sand		98-99.5	Split spoon	15-22-23 N=45				22		26-19-7	
99.5		-90.5+/-										

Boring Terminated at 99.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
0'-20' continuous flight auger, 20'-99.5' rotary wash

Abandonment Method:
Boring backfilled with auger cuttings and cement-bentonite grout upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation based on Google Earth imagery.

Notes:

WATER LEVEL OBSERVATIONS

- ▽ *Groundwater first encountered.*
- ▽ *Water level after 15 minutes*

Terracon
2822 Oneal Ln Bldg B
Baton Rouge, LA

Boring Started: 04-18-2018	Boring Completed: 04-18-2018
Drill Rig: GP #891	Driller: G. Whitmire
Project No.: EH175369	

BORING LOG NO. B-01

PROJECT: Port of Vinton

CLIENT: SWLA Economic Development Alliance
Lake Charles, LA

SITE: Gray Road and Johnny Breaux Road
Vinton, LA

LOCATION See Exploration Plan

Latitude: 30.1577° Longitude: -93.5575°

Approximate Surface Elev: 9 (Ft.) +/-

Depth	Material Description	Elevation (Ft.)
0.3	**4" TOPSOIL**	8.5+/-
2.0	**LEAN CLAY (CL)**, dark gray and brown, stiff, trace roots	7+/-
6.0	**LEAN CLAY (CL)**, gray, tan, and brown, medium stiff, trace sand	3+/-
11.0	**SANDY LEAN CLAY (CL)**, light gray and brown, stiff	-2+/-
16.0	**CLAYEY SAND (SC)**, tan and brown, medium dense	-7+/-
20.0	**FAT CLAY (CH)**, brown and gray, stiff to very stiff	-11+/-
	Boring Terminated at 20 Feet	

Depth (Ft.)	Water Level Observations	Test Type	Compressive Strength (tsf)	Strain (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
0-2		UC	1.20	9	18	109	27-16-11	
2-4					24			
4-6	Water level after 15 minutes (approx. 5 ft)	UC	0.61	7	22	101		
6-8					19			56
11-13	Groundwater first encountered (approx. 13 ft)				22			49
16-18					34			

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
0'-20' continuous flight auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation based on Google Earth imagery.

Notes:

WATER LEVEL OBSERVATIONS

- *Groundwater first encountered*
- *Water level after 15 minutes*

Terracon
2822 Oneal Ln Bldg B
Baton Rouge, LA

Boring Started: 04-12-2018	Boring Completed: 04-12-2018
Drill Rig: GP #891	Driller: G. Triplette
Project No.: EH175369	

BORING LOG NO. B-02

PROJECT: Port of Vinton

CLIENT: SWLA Economic Development Alliance
Lake Charles, LA

SITE: Gray Road and Johnny Breaux Road
Vinton, LA

LOCATION See Exploration Plan

Latitude: 30.16° Longitude: -93.561°

Approximate Surface Elev: 9 (Ft.) +/-

DEPTH		ELEVATION (Ft.)
0.3	**4" TOPSOIL**	8.5+/-
4.0	**LEAN CLAY (CL)**, dark gray and brown, stiff	5+/-
11.0	**LEAN CLAY (CL)**, light gray and tan, medium stiff to stiff - with ferrous nodules below 11 feet	-2+/-
20.0	**LEAN CLAY (CL)**, light gray, tan, and brown, stiff to very stiff	-11+/-
	Boring Terminated at 20 Feet	

DEPTH (Ft.)	TEST TYPE	COMPRESSIVE STRENGTH (tsf)	STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0-2	UC	1.03	11	20	102		
2-4				20		38-17-21	
4-6	UC	1.33	15	21	106		
8-10	UC	0.91	13.3	25	101	31-21-10	
14-16				27		34-22-12	

Water level observations shown at approximately 5 ft and 10 ft.

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
0'-20' continuous flight auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation based on Google Earth imagery.

Notes:

WATER LEVEL OBSERVATIONS
- *Groundwater first encountered*
- *Water level after 15 minutes*

Terracon
2822 Oneal Ln Bldg B
Baton Rouge, LA

Boring Started: 04-12-2018	Boring Completed: 04-12-2018
Drill Rig: GP #891	Driller: G. Triplette
Project No.: EH175369	

BORING LOG NO. B-03

PROJECT: Port of Vinton

CLIENT: SWLA Economic Development Alliance
Lake Charles, LA

SITE: Gray Road and Johnny Breaux Road
Vinton, LA

LOCATION See Exploration Plan

Latitude: 30.1555° Longitude: -93.5547°

Approximate Surface Elev: 8 (Ft.) +/-

DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)
0.3	**4" TOPSOIL**	7.5+/-
2.0	**SILT (ML)**, dark gray and brown, soft to medium stiff, with sand and roots - failure at low strain at 0'	6+/-
6.0	**LEAN CLAY (CL)**, light gray and brown, medium stiff to stiff - failure at low strain at 4'	2+/-
8.0	**CLAYEY SAND (SC)**, light gray and brown, loose to medium dense	0+/-
11.0	**CLAYEY SAND (SC)**, tan and brown, medium dense	-3+/-
16.0	**CLAYEY SAND (SC)**, tan and brown, loose to medium dense, with clay layers	-8+/-
20.0	**LEAN CLAY/FAT CLAY (CL/CH)**, tan, stiff to very stiff	-12+/-
	Boring Terminated at 20 Feet	

DEPTH (Ft.)	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0–2	UC	0.32	4.2	18	103	NP	
2–4				20			
4–6	UC	0.58	4.6	22	104		
6–8				19			35
14–16				25			23

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
0'-20' continuous flight auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation based on Google Earth imagery.

Notes:

WATER LEVEL OBSERVATIONS

- *Groundwater first encountered*
- *Water level after 15 minutes*

Terracon
2822 Oneal Ln Bldg B
Baton Rouge, LA

Boring Started: 04-12-2018	Boring Completed: 04-12-2018
Drill Rig: GP #891	Driller: G. Triplette
Project No.: EH175369	

CPT LOG NO. CPT-01

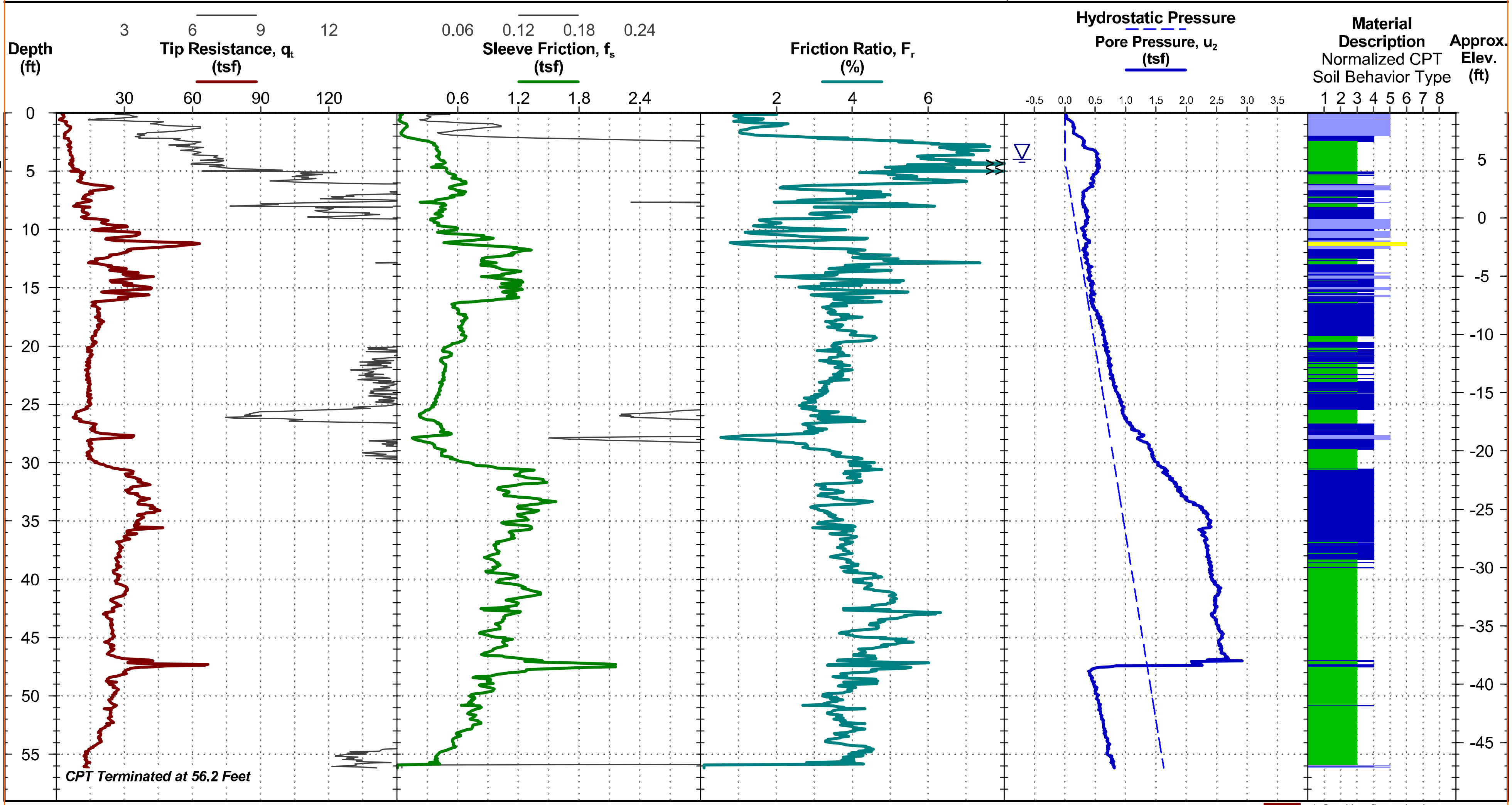
PROJECT: Port of Vinton

CLIENT: SWLA Economic Development Alliance
Lake Charles, LA

TEST LOCATION: See Exploration Plan

Approx. Surface Elev: 9 ft +/-
Latitude: 30.15767°
Longitude: -93.55757°

SITE: Gray Road and Johnny Breaux Road
Vinton, LA



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevation based on Google Earth imagery.

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

4 ft estimated water depth
(used in normalizations and correlations; See Supporting Information)

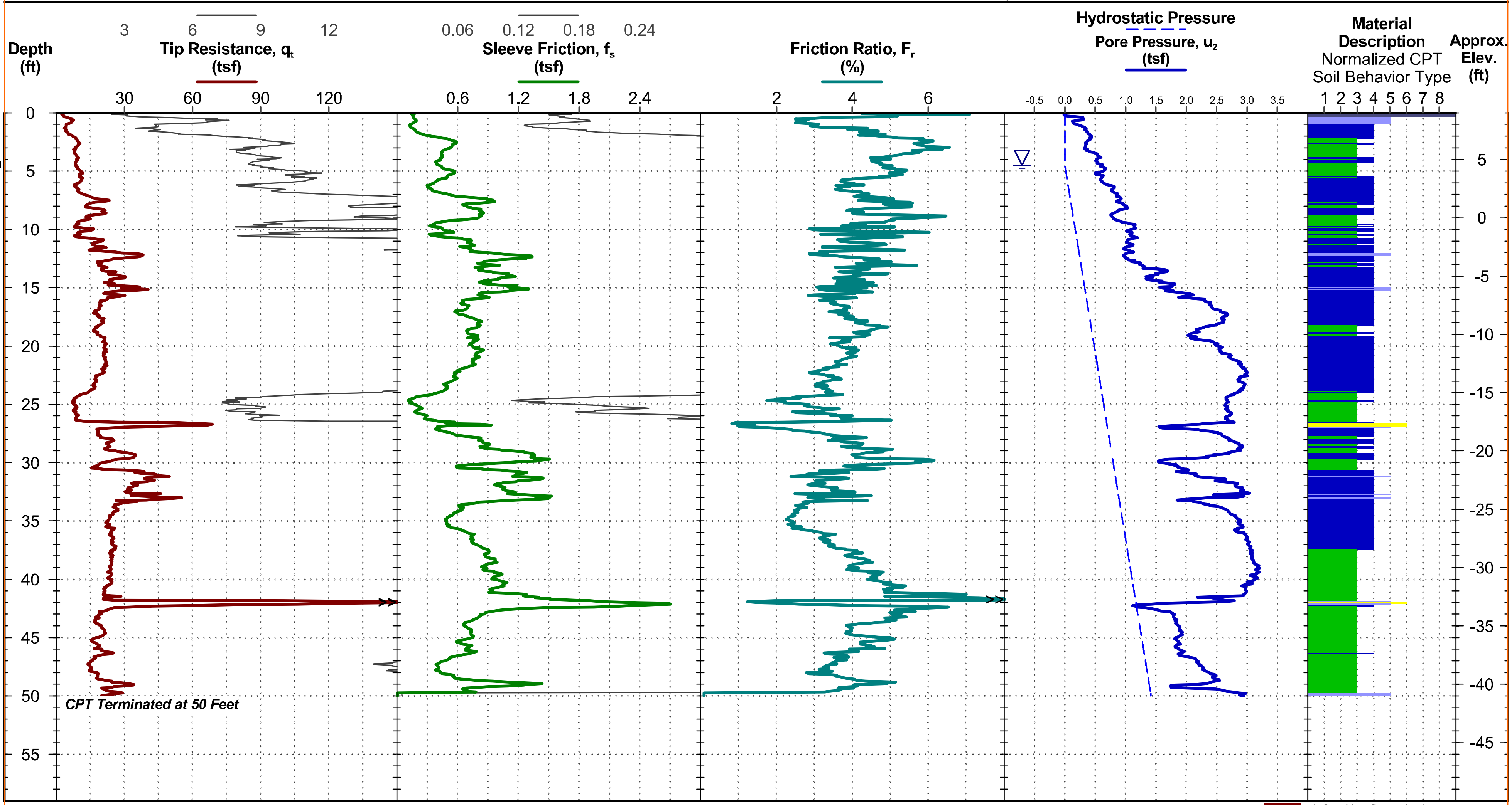
Probe no. DDG1377 with net area ratio of .8
U2 pore pressure transducer location
Manufactured by Vertek; calibrated 1/16/2018
Tip and sleeve areas of 10 cm² and 150 cm²
Ring friction reducer with O.D. of 1.915 in

Terracon
2822 Oneal Ln Bldg B
Baton Rouge, LA

CPT Started: 4/11/2018	CPT Completed: 4/11/2018
Rig: GP #891	Operator: G. Triplette
Project No.: EH175369	

CPT LOG NO. CPT-02

PROJECT: Port of Vinton	**CLIENT:** SWLA Economic Development Alliance Lake Charles, LA	**TEST LOCATION:** See Exploration Plan Approx. Surface Elev: 9 ft +/- Latitude: 30.16003° Longitude: -93.56098°
SITE: Gray Road and Johnny Breaux Road Vinton, LA		



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevation based on Google Earth imagery.

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

∇ 4.5 ft estimated water depth
(used in normalizations and correlations; See Supporting Information)

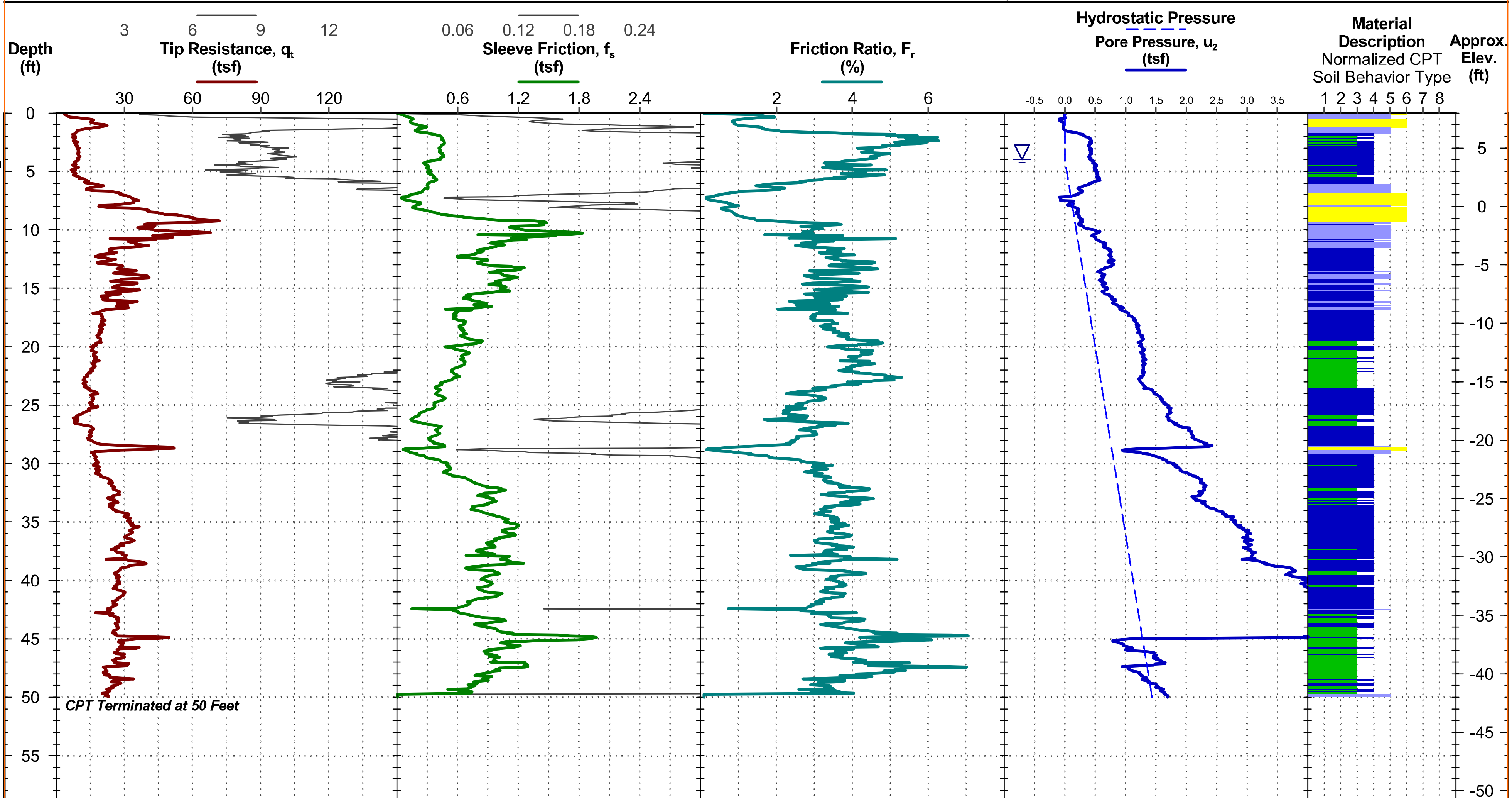
Probe no. DDG1377 with net area ratio of .8
U2 pore pressure transducer location
Manufactured by Vertek; calibrated 1/16/2018
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.915 in

Terracon
2822 Oneal Ln Bldg B
Baton Rouge, LA

CPT Started: 4/12/2018	CPT Completed: 4/12/2018
Rig: GP #891	Operator: G. Triplette
Project No.: EH175369	

CPT LOG NO. CPT-03

PROJECT: Port of Vinton	CLIENT: SWLA Economic Development Alliance Lake Charles, LA	TEST LOCATION: See Exploration Plan
SITE: Gray Road and Johnny Breaux Road Vinton, LA		Approx. Surface Elev: 8 ft +/- Latitude: 30.15554° Longitude: -93.55467°



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevation based on Google Earth imagery.

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

∇ 4 ft estimated water depth
(used in normalizations and correlations; See Supporting Information)

Probe no. DDG1377 with net area ratio of .8
U2 pore pressure transducer location
Manufactured by Vertek; calibrated 1/16/2018
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.915 in

Terracon
2822 Oneal Ln Bldg B
Baton Rouge, LA

CPT Started: 4/12/2018	CPT Completed: 4/12/2018
Rig: GP #891	Operator: G. Triplette
Project No.: EH175369	

SUPPORTING INFORMATION

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Port of Vinton ■ Vinton, LA

5/2/2018 ■ Terracon Project No. EH175369

Terracon GeoReport

SAMPLING

Auger Cuttings

Shelby Tube

Standard Penetration Test

WATER LEVEL

Water Initially Encountered

Water Level After a Specified Period of Time

Water Level After a Specified Period of Time

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

FIELD TESTS

N	**Standard Penetration Test Resistance (Blows/Ft.)**
(HP)	**Hand Penetrometer**
(T)	**Torvane**
(DCP)	**Dynamic Cone Penetrometer**
UC	**Unconfined Compressive Strength**
(PID)	**Photo-Ionization Detector**
(OVA)	**Organic Vapor Analyzer**

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength Qu, (tsf)**	**Standard Penetration or N-Value Blows/Ft.**
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

RELATIVE PROPORTIONS OF SAND AND GRAVEL		RELATIVE PROPORTIONS OF FINES	
Descriptive Term(s) of other constituents	Percent of Dry Weight	**Descriptive Term(s) of other constituents**	**Percent of Dry Weight**
Trace	<15	Trace	<5
With	15-29	With	5-12
Modifier	>30	Modifier	>12

GRAIN SIZE TERMINOLOGY		PLASTICITY DESCRIPTION	
Major Component of Sample	**Particle Size**	**Term**	**Plasticity Index**
Boulders	Over 12 in. (300 mm)	Non-plastic	0
Cobbles	12 in. to 3 in. (300mm to 75mm)	Low	1 - 10
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)	Medium	11 - 30
Sand	#4 to #200 sieve (4.75mm to 0.075mm	High	> 30
Silt or Clay	Passing #200 sieve (0.075mm)		

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]					Soil Classification: Group Symbol	Soil Classification: Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu ³ 4 and 1 £ Cc £ 3 [E]		GW	Well-graded gravel [F]
			Cu < 4 and/or 1 > Cc > 3 [E]		GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH		GM	Silty gravel [F,G,H]
			Fines classify as CL or CH		GC	Clayey gravel [F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu ³ 6 and 1 £ Cc £ 3 [E]		SW	Well-graded sand [I]
			Cu < 6 and/or 1 > Cc > 3 [E]		SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH		SM	Silty sand [G,H,I]
			Fines classify as CL or CH		SC	Clayey sand [G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A"		CL	Lean clay [K,L,M]
			PI < 4 or plots below "A" line [J]		ML	Silt [K,L,M]
		Organic:	Liquid limit - oven dried	< 0.75	OL	Organic clay [K,L,M,N]
			Liquid limit - not dried			Organic silt [K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line		CH	Fat clay [K,L,M]
			PI plots below "A" line		MH	Elastic Silt [K,L,M]
		Organic:	Liquid limit - oven dried	< 0.75	OH	Organic clay [K,L,M,P]
			Liquid limit - not dried			Organic silt [K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor				PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ³ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ³ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ³ 30% plus No. 200 predominantly sand, add "sandy" to group name.

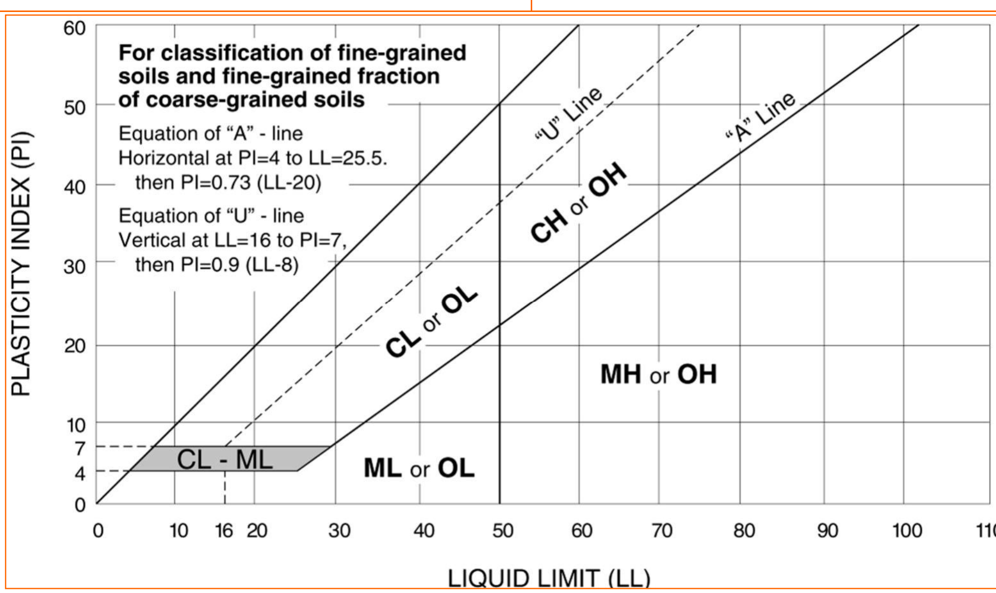
[M] If soil contains ³ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ³ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



CPT GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Port of Vinton ■ Vinton, LA

5/2/2018 ■ Terracon Project No. EH175369

Terracon GeoReport

DESCRIPTION OF GEOTECHNICAL CORRELATIONS

DESCRIPTION OF MEASUREMENTS AND CALIBRATIONS

To be reported per ASTM D5778:

Uncorrected Tip Resistance, q_c
Measured force acting on the cone divided by the cone's projected area

Corrected Tip Resistance, q_t
Cone resistance corrected for porewater and net area ratio effects
$q_t = q_c + u_2(1 - a)$
Where a is the net area ratio, a lab calibration of the cone typically between 0.70 and 0.85

Pore Pressure, u
Pore pressure measured during penetration
u_1 - sensor on the face of the cone
u_2 - sensor on the shoulder (more common)

Sleeve Friction, f_s
Frictional force acting on the sleeve divided by its surface area

Normalized Friction Ratio, F_r
The ratio as a percentage of f_s to q_t, accounting for overburden pressure

To be reported per ASTM D7400, if collected:

Shear Wave Velocity, V_s
Measured in a Seismic CPT and provides direct measure of soil stiffness

Normalized Tip Resistance, Q_{tn}
$Q_{tn} = ((q_t - \sigma_{V0})/P_a)(P_a/\sigma'_{V0})^n$
$n = 0.381(I_c) + 0.05(\sigma'_{V0}/P_a) - 0.15$

Over Consolidation Ratio, OCR
$OCR\ (1) = 0.25(Q_{tn})^{1.25}$
$OCR\ (2) = 0.33(Q_{tn})$

Undrained Shear Strength, S_u
$S_u = Q_{tn} \times \sigma'_{V0}/N_{kt}$
N_{kt} is a soil-specific factor (shown on S_u plot)

Sensitivity, S_t
$S_t = (q_t - \sigma_{V0}/N_{kt}) \times (1/f_s)$

Effective Friction Angle, ϕ'
$\phi'\ (1) = \tan^{-1}(0.373[\log(q_t/\sigma'_{V0}) + 0.29])$
$\phi'\ (2) = 17.6 + 11[\log(Q_{tn})]$

Unit Weight, γ
$\gamma = (0.27[\log(F_r)]+0.36[\log(q_t/atm)]+1.236) \times \gamma_{water}$
σ_{V0} is taken as the incremental sum of the unit weights

Small Strain Shear Modulus, G_0
$G_0\ (1) = \rho V_s^2$
$G_0\ (2) = 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$

Soil Behavior Type Index, I_c
$I_c = [(3.47 - \log(Q_{tn}))^2 + (\log(F_r) + 1.22)^2]^{0.5}$

SPT N_{60}
$N_{60} = (q_t/atm) / 10^{(1.1268 - 0.2817Ic)}$

Elastic Modulus, E_s (assumes $q/q_{ultimate}$ ~ 0.3, i.e. FS = 3)
$E_s\ (1) = 2.6\Psi G_0$ where $\Psi = 0.56 - 0.33\log Q_{tn,clean\ sand}$
$E_s\ (2) = G_0$
$E_s\ (3) = 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$
$E_s\ (4) = 2.5q_t$

Constrained Modulus, M
$M = \alpha_M(q_t - \sigma_{V0})$
For $I_c > 2.2$ (fine-grained soils)
$\alpha_M = Q_{tn}$ with maximum of 14
For $I_c < 2.2$ (coarse-grained soils)
$\alpha_M = 0.0188 \times 10^{(0.55Ic + 1.68)}$

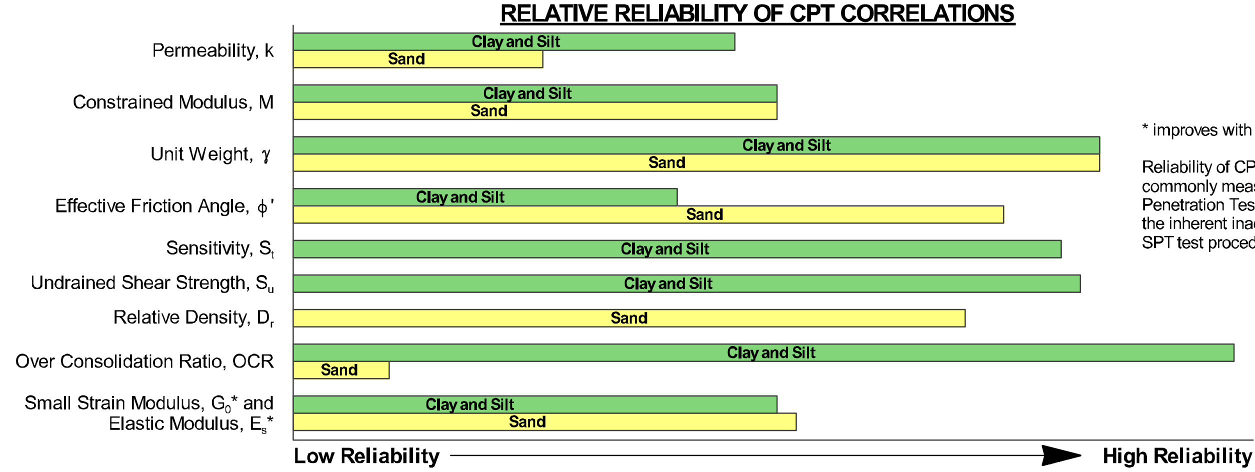
Hydraulic Conductivity, k
For $1.0 < I_c < 3.27$ $k = 10^{(0.952 - 3.04Ic)}$
For $3.27 < I_c < 4.0$ $k = 10^{(-4.52 - 1.37Ic)}$

Relative Density, D_r
$D_r = (Q_{tn} / 350)^{0.5} \times 100$

REPORTED PARAMETERS

CPT logs as provided, at a minimum, report the data as required by ASTM D5778 and ASTM D7400 (if applicable). This minimum data include q_t, f_s, and u. Other correlated parameters may also be provided. These other correlated parameters are interpretations of the measured data based upon published and reliable references, but they do not necessarily represent the actual values that would be derived from direct testing to determine the various parameters. To this end, more than one correlation to a given parameter may be provided. The following chart illustrates estimates of reliability associated with correlated parameters based upon the literature referenced below.

RELATIVE RELIABILITY OF CPT CORRELATIONS



* improves with seismic V_s measurements

Reliability of CPT-predicted N_{60} values as commonly measured by the Standard Penetration Test (SPT) is not provided due to the inherent inaccuracy associated with the SPT test procedure.

WATER LEVEL

The groundwater level at the CPT location is used to normalize the measurements for vertical overburden pressures and as a result influences the normalized soil behavior type classification and correlated soil parameters. The water level may either be "measured" or "estimated:"

Measured - Depth to water directly measured in the field

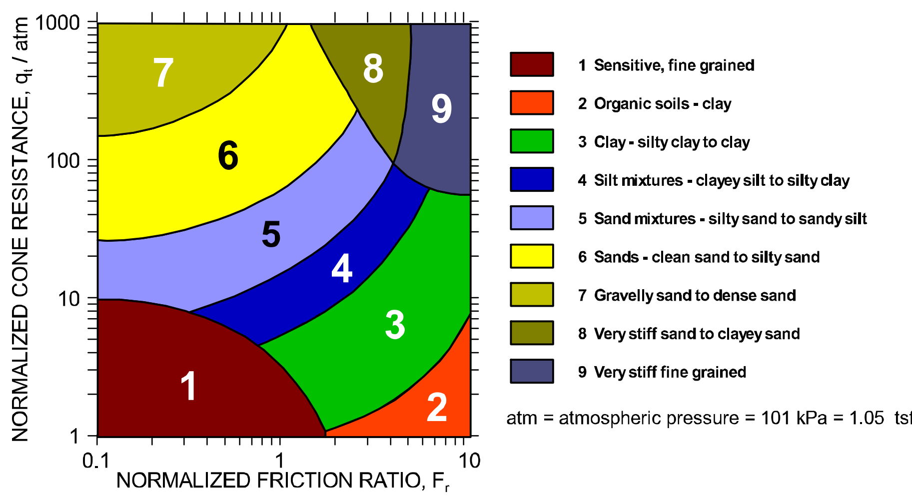
Estimated - Depth to water interpolated by the practitioner using pore pressure measurements in coarse grained soils and known site conditions

While groundwater levels displayed as "measured" more accurately represent site conditions at the time of testing than those "estimated," in either case the groundwater should be further defined prior to construction as groundwater level variations will occur over time.

CONE PENETRATION SOIL BEHAVIOR TYPE

The estimated stratigraphic profiles included in the CPT logs are based on relationships between corrected tip resistance (q_t), friction resistance (f_s), and porewater pressure (u_2). The normalized friction ratio (F_r) is used to classify the soil behavior type.

Typically, silts and clays have high F_r values and generate large excess penetration porewater pressures; sands have lower F_r's and do not generate excess penetration porewater pressures. The adjacent graph (Robertson *et al.*) presents the soil behavior type correlation used for the logs. This normalized SBT chart, generally considered the most reliable, does not use pore pressure to determine SBT due to its lack of repeatability in onshore CPTs.



atm = atmospheric pressure = 101 kPa = 1.05 tsf

REFERENCES

Kulhawy, F.H., Mayne, P.W., (1997). "Manual on Estimating Soil Properties for Foundation Design," Electric Power Research Institute, Palo Alto, CA.
Mayne, P.W., (2013). "Geotechnical Site Exploration in the Year 2013," Georgia Institue of Technology, Atlanta, GA.
Robertson, P.K., Cabal, K.L. (2012). "Guide to Cone Penetration Testing for Geotechnical Engineering," Signal Hill, CA.
Schmertmann, J.H., (1970). "Static Cone to Compute Static Settlement over Sand," *Journal of the Soil Mechanics and Foundations Division*, 96(SM3), 1011-1043.