ADDENDUM NO. 1

to the

CONTRACT DOCUMENTS
FOR

SEADRIFT DRAINAGE IMPROVEMENT PROJECT
GLO Contract No. 22-085-014-D245

FOR

CALHOUN COUNTY , TEXAS

April 25, 2024

Prepared by:

G&W Engineers, Inc.
205 West Live Oak
Port Lavaca, Texas 77979
(361) 552-4509

Approved By:

STATE OF TEXAS
SCOTT P. MASON
127893
LICENSED
PROFESSIONAL ENGINEER

04/25/24

Scott P. Mason, P.E.
Texas Serial No. 127893

Date:

G & W Engineers, Inc.
Texas Registered Engineering Firm F-04188
Project No. 5310.013a

Clarifications to the original Contract Documents, Contract Drawings and/or Specifications have been deemed necessary, and in certain cases, revisions to the original Contract Documents, Contract Drawings and/or Specifications are required. If discrepancies and/or inconsistencies exist between these specified revisions and the original Contract Documents, Contract Drawings and/or Specifications, said Addendum No. 1 shall govern.

CONTRACT DOCUMENTS AND DRAWINGS:

1. Pre-Bid Meeting attendance sheet

As requested on CivCastUSA, the pre-bid meeting attendance sheet has been included in the addendum.

2. Questions and Answers from Pre-Bid Meeting and Pre-Bid Minutes

The questions and answers shall be incorporated into the bid documents and are considered official and binding to the contract via this addendum number 1. Pre-Bid minutes are also attached to this addendum.

3. Questions and Answers from CivCastUSA

The questions and answers shall be incorporated into the bid documents via this addendum number 1 and are considered official and binding to the contract for all questions and answers up to the publishing of this addendum. Any question and answers after this date are not included and binding.

4. Geotechnical Boring Logs and Report

Included the full geotechnical report that includes additional technical information and additional boring locations than shown in the boring logs on the plan set.

5. Tax Exempt Status

The Calhoun County will issue a tax exemption certificate for materials paid for and approved that are permanently utilized with this project. Sales Tax on supplies, tools, etc. that are not a permanent part of this project are taxable and shall be the responsibility of the contractor.

PRE-BID CONFERENCE SIGN IN SHEET

Bid No. 2024.05 – Seadrift Drainage Improvements Project
for Calhoun County, Texas
April 16, 2024
10:00 a.m.

Please Print

	Company Name	Representative	Email Address	Phone No.
1.	G&W Engineers, Inc	Anthony Gohlke	anthonyg@gwengineers.com	361 552 4509
3.	"	Zachary Roach	zacharyL@gwengineers.com	"
4.	Keeley construction	Erick Gonzalez	egonzalez@keeleyconstruction.com	(830) 325-5869
5.	J&R contracting ~~Frankie Garcia~~	Frankie Garcia	frankgarcia@prodigy.net	361-220-0537
6.	Calhoun County Pct #4	Gary Reese	gary.reese@calhouncotx.org	361.785.3141
7.	Lester Contracting Inc.	Randy Morris	randy@lestercontracting.com	361-552-3024
8.				
9.				
10.				
11.				
12.				
13.				

Pre-Bid Meeting April 16th 10:00 am G&W Project #: 5310.013a

Seadrift Drainage Improvements – PRE-BID Conference – MEETING MINUTES
Held at County Commissioners office in Seadrift, Texas

Attendees:

In Person:
See Sign in Sheet uploaded to CivCASTUSA.com

Remote call ins:
Marla Jaska - G&W Engineers
Katy Sellers - KSBR

Introductions

- Project description from contract documents
- Main parts of the project primarily consist of two bridges, two culvert crossing and ditch and culvert improvements along 9th street
- Last Questions are to be asked no later than 04/19/24 5pm
- Last questions must be asked on CivCastUSA
- The forms of the contract documents must be filled out completely, instructions are provided in the contract documents
- Question: What was the other bid document requirement stated and what does "local" mean in terms of the local hiring preference stated in the bid documentation?
- Answer: Local preference means to attempt to get local sub-contractors for the project. Typically, "local" will be within 60 miles but as this project is in a more rural location the exact range is undefined. It is required that the contractor make an attempt to hire local if possible. The contractor must also adhere to the Davis Bacon wage rate requirements.
- Bid items preferences, alternate items, and options were discussed
- Question: what are the specific time requirement for working? What days are available for work? Are there working hour requirements?
- Answer: Differed answer to Commissioner Gary Reese. Hours are not specific but should be within sunrise to sunset. Working days are flexible but would like to request no work on Sundays.
- Call for any other plan related questions.
- Question: Can both of the bridges be worked on at the same time?
- Answer: With proper traffic control & emergency vehicle coordination. Coordination with the city and authorities need to be notified

Meeting Minutes (Continued)

- Question: The Barricading bid item is lump sum, how will that be paid out to the contractor?
- Answer: Barricade payment is defined in the note for Item 502 on plan sheet C4.1. The note reads as follows: "This item will be measured by the Lump Sum. The contractor will specify the number of calendar days to complete the work in the bid submittal. The payment for lump sum will be distributed equally for the bid duration of the project. The number of payments for the lump sum item will be calculated as the number of days bid, divided by 30 and rounded up to the nearest whole number to determine the number of monthly payments for barricades. If the contractor finishes the project in less than the number of days bid, the remaining balance for this item will be paid on the final estimate. If the project is not completed within the estimated number of working days, the contractor will not receive additional payment for this item unless time is justified and added by Change Order."
- Question: Do the rebar in the bridge need to be epoxy coated?
- Answer: Epoxy-coated reinforcing steel is required for the abutments and beam slab as stated in the note for Item 440 on plan C4.1. In addition, calcium nitrite inorganic corrosion inhibitor is required in the precast concrete piling, abutments, beams slab and precast beams as stated in the note s for Items 409, 421 and 425 on plan sheets C4.1.
- Question: Can the contractor block the water at the bridge locations to work at the bridge locations?
- Answer: No, the water at the bridge location must be able to flow freely since they are tidally influenced. Since the bridge will span is wider than the existing culvert structure the water should not affect the building of the bridge. The riprap of the abutment should end before or at the water line which will not require blocking the flow in the channel.
- Contractor wanted clarification on buildability of abutments
- Engineer explained that the span between abutments is larger than the current culvert span. The bridge components should be out of the existing water line.

- Question: Please clarify the ditch cleaning if the water cannot be blocked off?
- Answer: The Ditch cleaning will consist of a typical ditch cleaning upstream of the tidally affected area with tops, sides, and bottom of the ditch with the addition of the installation of the Hydroturf product. Downstream of the tidally affected are the cleaning area will consist of only the sides and tops of the banks leaving the tidally affected area alone.

- Question: Are the boring logs for the site available?
- Answer: The Geotech boring logs are shown for the 3rd and 4th Street bridge layouts on sheets C10.3 and C10.4. The full Geotech boring data report taken for the project can be found in the addendum.

Meeting Minutes (Continued)

- Question: Is there a predetermined disposal site for the debris and shrubbery cleared?
- Answer: The contractor is responsible for disposal of all debris and shrubbery in accordance with all applicable state, local and federal requirements as per the note 10 under the Contractor General Responsibilities on plan sheet C4.0. There are no specific provisions regarding a disposal site for the debris. The City's disposal site is only open on Fridays but it is possible that arrangements can be made to open the site on other days for the contractor if the contractor wishes to pursue this route.
- Reiterated that the Bid is due 5/9/24 by 2pm and that bids must be mailed or personally delivered
- Reiterated that any additional questions must be asked on CivCastUSA no later than Friday 04/19/24 by 5pm
- Meeting adjourned

205 W. Live Oak • Port Lavaca, TX 77979 • p: (361)552-4509 • f: (361)552-4987
TBPE Firm Registration No. F4188 • TBPLS Firm Registration No. 10022100

G&W Project #: 5310.013a

Seadrift Drainage Improvements Project – Request for Information Responses
Cutoff Time April 19, 2024 at 5:00 PM

EMAILED RFI's: None

PREBID CONFERNCE RFIs March 10, 2022 – 10:00 AM:

1. *Will the owner/engineer consider using fabric form concrete as an alternative to the HydroTurf Z product for ditch lining? ArmorForm.com has information about the product and installation procedures.?*
   The plans were made with certain materials and products in mind. Alternatives to specified materials / products can be reviewed on a case-by-case basis but the stated product or materials in the plans are preferred. Only true, accurate, and determined to be equals will be allowed.

2. *Will the engineer consider paying for 1' of Cement Stabilized Backfill as bedding under Box Culverts and pipe to pose as seal slab / stable bedding?*
   Sand bedding is specified for the 9th Street storm sewer and driveway structures as shown on the backfill details on plan sheets C5.2 and C5.3. Please bid as shown in the plans. An alternative bedding type may be approved but it will be at the contractor's expense.

3. *Is there Geo Tech for the length of project available you can share?*
   The Geotech boring logs are shown for the 3rd and 4th Street bridge layouts on plan sheets C10.3 and C10.4. The full Geotech boring data report taken for the project can be found in the addendum.

4. *Would you please post the pre-bid meeting attendees list?*
   A copy of the attendee sign in sheet will be attached to the addendum.

5. *Are there any additional bridge details you can share?*
   The bridge is a typical TxDOT design for a prestress concrete beam slab bridge. All applicable TxDOT standards have been included in the plans on sheets C6.1, C6.2 and C6.3.

Pre-bid Meeting Questions

1. *What Does "local" mean in terms of the local hiring preference stated in the bid documentation?*
   Local preference means to attempt to get local sub-contractors for the project. Typically, "local" will be within 60 miles but as this project is in a more rural location the exact range is undefined. It is required that the contractor make an attempt to hire local if possible.

2. *What was the other bid document requirement stated?*
   The contractor must adhere to the Davis Bacon wage rate requirements.

3. *Since the project is in a residential looking area, are there specific working time requirements?*

   Specific times are not specified in the plans. Sunrise to sunset would be within reason. Working on Saturday is acceptable but not Sunday. Coordinate any unusual work times with the Engineer and City.

4. *Can the contractor work on both bridges at the same time?*

   Yes, but please notify and work with the City and emergency services to coordinate dates and available routes for emergency vehicles.

5. *The barricading is bid out lump sum, how will that be paid out to the contractor?*

   Barricade payment is defined in the note for Item 502 on plan sheet C4.1. The note reads as follows: "This item will be measured by the Lump Sum. The contractor will specify the number of calendar days to complete the work in the bid submittal. The payment for lump sum will be distributed equally for the bid duration of the project. The number of payments for the lump sum item will be calculated as the number of days bid, divided by 30 and rounded up to the nearest whole number to determine the number of monthly payments for barricades. If the contractor finishes the project in less than the number of days bid, the remaining balance for this item will be paid on the final estimate. If the project is not completed within the estimated number of working days, the contractor will not receive additional payment for this item unless time is justified and added by Change Order."

6. *Does the rebar in the bridge need to be epoxy coated?*

   Epoxy-coated reinforcing steel is required for the abutments and beam slab as stated in the note for Item 440 on plan C4.1. In addition, calcium nitrite inorganic corrosion inhibitor is required in the precast concrete piling, abutments, beams slab and precast beams as stated in the notes for Items 409, 421 and 425 on plan sheets C4.1.

7. *Can the contractor block the water at the bridge locations to work at the bridge locations?*

   No, the water at the bridge location must be able to flow freely since they are tidally influenced. Since the bridge will span is wider than the existing culvert structure the water should not affect the building of the bridge. The riprap of the abutment should end before or at the water line which will not require blocking the flow in the channel.

8. *Please clarify the ditch cleaning if the water can not be blocked off ?*

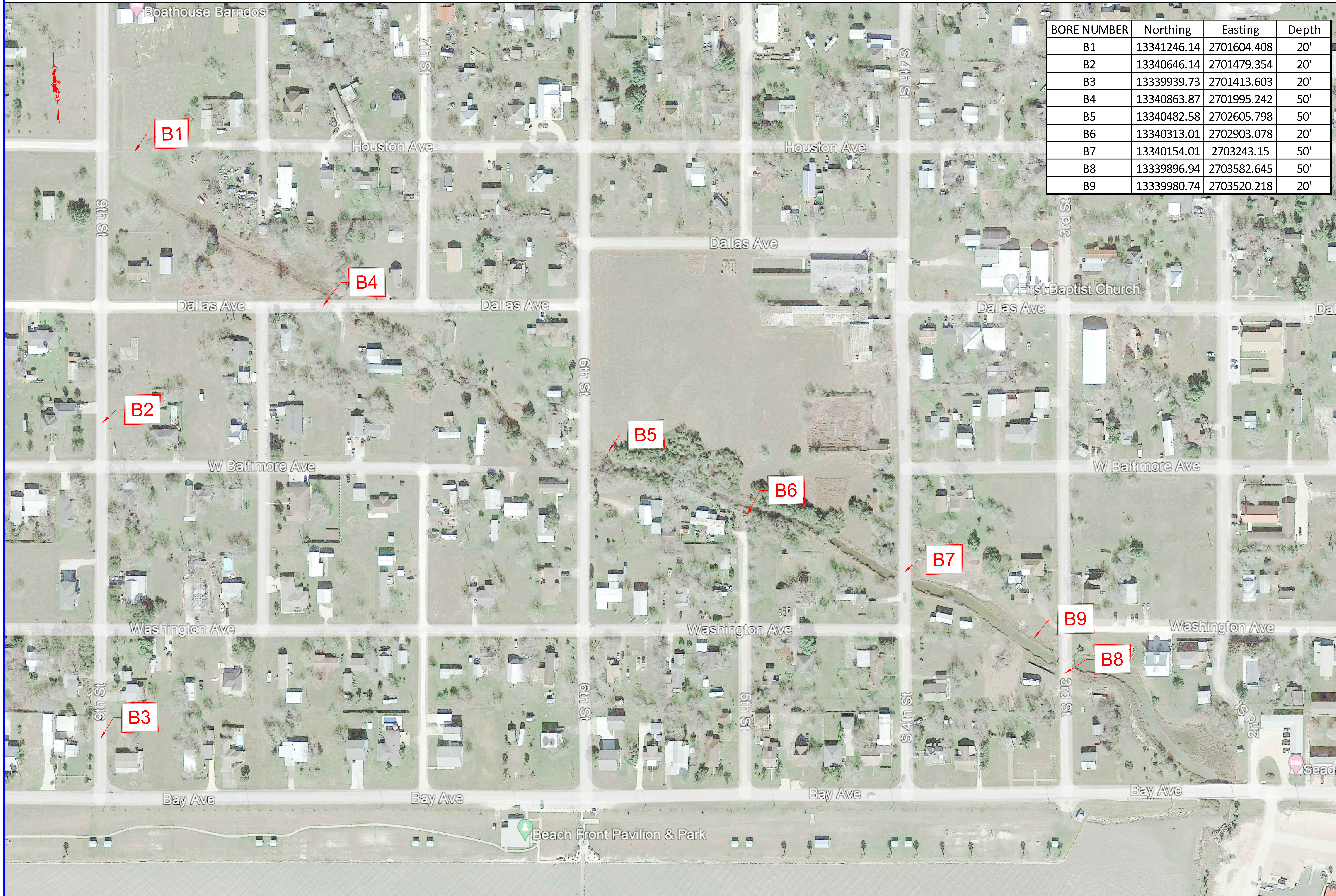
   The Ditch cleaning will consist of a typical ditch cleaning upstream of the tidally affected area with tops, sides, and bottom of the ditch with the addition of the installation of the Hydroturf product. Downstream of the tidally affected are the cleaning area will consist of only the sides and tops of the banks leaving the tidally affected area alone.

9. *Are the boring logs for the site available ?*

   The Geotech boring logs are shown for the 3rd and 4th Street bridge layouts on sheets C10.3 and C10.4. The full Geotech boring data report taken for the project can be found in the addendum.

10. Is there a predetermined disposal site for the debris and shrubbery cleared?

The contractor is responsible for disposal of all debris and shrubbery in accordance with all applicable state, local and federal requirements as per the note 10 under the Contractor General Responsibilities on plan sheet C4.0. There are no specific provisions regarding a disposal site for the debris. The City's disposal site is only open on Fridays but it is possible that arrangements can be made to open the site on other days for the contractor if the contractor wishes to pursue this route.



BORE NUMBER	Northing	Easting	Depth
B1	13341246.14	2701604.408	20'
B2	13340646.14	2701479.354	20'
B3	13339939.73	2701413.603	20'
B4	13340863.87	2701995.242	50'
B5	13340482.58	2702605.798	50'
B6	13340313.01	2702903.078	20'
B7	13340154.01	2703243.15	50'
B8	13339896.94	2703582.645	50'
B9	13339980.74	2703520.218	20'

REV.	DATE	DESCRIPTION	BY	CHK.
A	06/16/22	PRELIMINARY	STC	SM

Texas Registered Engineering Firm F-04188
Louisiana Registered Engineering Firm EF 6497

G & W ENGINEERS, INC.

ENGINEERING SURVEYING PLANNING

205 W. Live Oak St. Port Lavaca, Texas 77979 (361)552-4509
1801 7th St., Suite 260 Bay City Texas 77414 (979) 323-7100

SEADRIFT CDBG-MIT
BORE LOCATION MAP

PRELIMINARY

DWG PROJECT NO. 5310.13A
SCALE: 1"=100'-0"
DWN. BY: STC
CHK. BY: SM
DATE: 06/16/2022
SHT. NO. B1.0

TSI LABORATORIES, INC.

GEOTECHNICAL ENGINEERING STUDY

Calhoun County – GLO - CDBG - MIT Project
Seadrift, Texas

1810 S. Laurent St. * Victoria, TX 77901 * 361-578-6933

TSI LABORATORIES, INC.

TBPE FIRM REGISTRATION NO: F-9236

1810 SOUTH LAURENT
VICTORIA, TEXAS 77901

Telephone 361-578-6933
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Email: tsilabvictoria@gmail.com

September 19, 2022

Scott Mason, P.E.
G&W Engineers, Inc.
205 W. Live Oak
Port Lavaca, TX 77979

Subject: Calhoun County - GLO - CDBG - MIT Project
Seadrift, Texas

TSI File No.: V-221264

Dear Mr. Mason,

We are pleased to submit this report of our geotechnical engineering study for the Calhoun County - GLO - CDBG - MIT Project in Seadrift, TX. The findings and a description of the exploration and testing procedures are presented in the report along with our site preparation recommendations.

We appreciate the opportunity to assist in this phase of the project. Please feel free to contact us, if you have any questions regarding this report or if we may be of further service.

Respectfully submitted,

TSI Laboratories, Inc.

STATE OF TEXAS
DANIEL TESFAI
93944
LICENSED
PROFESSIONAL ENGINEER

Michael Tater, President.

Daniel Tesfai, P.E.

GEOTECHNICAL ENGINEERING STUDY

Calhoun County – GLO – CDBG - MIT Project
Seadrift, Texas

Prepared For:

Scott Mason, P.E.
G&W Engineers, Inc.

Prepared By:

TSI LABORATORIES, INC.
TBPE Firm Registration No.: F-9236

Victoria, Texas

September 19, 2022

TSI Project Number: V-221264

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING STUDY

Calhoun County – GLO - CDBG - MIT Project
Seadrift, TX

INTRODUCTION

Authorization and Scope

TSI Laboratories, Inc. (TSI) was retained to provide geotechnical study services by Scott Mason with G&W Engineers, Inc. The purpose of this study was to determine and evaluate the stratification and engineering properties of the site subsurface soils. TSI will provide geotechnical engineering recommendations and guidelines for use in site preparation, foundation design, and related site improvements planned for the Calhoun County GLO-CDBG–MIT - Project in Seadrift, Texas.

Project Description

The proposed project involves the design and construction of 4 bridges, retaining wall and pavement design for rebuilding 9th street. Based on the results of the study the bridge and retaining wall could be supported by straight shaft or driven pile foundation system.

FIELD AND LABORATORY TESTING

Field Testing

The site soils were explored by drilling five (5) 50-foot-deep borings for the bridges and four (4) 20-foot-deepborings. Boring locations were determined by the client and shown on the respective site plan. Soil was sampled continuously at 2-foot intervals to 10-foot depth with an additional sample taken at 5-foot depth intervals. The sampling method is determined based on the encountered soils.

Cohesive soils were sampled by hydraulically pushing a 3-inch diameter, thin-walled steel tube a distance of about 24-inches. Our field sampling procedures were in general accordance with ASTM D1587. For each recovered sample, our representative extruded the sample in the field, visually classified the soil, and measured the penetration resistance using a pocket penetrometer.

Granular soils were sampled as part of the Standard Penetration Test (SPT) by driving a 2-inch diameter split-barrel sampler. The sampler was driven 18-inches by a 140-pound hammer falling 30-inches in general accordance with the ASTM D1586. Our representative recorded the number of blows required to drive the sampler through three consecutive 6-inch intervals. As permitted by ASTM D1586, sampling was terminated if 50-blows were recorded within any single 6-inch interval. The sum of blows required to penetrate the final 12-inches is known as the SPT "N" value. A portion of the recovered sample was wrapped in aluminum foil and placed into a sample container and transported to our laboratory for testing.

Laboratory Testing

The soil samples selected for laboratory testing were examined and visually classified by the sample's representative of the various soil strata encountered. Atterberg limits, moisture contents and percent fines tests were performed to assist in classifying the soils according to Unified Soil Classification System (ASTM D2487). Unconfined compressive strength tests were also performed to provide indicators of soil strength. The classification test results are presented on the boring logs. The test procedures are described in the Appendix.

SUBSURFACE CONDITIONS

Soil classifications are described in detail in the Boring Logs provided in the Appendix and summarized in the table below.

Boring No.	Soil Type				Ground water table
B-1	0-8' CH	8-20' SC	-	-	11'
B-2	2 Coarse/shell base 10.0"	0-1' SC	1-13' CL/CH	13-20' SM-SC	12'
B-3	2 Coarse/shell base 18.0"	1-6' CH	6-18' SC/ SM-SC	18-20' CH	12'
B-4	2 Coarse/ shell base 10.0"	1-6' CH	6-28' SC/ SM-Sc	28-50' CL	10'
B-5	0-4' CL	4-23' SM-SC	23-43' CL	43-50' CH	7'
B-6	0-4' CL	4-28' SC	28-50' CL	-	6'
B-7	0-6 CH/CL	6-28' SM-SC	28-50' CL	-	7'
B-8	0-20' SM-SC	-	-	-	8'
B-9	0-33' SC/ SM-SC	33-50' CH	-	-	8'

The site soil has been evaluated by performing various field and laboratory tests on the subsurface samples recovered during the drilling operations. The types of tests conducted on the subsurface samples and the results of the tests are tabulated on the Logs of Borings, which are provided in the Appendix. The properties of each stratum are discussed below.

The corresponding boring logs, depicting the stratum soil descriptions, type of sampling used during sample retrieval, laboratory test data, and other field data, is presented in the Appendix at the end of this report. The key to the boring log symbols and soil classifications Sheet, which defines the terms and descriptive symbols used on each boring log, is also presented in the Appendix.

Groundwater

Groundwater was encountered between 6 to 12-foot in all of the borings during drilling operations as indicated in the table above. It is noted that groundwater levels fluctuate with seasonal climatic variations and the contractor should verify that groundwater will not adversely affect design or construction at this site.

RECOMMENDATIONS

The foundation system for the proposed bridge and retaining wall must satisfy two independent engineering criteria with respect to the soil conditions. First, the foundation system should be designed with an appropriate factor of safety against bearing capacity failure of the foundation soils. Second, the movement of the foundation system due to compression (consolidation) or expansion (swell) of the soils supporting the foundation system must be within tolerable limits for the structure.

Minimum Embedment Requirement

We recommend that the design of the shafts or piles consider the following minimum embedment requirements:

- The shafts or piles should have a depth of embedment adequate for support of the imposed axial and lateral loads.
- Computation of design capacities of the shafts or piles should provide for reductions in capacities due to construction-related disturbance and shrink-swell of surficial soils with changes in moisture.
- Shafts or piles should penetrate into the base stratum (the stratum in which the shaft or pile tip is placed) at least two (2) times their diameter or width.

Drilled Straight-Shafts

Drilled straight shafts may be utilized to support the proposed bridge. The drilled shafts if extended to a depth of 15 to 23-foot of the existing grade, the shafts should be sized for a net dead plus sustained live load bearing pressure of 3,200 psf or a net total load bearing pressure of 4,800 psf, whichever condition results in a larger bearing surface. However, if extended below 24-foot and below a net dead plus sustained live load bearing pressure of 3.8 ksf or a net total load bearing pressure of 5.7 ksf. The same value can also be used for the cast in place box culvert with 15 inches thick perimeter beams.

The shafts should contain sufficient vertical reinforcing steel throughout the entire shaft length to resist uplift (tensile) forces due to post-construction heave of the clayey soils. The magnitude of uplift is difficult to predict and will vary with in-situ moisture contents. For purposes of establishing sufficient reinforcing to resist uplift, the uplift pressures can be approximated by using a uniform uplift pressure of 550 psf over the perimeter of the shaft embedded. The amount of reinforcing steel required can be computed by assuming the dead load of the structure surcharges the shaft, that the above estimated tensile force acts vertically on the shaft, and that the shaft embedment acts as a rigid anchor. However, in no case should the percentage of steel be less than 0.5% (based on 40 ksi steel).

Drilled shaft edge-to-edge spacing of less than two (2) shaft diameters will require axial capacity reduction. TSI should be contacted for additional recommendations if the clear spacing between drilled shafts is less than two shaft diameters.

Allowable side shear value of 1,100 psf with an assumed factor of safety of at least two (2) may be used to aid in resisting axial compressive loads on the piers. The side shear should be neglected for fill material, the upper 5-foot of soil in contact with the pier shaft, and within one (1) pier diameter of the bottom of the shaft.

Driven Piles

Driven concrete piles may be used to support the proposed bridges. The allowable unit skin and end bearing capacities provided in the following table are recommended for the design of full displacement driven piles. The friction factor defines the increase in pile friction capacity as a function of depth. These values include a factor of safety of 2. Allowable tension can be taken as 80% of the friction value. The end bearing factor can be used to estimate end bearing capacity. These values include a factor of safety of 2.5.

Penetration[1] (foot)	Allowable Unit Skin Friction (psf)	Allowable Unit End Bearing Pressure[2] (psf)
0 to 5	Disregard	
6 to 13	250	Disregard
14 to 30	325	3,260
31 to 50	360	3,580

[1] Penetration below grade existing at the time of field investigation
[2] Neglect for pile dimension less than 2-foot

The parameters required for p-y curves and designs from the boring logs are listed in the table below:

Soil Type	γ	C	ϕ	K	K_o	*E_{50}
Medium Dense Silty Sand (SM)	115	0	24	60	0.60	-
Stiff Clay (CH)	105	1,400	0	140	0.53	0. 007

* Values estimated from known correlations.

Where: γ = Wet Unit Weight, pcf
c = soil cohesion, psf
ϕ = Angle of internal Friction, deg
K_o = Lateral Earth Pressure Coefficients, At-Rest
K = modulus of subgrade cyclic reaction (pci)

Driven piles generally derive most of their load carrying capacity from skin friction. Therefore, end bearing is normally negligible for driven piles. We recommend the allowable unit end bearing values presented in the above table should be neglected if the selected driven pile dimension is less than 2-foot. In addition, the allowable unit skin friction and unit end bearing values provided in the above table are recommended for driven concrete (full displacement) piles. TSI should be contacted for additional recommendations if driven piles other than full-displacement concrete piles are planned to be utilized at this site.

Lateral resistance of driven piles is primarily developed by passive resistance of the soil against the side of the pile. A detailed lateral load analysis of the proposed piles was beyond the scope of this study. If requested, a detailed lateral capacity analysis of the proposed driven piles can be provided for this project.

Design of piles should also include an evaluation of the structural capacity of the pile which may limit the allowable capacity. Any pile splices must provide positive load transfer both in compression and tension since driving displacement piles within clayey soils could result in heave. As the pile is driven, it displaces soil upward toward the surface. This upward soil movement can "drag" the adjacent piles up and lift them off of their bearing layer causing tension along the piles.

Pile groups subjected to axial loads can be influenced by numerous factors which may include pile type, size and length, pile spacing, overall group size, loading conditions, installation procedures, and soil type and strength. With a center-to-center spacing of at least three (3) pile widths, the group effect should be insignificant on the load carrying capacity of the piles. Therefore, the combined axial load capacity for such a group may be taken as the sum of the individual pile capacities in the group.

Post construction settlements of single isolated piles will depend on the elastic properties of the pile, the applied load, and the interaction of the soil and pile. Settlement is anticipated to be primarily elastic and will occur relatively rapidly as load is applied. Significant consolidation settlement due to applied load is not anticipated at this site for the pile capacities given. Our experience indicates that single, isolated piles loaded to about one-half of their ultimate capacity should experience settlement of less than one inch.

Post construction settlements of groups of piles are generally greater than single isolated piles for the same load per pile. Based on the previously recommended spacing, we anticipate that settlement of the pile group should be one inch or less under working loads. In general, differential settlements should be on the order of one-half to two-thirds of the total settlement.

The installation of the piles should preferably be accomplished by driving alone. However, predrilling, or controlled jetting may be required to achieve the design penetrations if excessive resistance to penetration occurs during driving. We anticipate that predrilling may be necessary to achieve penetration to any appreciable depth into the sandy soils observed at this site. The effects and methods of pile installation should be given proper consideration when choosing and designing pile foundation systems. In most situations, the greatest stress a driven pile will experience is during installation. Pile and soil properties, embedment length requirements, and driving equipment are only a few of the many variables to consider in determining the most efficient method of pile installation.

Driving piles to completed embedment depths may be facilitated by predrilling to a depth somewhat less than the anticipated final embedment depth. Under no circumstances should predrilling extend deeper than 5-foot above the final pile embedment depth unless refusal occurs. The predrilled excavation should be about 4-inches less in width than the pile size to promote the development of skin friction resistance. Extreme care should be exercised during predrilling since it can affect the lateral and axial capacities of the pile.

Production piles should be driven to a predetermined (design) depth with blow count as a secondary consideration. Because set-up during interruption can produce increased resistance to driving, a pile should be driven to its design depth without any delays, if possible.

If a pile exhibits a resistance lower than the terminal resistance values given by driving formulae at an appreciable depth below the predetermined depth, the pile may be re-tapped after a suitable elapsed time and after the installation of other nearby piles. The re-tap should be performed at the contractor’s discretion to prove the acceptability of the pile. Should, in the Geotechnical Engineer’s judgment, the re-tap not indicate adequate capacity, a new pile should be installed to provide the required capacity.

In addition, we recommend the use of a “Pile Driving Analyzer” (PDA) during pile installation. The PDA can monitor driving stresses and hammer energy during pile installation, and also provides a continuous record of the pile installation. Such information can be beneficial in evaluation of the acceptability of a driven pile.

Settlement Considerations

Total settlements, based on the indicated bearing pressures, should be about 1-inch for properly designed and constructed drilled piers. Settlement beneath individual piers will be primarily elastic with most of the settlement occurring during construction. Differential settlement may also occur between adjacent piers. The amount of differential settlement could approach 50 to 75% of the total pier settlement. For properly designed and constructed piers, differential settlement between adjacent piers is estimated to be less than ¾-inch. Settlement response of drilled piers is impacted more by the quality of construction than by soil structure interaction.

Improper pier installation could result in differential settlements significantly greater than we have estimated. In addition, larger magnitudes of settlement should be expected if the soil is subjected to bearing pressures higher than the allowable values presented in this report.

Sheet Pile Recommendations

Retaining structures must be designed in a way such that serviceability and ultimate limit states are not reached. Serviceability limit states are mostly associated with excessive deflection of the structures toward the open space and away from the ground they support. This leads to a corresponding loss of ground behind the retaining structure; if a structure is present there, it may be damaged by the resulting settlements. Ultimate limit states include bearing capacity failure, sliding, overturning and general stability of the retaining structure and the soil it supports. Additionally, the retaining structure has to retain structural integrity through its useful life.

The sheet piles can be installed by vibratory hammers as sand liquefaction due to the vibration aids the rate of advance. For short sheet piles, jacking can be quite efficient, as agile, hydraulic machines now exist that can quickly push sheet piles into the ground.

Lateral Earth Pressure

Below-grade walls or retaining structures may be used for some structures in the facility. The walls will be subject to lateral earth pressures from a combination of soil pressure, hydrostatic water pressure, and surcharge loads. The earth pressure, σh, for soils adjacent to the below-grade walls, is expected to approach at-rest conditions and may be computed as:

$\sigma h = K_o \gamma H$

where K_o = At-rest earth pressure coefficient = 0.47
γ = Unit weight of the adjacent soil, lb./ft^3= 56
H = Wall height, foot

Lateral earth pressures resulting from the soil are calculated by multiplying the equivalent fluid density of the surrounding soils, defined as $K_o\gamma$, by the depth below the ground surface. For water pressures, multiply the unit weight of water by the depth below the ground surface, finished grade, or 100-year flood elevation, whichever is greater.

The equivalent fluid densities of 80 pcf for moist and 37 pcf for submerged can be used. These values do not include a hydrostatic pressure component. The sum of the pressures resulting from soil and water, acting as a triangular distribution, should be used for the wall design.

Engineering Design Manuals often specifies that lateral loads due to surcharge loading from cranes and H-20 trucks shall be included. Lateral earth pressures from uniformly distributed surcharge loads can be calculated by using a rectangular stress distribution of the imposed vertical load multiplied by the appropriate lateral earth pressure coefficient. For this reason, a lateral earth pressure coefficient of 0.52 to 0.65 may be used.

If there is movement of sheet pile bulkhead during its service life, both active and passive pressures will be mobilized. Based on existing surface at time of drilling operations, coefficients of 0.36 and 2.77 up to 13-foot depth, 0.49 and 2.03 between 14 to 40-foot depth and below that 0.30 and 3.26 may be used for calculation earth pressures for Active and Passive Rankine earth pressure, respectively.

A surcharge imposed on the soil adds to the lateral earth pressure exerted against the retaining wall due to the loading on the piling of the mooring structure. This added pressure must, be considered in the design, and can be computed as:

$P' = qHK_a$

Where q = surcharge load

H = height of the wall

K_a = coefficient of active earth pressure = $(1-\sin\phi) / (1+\sin\phi)$

Below-grade walls should be checked against failure due to overturning, sliding, and overall slope stability. Such analysis can only be performed during a detailed study once the dimensions of the bulkheads are known.

The boring logs indicate that the soil conditions encountered should not pose any difficulty to the dredging contractor. The maximum side slope conditions should not exceed two (2) horizontals to one (1) vertical ratio.

Foundation Construction

Drilled Straight-Shafts

The drilling contractor should be experienced in the subsurface conditions observed at the site, and the excavations should be performed with equipment capable of providing clean bearing area, free of water. Drilled straight-shaft foundations should be installed in general accordance with the procedures presented in "Drilled Shafts: Construction Procedures and Design Methods," by Reese, L. C. and O'Neill, M. W., FHA Publication No. FHWA-IF-99-025, 1999 and "Standard Specification for the Construction of Drilled Piers", ACI Publication No. 336.1-01, 2001.

Foundation installation should be closely monitored by a qualified technician experienced in drilled straight-shaft installation techniques. At a minimum, the technician should monitor shaft excavation, note any unusual installation occurrences, monitor concrete placement, and generally evaluate if foundation installation is being performed in accordance with the project specifications.

As stated previously, groundwater was observed in all the borings during drilling. Based on the subsurface and groundwater conditions observed at the borings, the installation of drilled straight-shafts will require the use of temporary steel casing. We recommend that provisions be incorporated into the plans and specifications to utilize casing to control sloughing and/or groundwater seepage during shaft construction. To evaluate the constructability of drilled straight-shafts and the potential variability of groundwater conditions, we recommend at least two test shafts prior to the installation of production shafts. The installation of test shafts should be observed by TSI.

If casing is used and seepage persists, the water accumulating in the foundation excavation should be pumped out. The condition of the bearing surface should be evaluated immediately prior to placing concrete. Where casing is used, removal of the casing should be performed with extreme care and under proper supervision to minimize mixing of the surrounding soil and water with the fresh concrete. Rapid withdrawal of the casing may develop a suction that could cause the soil and water to flow into the excavation. An insufficient head of concrete in the casing during withdrawal could also allow the water to intrude into the wet concrete. The casing must be removed in order to utilize the skin friction values previously provided. Under no circumstances should loose soil be placed in the annulus between the casing and the drilled shaft sidewalls.

Driven Pile Foundation Installation

Piling should be installed in accordance with TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, 2014, Items 404 and 409. If piles are to be installed to any appreciable depth, a pile drivability analysis should be performed. A drivability analysis will help in evaluating the pile-hammer combination best suited for pile installation, reducing the need for installation aids, and reducing the risk of pile damage resulting from excessive driving stresses.

Foundation Construction Monitoring

The performance of the recommended foundation systems for the proposed structures will be highly dependent upon the quality of construction. Thus, we recommend that foundation installation be monitored full time by an experienced TSI soil technician under the direction of our geotechnical engineer.

PAVEMENT RECOMMENDATIONS

We anticipate that the subgrade will consist of low to medium plasticity, surficial on-site soils. We recommend that the top HMAC and base material be recycled and stabilized with a minimum of 5% cement by weight and used as base material. This percentage is typically equivalent to about 30 pounds of cement per square yard per 6-inch treated depth. On top, additional 3-inches of crushed limestone or crushed concrete meeting the requirements of TxDOT 2014 Standard Specifications Item 247, Type A, B, or D, Grade 1 should be added. The base material should be compacted to at least 95% of the Modified Effort (ASTM D1557) maximum dry density at a moisture content within 2% of the optimum moisture content.

Upon completion of the base section the surface should be primed and 2½-inches of type "D" HMAC be placed. The asphaltic surface shall meet the requirements of the current TxDOT 2014 Specification Item 340 for Dense Graded Hot Mix Asphalt (small quantity) for projects with total production of less than 5,000 tons and TxDOT 2014 Specifications Item 341 Dense Graded Hot Mix Asphalt for projects with total production of 5,000 tons or greater. The hot mix asphaltic surface will be compacted to between 3.0 and 8.5% in place air voids in conformance with the specification. It is recommended that the testing required by this specification be performed during production. The target design laboratory density should be 97% of the maximum theoretical density for from Rice Test.

Site Grading

On most project sites, the site grading is accomplished relatively early in the construction phase. As construction proceeds, rainfall and surface water saturates in some areas, heavy traffic from concrete trucks and other delivery vehicles disturbs the subgrade. As a result, the pavement subgrades, initially prepared early in the project, should be carefully evaluated as the time for pavement construction approaches.

We recommend the moisture content and density of the top 8-inches of the subgrade be evaluated and the pavement subgrades be proof rolled two (2) days prior to commencement of actual paving operations. Areas not in compliance with the required ranges of moisture or density should be moisture conditioned and recompacted. Particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Areas where unsuitable conditions are located should be repaired by removing and replacing the materials with properly compacted fills.

Rolling Pattern

A minimum compaction temperature of 175°F (80°C) is the cutoff point, because after this point, the mat temperature is so low that compaction possibilities decrease rapidly. In some cases, the material is too hot to be properly compacted. This is noticeable from the instability of the material under the roller. It is essential that the first pass be made as soon as possible so that the temperature relationships mentioned above will be maintained. The greatest part of compaction is attained with the first breakdown pass. To eliminate or minimize compactor marks the final finishing passes may have to be delayed until the mat cools to the proper temperature.

Weather Limitations

Adverse weather conditions would affect the quality of the asphaltic concrete pavement. These include, but are not limited to the following:

1. Frozen subgrade as evident by the fact that a shaded surface thermometer reads 32°F or less, or the subgrade is excessively hard, or the entrapped water has turned to ice.
2. For thin lifts temperature requirements such as 80°F.
3. Muddy subgrade due to the material being too wet.

4. Standing water on the subgrade (this can usually be remedied by using pumps and/or an air hose).
5. A light rain is sometimes OK as long the mat does not cool down too quickly.

The pavement design methods described above are intended to provide structural sections with adequate thickness over a particular subgrade such that wheel loads are reduced to a level the subgrade can support. The support characteristics of the subgrade for pavement design do not account for shrink/swell movements of an expansive clay subgrade. 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. Post-construction subgrade movements and some cracking of pavements are not uncommon for clayey subgrade conditions such as those observed at this site. Minimizing moisture changes in the subgrade is important to reduce shrink/swell movements. Although cement treatment will help to reduce such movement/cracking this movement/cracking cannot be economically eliminated.

GENERAL COMMENTS

TSI should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. TSI also should be retained to provide testing and observation during excavation, grading, foundation, and construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, and 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.

For any excavation construction activities at this site, all Occupational Safety and Health Administration (OSHA) guidelines and directives should be followed by the Contractor during construction to insure a safe working environment. In regard to worker safety, OSHA Safety and Health Standards require the protection of workers from excavation instability in trench situations.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either expressed or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless TSI reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX

N

Map is not to scale.

Legend

Geotechnical Borings

TSI LABORATORIES, INC.			
Calhoun County – GLO – CDBG – MIT Project Seadrift, TX			
Drawn by: BT	Date: 09/02/22		Figure 1
Checked by: AN	Date: 09/02/22		
TSI Project No. V-221264			
Boring Location Map			

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-1
PROJECT NO.: V-221264
DATE: 08/18/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-20.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 11.0'

FIELD DATA				LABORATORY DATA									
DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Atterberg Limits %: Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0	Steel Tube	Fat Clay	P=4.5+									FAT CLAY - with sand, very dark gray (CH)
	Steel Tube	Fat Clay	P=4.5+	23	105	56	20	36	78	7.2	4.6	
												- brown and light brown (CH)
	Steel Tube	Fat Clay	P=3.5									
	Steel Tube	Fat Clay	P=3.0									
8	Steel Tube	Clayey Sand	P=2.5	18	109	24	13	11	37	0.9	6.6	CLAYEY SAND - brown and light brown (SC)
	Split Spoon	Clayey Sand	N=21	20		23	12	11	40			
16												
												- light red (SC)
	Split Spoon	Clayey Sand	N=45	25		23	13	10	16			
												Boring terminated at 20.0'
24												
32												
40												
48												
56												

Steel Tube Sample

Split Spoon Sample

Disturbed Sample

REMARKS:

TSI
Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-2
PROJECT NO.: V-221264
DATE: 08/18/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-20.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 12.0'

DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0	Disturbed											2 Coarse Shell Base - 10.0"
	Steel Tube		P=4.0	14					30			CLAYEY SAND - dark gray (SC)
	Steel Tube		P=3.0			70	24	46				FAT CLAY - dark gray (CH)
	Steel Tube		P=2.5	35	89	69	24	45	85	1.7	4.5	
	Steel Tube		P=3.0									
8	Steel Tube		P=4.0	17	116	32	12	20	78	2.1		LEAN CLAY - with sand, dark gray (CL)
	Split Spoon		N=14									SILTY CLAYEY SAND - light brown (SM-SC)
16												
	Split Spoon		N=20	21		22	16	6	23			
												Boring terminated at 20.0'
24												
32												
40												
48												
56												

Steel Tube Sample
Split Spoon Sample
Disturbed Sample

REMARKS:
Boring began to cave in at 13.0'

TSI Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-3
PROJECT NO.: V-221264
DATE: 08/29/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-20.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 12.0'

FIELD DATA: N : BLOWS/FT, T : INCH/100 BLOWS, P : TONS/SQ. FT., R : PERCENT, RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Atterberg Limits %: Liquid Limit	Atterberg Limits %: Plastic Limit	Atterberg Limits %: Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
									2 Coarse
									Flex Base/Shell Base - 18.0"
P=4.5+									CLAYEY SAND - reddish brown (SC)
P=4.0	31	91	63	20	43	77	2.5	2.5	FAT CLAY - with sand, brown and gray (CH) - brown (CH)
P=3.5									CLAYEY SAND - light brown (SC)
P=4.0	17	116	35	16	19	48	1.9	7.6	
N=14	21		22	15	7	24			SILTY CLAYEY SAND - light brown (SM-SC)
N=22	17		50	20	30	65			FAT CLAY - light brown (CH)
									Boring terminated at 20.0'

DEPTH (FEET): 0, 8, 16, 24, 32, 40, 48, 56

SAMPLE / SOIL TYPE

Steel Tube Sample
Split Spoon Sample
Disturbed Sample

REMARKS:
Boring began to cave in at 10.0'

TSI
Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-4
PROJECT NO.: V-221264
DATE: 8/26/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-50.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 10.0'

DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0	Disturbed			24								2 Coarse Flex Base & Shell Base - 10.0" CLAYEY SAND - dark brown (SC)
	Steel Tube		P=3.5									
	Steel Tube		P=3.0	32	89	70	25	45	76	1.7	1.3	FAT CLAY - with sand, dark brown (CH) - light brown (CH)
	Steel Tube		P=4.5									
	Steel Tube		P=3.5	17	113	26	12	14	34	0.8	1.3	CLAYEY SAND - organgish brown (SC)
8	Steel Tube		P=3.5									
	Split Spoon		N=14	28		20	15	5	18			SILTY CLAYEY SAND - organish brown (SC)
16												
	Split Spoon		N=21									
24	Split Spoon		N=26	31		23	13	10	24			CLAYEY SAND - organish brown (SC)
	Split Spoon		N=51									LEAN CLAY - with sand and gravel, orangish brown (CL)
32												
	Split Spoon		N=69	28		49	18	31	79			- reddish brown (CL)
	Split Spoon		N=76									
40												
	Split Spoon		N=70	25		41	20	21	73			- organish brown (CL)
48	Split Spoon		N=88	30								- light brown (CL)
												Boring terminated at 50.0'
56												

Steel Tube Sample
Split Spoon Sample
Disturbed Sample

REMARKS:
Boring began to cave in at 11.0'

TSI Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-5
PROJECT NO.: V-221264
DATE: 8/29/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-50.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 7.0'

FIELD DATA				LABORATORY DATA								
DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Atterberg Limits % Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0	Steel Tube	CL	P=3.5									LEAN CLAY - brown and red (CL)
	Steel Tube	CL	P=3.0	17	115	47	18	29	52	7.6	5.4	
	Steel Tube	SC	P=2.5	20	115	26	16	10	33	1.3	3.5	CLAYEY SAND - light brown (SC)
	Split Spoon	SC	N=13									
8	Split Spoon	SC	N=19	21		24	13	11	26			
	Split Spoon	SM-SC	N=22									SILTY CLAYEY SAND -light brown (SM-SC)
16	Split Spoon	SM-SC	N=25	24		21	14	7	13			
24	Split Spoon	CL	N=28									LEAN CLAY - with sand, light brown (CL)
	Split Spoon	CL	N=44	23		31	14	17	75			
32	Split Spoon	CL	T=62									- red (SC)
40	Split Spoon	CL	N=60									
	Split Spoon	CH	N=70									FAT CLAY - red (CH)
48	Split Spoon	CH	N=87	30		60	21	39	95			
56												Boring terminated at 50.0'

Steel Tube Sample

Split Spoon Sample

Disturbed Sample

REMARKS:
Boring began to cave in at 11.0'

TSI
Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-6
PROJECT NO.: V-221264
DATE: 09/02/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-50.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 6.0'

FIELD DATA			LABORATORY DATA								
DEPTH (FEET)	SAMPLE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Atterberg Limits % Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0	Steel Tube Sample	P=2.0									LEAN CLAY - dark gray (CL)
	Disturbed Sample		16		49	19	30	56			
	Disturbed Sample		17	113	35	14	21	41	1.0	5.7	CLAYEY SAND - light gray (SC)
	Split Spoon Sample	N=17									
8	Split Spoon Sample	N=28									
	Split Spoon Sample	N=8	27		23	14	9	12			
16											
	Split Spoon Sample	N=19									
24	Split Spoon Sample	N=25									- light red (SC)
	Split Spoon Sample	N=45	22		28	14	14	55			LEAN CLAY - light red (CL)
32											
	Split Spoon Sample	N=63									- with sand, red (CL)
	Split Spoon Sample	N=69									
40											
	Split Spoon Sample	N=65	25		47	18	29	75			
48	Split Spoon Sample	N=86									
											Boring terminated at 50.0'
56											

Steel Tube Sample
Split Spoon Sample
Disturbed Sample

REMARKS:
Boring began to cave in at 6.0'

TSI Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-7
PROJECT NO.: V-221264
DATE: 09/02/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-50.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 7.0'

DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft.3	Atterberg Limits % Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0			P=3.5									LEAN CLAY - dark gray (CL)
			P=4.5	22	98	73	23	50	72	3.1	3.6	FAT CLAY - with sand, gray (CH)
												LEAN CLAY - light brown (CL)
			N=13									SILTY CLAYEY SAND - light brown (SM-SC)
8			N=19	21		29	13	16	22			
			N=22									
16												
			N=25	28		22	14	8	13			CLAYEY SAND - light brown (SC)
												- light red (SC)
24			N=28									
			N=44									LEAN CLAY - red (CL)
32												- with sand, red (CL)
			N=60									
			N=72	23		47	18	29	83			
40												
			N=68									
48			N=86									
												Boring terminated at 50.0'
56												

Steel Tube Sample
Split Spoon Sample
Disturbed Sample

REMARKS:

TSI Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-8
PROJECT NO.: V-221264
DATE: 09/02/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S): Dry Auger 0-20.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 8.0'

DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Atterberg Limits % Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0	Steel Tube		P=3.5									CLAYEY SAND - with gravel, gray (SC)
	Steel Tube		P=2.75	15	110	45	20	25	29	1.5	8.4	
	Steel Tube		P=1.0									- gray (SC)
	Split Spoon		N=13	18		17	11	6	30			SILTY CLAYEY SAND - light gray (SM-SC)
8	Split Spoon		N=20									
	Split Spoon		N=21									
16												
	Split Spoon		N=24	29		24	13	11	13			CLAYEY SAND - light gray (SC)
												Boring terminated at 20.0'
24												
32												
40												
48												
56												

Steel Tube Sample
Split Spoon Sample
Disturbed Sample

REMARKS:
Boring began to cave in at 8.0'

TSI Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County - GLO - CDBG - Mit Project
Seadrift, TX

CLIENT: G&W Engineers, Inc.

BORING NO.: B-9
PROJECT NO.: V-221264
DATE: 09/02/22
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S): Dry Auger 0-50.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 8.0'

DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft.3	Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0			P=3.5	19	105	48	20	28	44	2.4	15.0	CLAYEY SAND - with gravel, dark gray (SC)
			P=2.75									- with gravel, dark gray (SC)
			P=1.0	21	107	31	12	19	45	1.4	15.0	- gray (SC)
			N=11									SILTY CLAYEY SAND - light gray (SM-SC)
8			N=20									
			N=20	20		20	12	8	22			
16			N=26									
24			N=27									CLAYEY SAND - light red (SC)
			N=44									- red (SC)
32			N=60	28		58	23	35	86			FAT CLAY - red (CH)
40			N=68									
			N=66	32		61	25	36	95			
48			N=85									
56												Boring terminated at 50.0'

Atterberg Limits %: Liquid Limit, Plastic Limit, Plasticity Index

Steel Tube Sample

Split Spoon Sample

Disturbed Sample

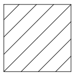
REMARKS:
Boring began to cave in at 8.0'

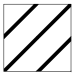
TSI Laboratories, Inc.

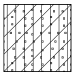
KEY TO SYMBOLS

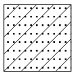
Symbol Description

Strata symbols

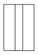
 **Low plasticity clay**

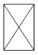
 **High plasticity clay**

 **Poorly graded clayey silty sand**

 **Clayey sand**

Soil Samplers

 **Steel Tube Sample**

 **Split Spoon Sample**

 **Disturbed Sample**

Consistency of Sands & Gravels	
Consistency	**Penetration Resistance (N)* Blows Per Foot**
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	Over 50

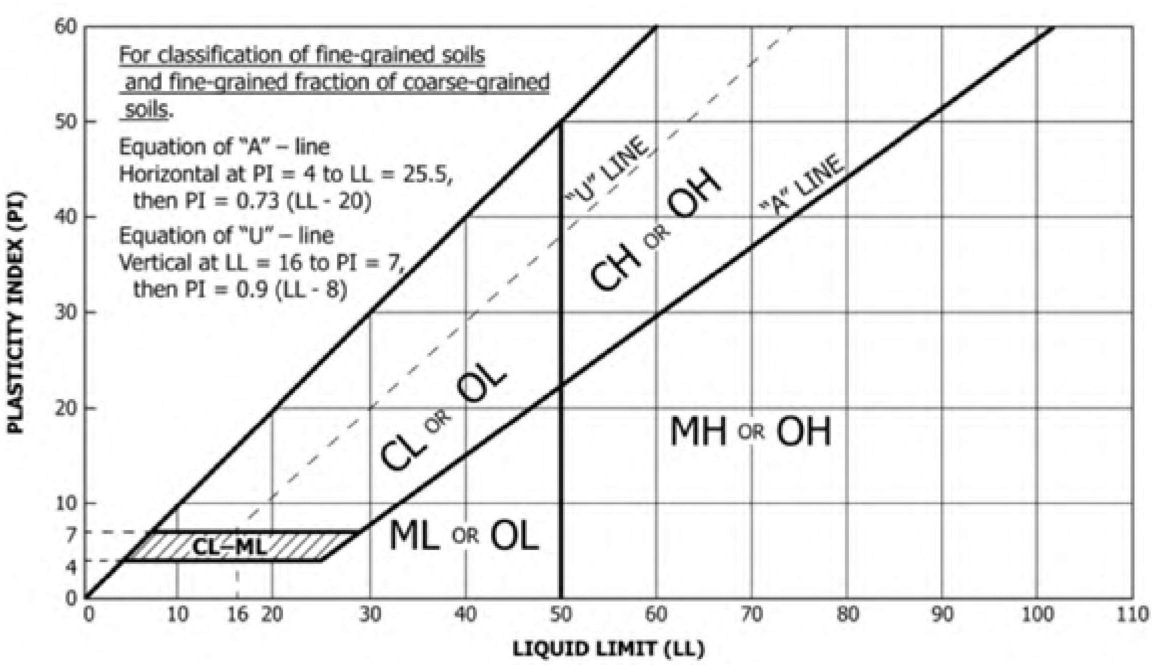
Consistency/Strength of Clays & Silty Clays		
Consistency	**Undrained Shear Strength, tsf**	**Pocket Penetrometer (p)**
Very Soft	Less than 0.125	0 – 0.5
Soft	0.125 – 0.25	0.5 – 1.0
Firm	0.25 – 0.50	1.0 – 1.75
Stiff	0.50 – 1.0	1.75 – 3.5
Very Stiff	1.0 – 2.0	3.5 – 4.5
Hard	Over 2.0	Over 4.5

*N=Number of Blows from 140 lb. hammer falling 30"to drive a 1-3/8" ld. split barrel sample (ASTM D-1586)

Soil Grain Analysis
US Standard Sieves

Boulders	6"	Cobbles	3"	Gravel	¾"	Gravel	#4	Sand	#10	Sand	#40	Sand	#200	Silt		Clay
				Coarse		Fine		Coarse		Medium		Fine				
	152		76.2		19.1		4.76		2.0		0.420		0.074		0.002	

Soil Grain Size in Millimeters
ASTM D-2488



FIELD AND LABORATORY TESTING PROCEDURES
(TEST PROCEDURES ARE PRESENTED FOR INFORMATIONAL PURPOSES)

FIELD TESTING

A. Boring Procedure Between Samples

The borehole is extended downward, between samples, by continuous flight, hollow or solid stem augers or by rotary drilling techniques using bentonite drilling fluid or water.

B. Penetration Test and Split-Barrel Sampling of Soils
ASTM D-1586

This sampling method consists of driving a 2-inch outside diameter split barrel sampler using a 140 pound hammer freely falling through a distance of 30 inches. The sampler is first seated 6 inches into the material to be sampled and then driven an additional 12 inches. The number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance. Recovered samples are first classified as to color and texture by the driller. Later, in the laboratory, the driller's field classification is reviewed by the soils engineer who examines each sample.

C. Thin-Walled Tube Geotechnical Sampling
ASTM D-1587

This method consists of pushing thin walled steel tubes, usually 3 inches in diameter, into the soils to be sampled using hydraulic or other means. Cohesive soils are usually to be sampled in this manner and relatively undisturbed samples are recovered.

D. Soil Investigation and Sampling by Auger Borings
ASTM D-1452

This method consists of augering a hole and removing representative soil samples from the auger flight or bit at 5 foot depth intervals or with each change in substrata. Disturbed sampled are obtained and this method is, therefore, limited to situations where it is satisfactory to determine the approximate subsurface profile.

E. Diamond Core Drilling for Site Investigation
ASTM D-2113

This method consists of advancing a hole into hard strata by rotating a single or double tube core barrel equipped with a cutting bit. Diamond, tungsten carbide, or other cutting agents may be used for the bit. Wash water is used to remove the cuttings and cool the bit. Normally, a 2 inch outside diameter by 1-3/8 inch inside diameter (NX) coring bit is used unless otherwise noted. The rock or hard material recovered within the core barrel is examined in the field and in the laboratory and the cores are stored in partitioned boxes. The core recovery is the length of the material recovered and is expressed as a percentage of the total distance penetrated.

F. Visual – Manual Soil Classification Procedure
ASTM D-2488

This procedure is a visual – manual soil classification methodology for the description of soil for engineering purposed when precise soils classification is not required.

LABORATORY TESTING

A. Atterberg Limits: Liquid Limit, Plastic Limit and Plasticity Index of Soils
ASTM D-4318, TEX 104-E, 105-E and 106-E

Atterberg Limits determine the soil's plasticity characteristics. The soil's Plasticity Index (PI) is representative of this characteristic and is the difference between the Liquid Limit (LL) and the Plastic Limit (PL). The LL is the moisture content at which the soil will flow as a heavy viscous fluid. The PL is the moisture content at which the soil begins to lose its plasticity. The test results are presented on the boring logs adjacent to the appropriate sampling information.

B. Particle Size Analysis of Soils
ASTM D-422 and TEX 110-E

Grain size analysis tests are performed to determine the particle size and distribution of the samples tested. The grain size distribution of the soils coarser than the Standard Number 200 sieve is determined by passing the sampled through a standard set of nested sieves.

C. Laboratory Determination of Water (Moisture) Content of Soil and Rock
ASTM D-2216 and TEX 103-E

The moisture content of soil is defined as the ratio, expressed as a percentage, of the weight of water in a given soil mass to the weight of solid particles. It is determined by measuring the wet and oven dry weights of a soil sample. The test results are presented on the boring logs.

D. Unconfined Compressive Strength of Cohesive Soil
ASTM D-2166

The unconfined compressive strength of soil is determined by placing a section of an undisturbed sample into a loading frame and applying an axial load until the sample fails in shear. The test results are presented on the boring logs adjacent to the appropriate sampling information.

E. California Bearing Ratio (CBR) of Lab Compacted Soils
ASTM D-1883

The CBR test is performed by compacting soil in a 6 inch diameter mold at the desired density, soaking the sample for four days under a surcharge load approximating the pavement weight and then testing the soils in punching shear. A 2 inch diameter piston is forced into the soil to determine the resistance to penetration. The CBR is the ratio of the actual load required to produce 0.1 inches of penetration to that producing the same penetration in a standard crushed stone.

F. Swell Test
ASTM D-4546

The swell test is performed by confining a 1 inch thick specimen in a 2-1/2 inch diameter stainless steel ring and loading the specimen to the approximate overburden pressure. The test specimen is then inundated with distilled water and allowed to swell for 48 hours. The volumetric swell is measured as a percentage of the total volume and is converted mathematically to linear swell.

G. Compaction Tests
ASTM D-698, D-1557, TEX 113-E or 114-E

The compaction test is performed by compacting soil in a steel mold at varying moisture contents. Layers are compacted using a hammer weight and number of blows per layer which vary with the different test procedures. ASTM D-698, D-1557, TEC 113-E and 114-E. The data is plotted and the maximum weight and optimum moisture content is determined.

H. Classification of Soils for Engineering Purposes
Unified Soil Classification System, D-2487

This standard describes a system for classifying mineral and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index and shall be used when precise classification is required.

RECOMMENDED SPECIFICATIONS FOR PLACEMENT OF SELECT FILL

1. General

The soils engineer shall be the owners representative to control the placement of compacted fill. The soils engineer shall approve the subgrade preparation, the fill materials, the method of placement and compaction, and shall give written approval of the completed fill.

2. Preparation of Existing Ground

All topsoil, plants and other organic material shall be removed. The exposed surface shall be scarified, moistened if necessary, and compacted in the manner specified for subsequent layers of fill.

3. Select Fill Material

Fill shall have a liquid limit of less than 35 and a Plasticity Index between 8 and 18. The fill shall contain no organic material or other perishable material, and no stones larger than 6 inches. Fill material shall be approved by the soils engineer.

4. Placing Select Fill

Fill materials shall be placed in horizontal layers not exceeding 8 inches thickness after compaction. Successive loads of material shall be dumped so as to secure even distribution, avoiding the formation of layers of lenses of dissimilar materials. The contractor shall route hill hauling equipment to distribute travel evenly over the fill area.

5. Compaction of Select Fill

a. Moisture Control: The moisture content of the fill material shall be distributed uniformly throughout each layer of the material. The allowable range of moisture content during compaction shall be within plus two (+2) and minus two (-2) percentage points of the optimum moisture content. The contractor may be directed to add necessary moisture to the material either in the borrow area or upon the fill surface or to dry the material, as directed by the soils engineer. The drying of cohesive soils between lifts to moisture contents less than 70% of optimum before the placement of subsequent lifts shall be avoided or the fill reworked at the proper moisture content.

b. Compaction: The material in each layer shall be compacted to obtain proper densities. Compaction by the hauling equipment alone will not be considered sufficient. Structural fills, including pavement subgrade, subbase, and base, shall be compacted to densities equivalent to the percentages of the Standard Proctor (ASTM D-698) or Modified Proctor (ASTM D-1557) maximum dry density listed in the table below. The Texas Department of Highways and Public Transportation Method TEX 113-E or TEX 114-E compaction test, which varies the compactive effort with soil type, may be substituted for the Standard or Modified Proctor methods and percentages listed in the table below.

	PERCENT COMPACTION	
Area	Fine Grained Soils ASTM D-698 (Standard) or TEX 114-E	Coarse Grained Soils ASTM D-1557 (Modified) or TEX 113-E
Within five (5) feet of building lines, under footings, floor slabs, slab-on-grade foundation and structures attached to the building (i.e. walls, patios, steps)	95	95+
More than five (5) feet beyond building lines, under walks, and fill area to be landscaped	90	90
Pavement subgrade and subbase, including lime treated soils	95	95+

Soils classified as coarse grained soils are those with more than 50%, by weight, retained on the No. 200 Standard Sieve.

6. Comparison Testing

Field density tests for the determination of the compaction of the fill shall be performed by TSI Laboratories, Inc. in accordance with recognized procedures for making such tests. A representative number of tests shall be made in each compacted lift at locations selected by the soils engineer or his/her representative. For general structural and paving fills, we suggest one test per 3,000 square feet per lift with a minimum of three tests per lift.

IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

The following observations and suggestions are provided to help you better utilize your geotechnical engineering report and to reduce construction problems and delays related to the soil and groundwater conditions.

REPORT IS BASED UPON SPECIFIC SITE AND PROJECT

A geotechnical report is based on a subsurface exploration conducted on a specific site and planned using specific project information. The project information typically includes structure size and configuration, type of construction, and general location on the site. Limitations, such as existing buildings or utilities, specific foundation requirements for structures, budget limitations, and the level of risk assumed by the client may affect the scope of the exploration.

Since the report applies to a specific structure and site, the geotechnical report should not be used in the following circumstances unless the geotechnical engineer has reviewed the changes and concurs in the use of the report.

- When the nature of the proposed structure is changed, such as an office building instead of a warehouse or parking garage, or a refrigerated warehouse instead of one which is not refrigerated
- When the size, configuration, or floor elevations is changed
- When the location of the structure on the site is changed
- When there is a change of ownership

FINDINGS ARE PROFESSIONAL ESTIMATES

The actual subsurface conditions are determined only at the boring locations and only at the time the samples are taken. The information is extrapolated by the geotechnical engineer who then renders professional opinions regarding the characteristics of the subsurface materials, the behavior of the soils during construction, and appropriate foundation designs. No exploration, however complete, can be assured of sampling the entire range of soil conditions. The soils may vary between or beyond the borings and stratum transitions may be more gradual or more abrupt, and all types of oils and rock existing on the site may not be found in the borings. The geotechnical engineer is often retained during construction to evaluate variances and recommend solutions to problems encountered on the site.

SUBSURFACE CONDITIONS CAN CHANGE

Grading operations on or close to the site, floods, groundwater fluctuations, utility construction, and utility leaks are among the events that can change the subsurface conditions. The geotechnical engineer should e kept apprised of such events.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

A geotechnical report may have been made to evaluate foundation alternatives only, for preliminary site evaluation, or for other limited purposes. The exploration may also have been limited by the direction of the client, budget limitations, or the level of risk assumed by the client. Therefore, no one other than the original client should use the report for its intended purpose or other purposes without conferring with the geotechnical engineer.

GEOTECHNICAL REPORTS ARE SUBJECT TO MISINTERPRETATION

Geotechnical reports are based on the project information available at the time the report was made and the judgment and opinions of the geotechnical engineer. This specialized information is subject to misinterpretation by other design professionals, contractors and owners. The geotechnical engineer should be retained during the design process to interpret the recommendations and review the adequacy of the plans and specifications relative to geotechnical issues. The boring logs should no be separated from the geotechnical report, but, rather the entire report should be made available to the contractors and others needing this information.

TSI LABORATORIES, INC.

GEOTECHNICAL ENGINEERING STUDY
SUPPLEMENTAL REPORT

Calhoun County-GLO-CDBG-MIT
3rd Street & 4th Street
Seadrift, TX

1810 S. Laurent St. * Victoria, TX 77901 * 361-578-6933

TSI LABORATORIES, INC.
TBPE FIRM REGISTRATION NO: F-9236

1810 SOUTH LAURENT
VICTORIA, TEXAS 77901

Telephone 361-578-6933
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Email: tsilabvictoria@gmail.com

May 19, 2023

G & W Engineers
Scott Mason
205 W. Live Oak
Port Lavaca, TX 77979

Subject: Calhoun County-GLO-CDBG-MIT
3rd Street & 4th Street
Seadrift, TX

TSI File No.: V-231109

Dear Mr. Mason,

We are pleased to submit this supplemental report of our geotechnical engineering study for additional borings for the Calhoun County-GLO-CDBG-MIT on 3rd Street & 4th Street in Seadrift, TX. The findings and a description of the exploration and testing procedures are presented in the report along with our site preparation recommendations.

We appreciate the opportunity to assist in this phase of the project. Please feel free to contact us if you have any questions regarding this report or if we may be of further service.

Respectfully submitted,

TSI Laboratories, Inc.

STATE OF TEXAS
DANIEL TESFAI
93944
LICENSED
PROFESSIONAL ENGINEER

Michael Tater, President.

Daniel Tesfai, P.E.

GEOTECHNICAL ENGINEERING STUDY
SUPPLEMENTAL REPORT

Calhoun County-GLO-CDBG-MIT
3rd & 4th Street
Seadrift, Texas

Prepared For:

Scott Mason
G&W Engineers, Inc.

Prepared By:

TSI LABORATORIES, INC.
TBPE Firm Registration No.: F-9236

Victoria, Texas

May 19, 2023

TSI Project Number: V-231109

TSI LABORATORIES, INC.

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Seadrift, TX

INTRODUCTION

Authorization and Scope

TSI Laboratories, Inc. (TSI) was retained to provide geotechnical study services for additional borings by Scott Mason with G&W Engineers, Inc. The purpose of this study was to determine and evaluate the stratification and engineering properties of the site subsurface soils. TSI will provide geotechnical engineering recommendations and guidelines as a supplemental report for use in site preparation, foundation design, and related site improvements planned for the Calhoun County GLO-CDBG–MIT Project located on 3rd & 4th Street in Seadrift, Texas. This supplemental report shall be used in conjunction with the original report, identified as TSI Project No. V-221264.

Project Description

The proposed project involves taking additional borings for the design and construction of bridges, a retaining wall and pavement design for rebuilding 3rd and 4th street. Based on the results of the study the bridge and retaining wall could be supported by straight shaft or driven pile foundation system.

FIELD AND LABORATORY TESTING

Field Testing

The site soil was explored by drilling additional three (3) 50-foot-deep borings. Boring locations were determined by the client and shown on the respective site plan. Soil was sampled continuously at 2-foot intervals to 10-foot depth with an additional sample taken at 5-foot depth intervals. The sampling method is determined based on the encountered soils.

Cohesive soils were sampled by hydraulically pushing a 3-inch diameter, thin-walled steel tube a distance of about 24-inches. Our field sampling procedures were in general accordance with ASTM D1587. For each recovered sample, our representative extruded the sample in the field, visually classified the soil, and measured the penetration resistance using a pocket penetrometer.

Granular soils were sampled as part of the Standard Penetration Test (SPT) by driving a 2-inch diameter split-barrel sampler. The sampler was driven 18-inches by a 140-pound hammer falling 30-inches in general accordance with the ASTM D1586. Our representative recorded the number of blows required to drive the sampler through three consecutive 6-inch intervals. As permitted by ASTM D1586, sampling was terminated if 50-blows were recorded within any single 6-inch interval. The sum of blows required to penetrate the final 12-inches is known as the SPT "N" value. A portion of the recovered sample was wrapped in aluminum foil and placed into a sample container and transported to our laboratory for testing.

Laboratory Testing

The soil samples selected for laboratory testing were examined and visually classified by the sample's representative of the various soil strata encountered. Atterberg limits, moisture contents and percent fines tests were performed to assist in classifying the soils according to Unified Soil Classification System (ASTM D2487). Unconfined compressive strength tests and direct shear tests were also performed to provide indicators of soil strength. The classification test results are presented on the boring logs. The test procedures are described in the Appendix.

SUBSURFACE CONDITIONS

Soil classifications are described in detail in the Boring Logs provided in the Appendix and summarized in the table below.

Boring No.	Soil Type				Ground water table
B-1	0-28' SC	33-50' CH/CL			8'
B-2	10.0" (GC)	1-6' CL	6-23' SM	23-50' CL/CH	12'
B-3	0-28' SC	28-43' CL	43-50' SC		12'

The site soil has been evaluated by performing various field and laboratory tests on the subsurface samples recovered during the drilling operations. The types of tests conducted on the subsurface samples and the results of the tests are tabulated on the Logs of Borings, which are provided in the Appendix. The properties of each stratum are discussed below.

The corresponding boring logs, depicting the stratum soil descriptions, type of sampling used during sample retrieval, laboratory test data, and other field data, is presented in the Appendix at the end of this report. The key to the boring log symbols and soil classifications Sheet, which defines the terms and descriptive symbols used on each boring log, is also presented in the Appendix.

RECOMMENDATIONS

The foundation system for the proposed bridge and retaining wall must satisfy two (2) independent engineering criteria with respect to the soil conditions. First, the foundation system should be designed with an appropriate factor of safety against bearing capacity failure of the foundation soils. Second, the movement of the foundation system due to compression (consolidation) or expansion (swell) of the soils supporting the foundation system must be within tolerable limits for the structure.

Minimum Embedment Requirement

We recommend that the design of the shafts or piles consider the following minimum embedment requirements:

- The shafts or piles should have a depth of embedment adequate for support of the imposed axial and lateral loads.
- Computation of design capacities of the shafts or piles should provide for reductions in capacities due to construction-related disturbance and shrink-swell of surficial soils with changes in moisture.
- Shafts or piles should penetrate into the base stratum (the stratum in which the shaft or pile tip is placed) at least two (2) times their diameter or width.

Drilled Straight-Shafts

Drilled straight shafts may be utilized to support the proposed bridge. The drilled shafts if extended to a **depth of 15 to 28-foot from the existing grade**, the shafts should be sized for a net total load bearing pressure of 4.4 ksf, whichever condition results in a larger bearing surface. However, if extended below 28-foot and below a net total load bearing pressure of 5.3 ksf. The same value can also be used for the cast in place box culvert with 15-inches thick perimeter beams.

The shafts should contain sufficient vertical reinforcing steel throughout the entire shaft length to resist uplift (tensile) forces due to post-construction heave of the clayey soils. The magnitude of uplift is difficult to predict and will vary with in-situ moisture contents. For purposes of establishing sufficient reinforcing to resist uplift, the uplift pressures can be approximated by using a uniform uplift pressure of 475 psf over the perimeter of the shaft embedded. The amount of reinforcing steel required can be computed by assuming the dead load of the structure surcharges the shaft, that the above estimated tensile force acts vertically on the shaft, and that the shaft embedment acts as a rigid anchor. However, in no case should the percentage of steel be less than 0.5% (based on 40 ksi steel).

Drilled shaft edge-to-edge spacing of less than two (2) shaft diameters will require axial capacity reduction. TSI should be contacted for additional recommendations if the clear spacing between drilled shafts is less than two shaft diameters.

An allowable side shear value of 1 ksf with an assumed factor of safety of at least two (2) may be used to aid in resisting axial compressive loads on the piers. The side shear should be neglected for fill material, the upper 5-foot of soil in contact with the pier shaft, and within one (1) pier diameter of the bottom of the shaft.

Driven Piles

Driven concrete piles may be used to support the proposed bridges. The allowable unit skin and end bearing capacities provided in the following table are recommended for the design of full displacement driven piles. The friction factor defines the increase in pile friction capacity as a function of depth. These values include a factor of safety of 2. Allowable tension can be taken as 80% of the friction value. The end bearing factor can be used to estimate end bearing capacity. These values include a factor of safety of 2.5.

Penetration[1] (foot)	Allowable Unit Skin Friction (psf)	Allowable Unit End Bearing Pressure[2] (psf)
0 to 5	Disregard	
6 to 13	230	Disregard
14 to 28	320	3,190
29 to 50	375	3,760

[1] Penetration below grade existing at the time of field investigation.
[2] Neglect for pile dimension less than 2-foot

The parameters required for p-y curves and designs from the boring logs are listed in the table below:

Soil Type	γ	C	ϕ	K	K_o	$^*E_{50}$
Medium Dense Silty Sand (SM)	115	0	22	62	0.63	-
Stiff Clay (CH)	105	1,300	0	132	0.55	0. 007

* Values estimated from known correlations.

Where:
γ = Wet Unit Weight, pcf
c = soil cohesion, psf
ϕ = Angle of internal Friction, deg
K_o = Lateral Earth Pressure Coefficients, At-Rest
K = modulus of subgrade cyclic reaction (pci)

Driven piles generally derive most of their load carrying capacity from skin friction. Therefore, end bearing is normally negligible for driven piles. We recommend the allowable unit end bearing values presented in the above table should be neglected if the selected driven pile dimension is less than 2-foot. In addition, the allowable unit skin friction and unit end bearing values provided in the above table are recommended for driven concrete (full displacement) piles. TSI should be contacted for additional recommendations if driven piles other than full-displacement concrete piles are planned to be utilized at this site.

Lateral resistance of driven piles is primarily developed by passive resistance of the soil against the side of the pile. A detailed lateral load analysis of the proposed piles was beyond the scope of this study. If requested, a detailed lateral capacity analysis of the proposed driven piles can be provided for this project.

Design of piles should also include an evaluation of the structural capacity of the pile which may limit the allowable capacity. Any pile splices must provide positive load transfer both in compression and tension since driving displacement piles within clayey soils could result in heave. As the pile is driven, it displaces soil upward toward the surface. This upward soil movement can "drag" the adjacent piles up and lift them off of their bearing layer causing tension along the piles.

Pile groups subjected to axial loads can be influenced by numerous factors which may include pile type, size and length, pile spacing, overall group size, loading conditions, installation procedures, and soil type and strength. With a center-to-center spacing of at least three (3) pile widths, the group effect would be insignificant on the load carrying capacity of the piles. Therefore, the combined axial load capacity for such a group may be taken as the sum of the individual pile capacities in the group.

Post construction settlements of single isolated piles will depend on the elastic properties of the pile, the applied load, and the interaction of the soil and pile. Settlement is anticipated to be primarily elastic and will occur relatively rapidly as load is applied. Significant consolidation settlement due to applied load is not anticipated at this site for the pile capacities given. Our experience indicates that single, isolated piles loaded to about one-half of their ultimate capacity should experience settlement of less than one inch.

Post construction settlements of groups of piles are generally greater than single isolated piles for the same load per pile. Based on the previously recommended spacing, we anticipate that settlement of the pile group should be one inch or less under working loads. In general, differential settlements should be on the order of one-half to two-thirds of the total settlement.

The installation of the piles should preferably be accomplished by driving alone. However, predrilling, or controlled jetting may be required to achieve the design penetrations if excessive resistance to penetration occurs during driving. We anticipate that predrilling may be necessary to achieve penetration to any appreciable depth into the sandy soils observed at this site. The effects and methods of pile installation should be given proper consideration when choosing and designing pile foundation systems. In most situations, the greatest stress a driven pile will experience is during installation. Pile and soil properties, embedment length requirements, and driving equipment are only a few of the many variables to consider in determining the most efficient method of pile installation.

Driving piles to completed embedment depths may be facilitated by predrilling to a depth somewhat less than the anticipated final embedment depth. Under no circumstances should predrilling extend deeper than 5-foot above the final pile embedment depth unless refusal occurs. The predrilled excavation should be about 4-inches less in width than the pile size to promote the development of skin friction resistance. Extreme care should be exercised during predrilling since it can affect the lateral and axial capacities of the pile.

Production piles should be driven to a predetermined (design) depth with blow count as a secondary consideration. Because set-up during interruption can produce increased resistance to driving, a pile should be driven to its design depth without any delays, if possible.

If a pile exhibits a resistance lower than the terminal resistance values given by driving formulae at an appreciable depth below the predetermined depth, the pile may be re-tapped after a suitable elapsed time and after the installation of other nearby piles. The re-tap should be performed at the contractor's discretion to prove the acceptability of the pile. Should, in the Geotechnical Engineer's judgment, the re-tap not indicate adequate capacity, a new pile should be installed to provide the required capacity.

In addition, we recommend the use of a "Pile Driving Analyzer" (PDA) during pile installation. The PDA can monitor driving stresses and hammer energy during pile installation, and also provides a continuous record of the pile installation. Such information can be beneficial in evaluation of the acceptability of a driven pile.

Settlement Considerations

Total settlements, based on the indicated bearing pressures, should be about 1-inch for properly designed and constructed drilled piers. Settlement beneath individual piers will be primarily elastic with most of the settlement occurring during construction. Differential settlement may also occur between adjacent piers. The amount of differential settlement could approach 50 to 75% of the total pier settlement. For properly designed and constructed piers, differential settlement between adjacent piers is estimated to be less than ¾-inch. Settlement response of drilled piers is impacted more by the quality of construction than by soil structure interaction.

Improper pier installation could result in differential settlements significantly greater than we have estimated. In addition, larger magnitudes of settlement should be expected if the soil is subjected to bearing pressures higher than the allowable values presented in this report.

Sheet Pile Recommendations

Retaining structures must be designed in a way such that serviceability and ultimate limit states are not reached. Serviceability limit states are mostly associated with excessive deflection of the structures toward the open space and away from the ground they support. This leads to a corresponding loss of ground behind the retaining structure; if a structure is present there, it may be damaged by the resulting settlements. Ultimate limit states include bearing capacity failure, sliding, overturning and general stability of the retaining structure and the soil it supports. Additionally, the retaining structure has to retain structural integrity through its useful life.

The sheet piles can be installed by vibratory hammers as sand liquefaction due to the vibration aids the rate of advance. For short sheet piles, jacking can be quite efficient, as agile, hydraulic machines now exist that can quickly push sheet piles into the ground.

Lateral Earth Pressure

Below-grade walls or retaining structures may be used for some structures in the facility. The walls will be subject to lateral earth pressures from a combination of soil pressure, hydrostatic water pressure, and surcharge loads. The earth pressure, σh, for soils adjacent to the below-grade walls, is expected to approach at-rest conditions and may be computed as:

$\sigma h = K_o \gamma H$

where K_o = At-rest earth pressure coefficient = 0.53
γ = Unit weight of the adjacent soil, lb./ft^3= 56
H = Wall height, foot

Lateral earth pressures resulting from the soil are calculated by multiplying the equivalent fluid density of the surrounding soils, defined as $K_o\gamma$, by the depth below the ground surface. For water pressures, multiply the unit weight of water by the depth below the ground surface, finished grade, or 100-year flood elevation, whichever is greater.

The equivalent fluid densities of 80 pcf for moist and 37 pcf for submerged can be used. These values do not include a hydrostatic pressure component. The sum of the pressures resulting from soil and water, acting as a triangular distribution, should be used for the wall design.

Engineering Design Manuals often specify that lateral loads due to surcharge loading from cranes and H-20 trucks shall be included. Lateral earth pressures from uniformly distributed surcharge loads can be calculated by using a rectangular stress distribution of the imposed vertical load multiplied by the appropriate lateral earth pressure coefficient. For this reason, a lateral earth pressure coefficient of 0.52 to 0.65 may be used.

If there is movement of sheet pile bulkhead during its service life, both active and passive pressures will be mobilized. Based on existing surface at time of drilling operations, coefficients of 0.56 and 1.82 up to 13-foot depth, 0.47 and 2.12 between 14 to 28-foot depth and below that 0.35 and 2.85 may be used for calculation earth pressures for Active and Passive Rankine earth pressure, respectively.

A surcharge imposed on the soil adds to the lateral earth pressure exerted against the retaining wall due to the loading on the piling of the mooring structure. This added pressure must, be considered in the design, and can be computed as:

$P' = qHK_a$

Where q = surcharge load

H = height of the wall

K_a = coefficient of active earth pressure = $(1-\sin\phi) / (1+\sin\phi)$

Below-grade walls should be checked against failure due to overturning, sliding, and overall slope stability. Such analysis can only be performed during a detailed study once the dimensions of the bulkheads are known.

The boring logs indicate that the soil conditions encountered should not pose any difficulty to the dredging contractor. The maximum side slope conditions should not exceed two (2) horizontals to one (1) vertical ratio.

Foundation Construction

<u>Drilled Straight-Shafts</u>

The drilling contractor should be experienced in the subsurface conditions observed at the site, and the excavations should be performed with equipment capable of providing clean bearing area, free of water. Drilled straight-shaft foundations should be installed in general accordance with the procedures presented in "Drilled Shafts: Construction Procedures and Design Methods," by Reese, L. C. and O'Neill, M. W., FHA Publication No. FHWA-IF-99-025, 1999 and "Standard Specification for the Construction of Drilled Piers", ACI Publication No. 336.1-01, 2001.

Foundation installation should be closely monitored by a qualified technician experienced in drilled straight-shaft installation techniques. At a minimum, the technician should monitor shaft excavation, note any unusual installation occurrences, monitor concrete placement, and generally evaluate if foundation installation is being performed in accordance with the project specifications.

As stated previously, groundwater was observed in all the borings during drilling. Based on the subsurface and groundwater conditions observed at the borings, the installation of drilled straight-shafts will require the use of temporary steel casing. We recommend that provisions be incorporated into the plans and specifications to utilize casing to control sloughing and/or groundwater seepage during shaft construction. To evaluate the constructability of drilled straight-shafts and the potential variability of groundwater conditions, we recommend at least two test shafts prior to the installation of production shafts. The installation of test shafts should be observed by TSI.

If casing is used and seepage persists, the water accumulating in the foundation excavation should be pumped out. The condition of the bearing surface should be evaluated immediately prior to placing concrete. Where casing is used, removal of the casing should be performed with extreme care and under proper supervision to minimize mixing of the surrounding soil and water with the fresh concrete. Rapid withdrawal of the casing may develop suction that could cause the soil and water to flow into the excavation. An insufficient head of concrete in the casing during withdrawal could also allow the water to intrude into the wet concrete. The casing must be removed in order to utilize the skin friction values previously provided. Under no circumstances should loose soil be placed in the annulus between the casing and the drilled shaft sidewalls.

Driven Pile Foundation Installation

Piling should be installed in accordance with TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, 2014, Items 404 and 409. If piles are to be installed to any appreciable depth, a pile drivability analysis should be performed. A drivability analysis will help in evaluating the pile-hammer combination best suited for pile installation, reducing the need for installation aids, and reducing the risk of pile damage resulting from excessive driving stresses.

Foundation Construction Monitoring

The performance of the recommended foundation systems for the proposed structures will be highly dependent upon the quality of construction. Thus, we recommend that foundation installation be monitored full time by an experienced TSI soil technician under the direction of our geotechnical engineer.

GENERAL COMMENTS

TSI should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. TSI also should be retained to provide testing and observation during excavation, grading, foundation, and construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, and 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.

For any excavation construction activities at this site, all Occupational Safety and Health Administration (OSHA) guidelines and directives should be followed by the Contractor during construction to insure a safe working environment. In regard to worker safety, OSHA Safety and Health Standards require the protection of workers from excavation instability in trench situations.

This report has been prepared for the exclusive use of our client for the specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either expressed or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless TSI reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX

Boring Locations Map

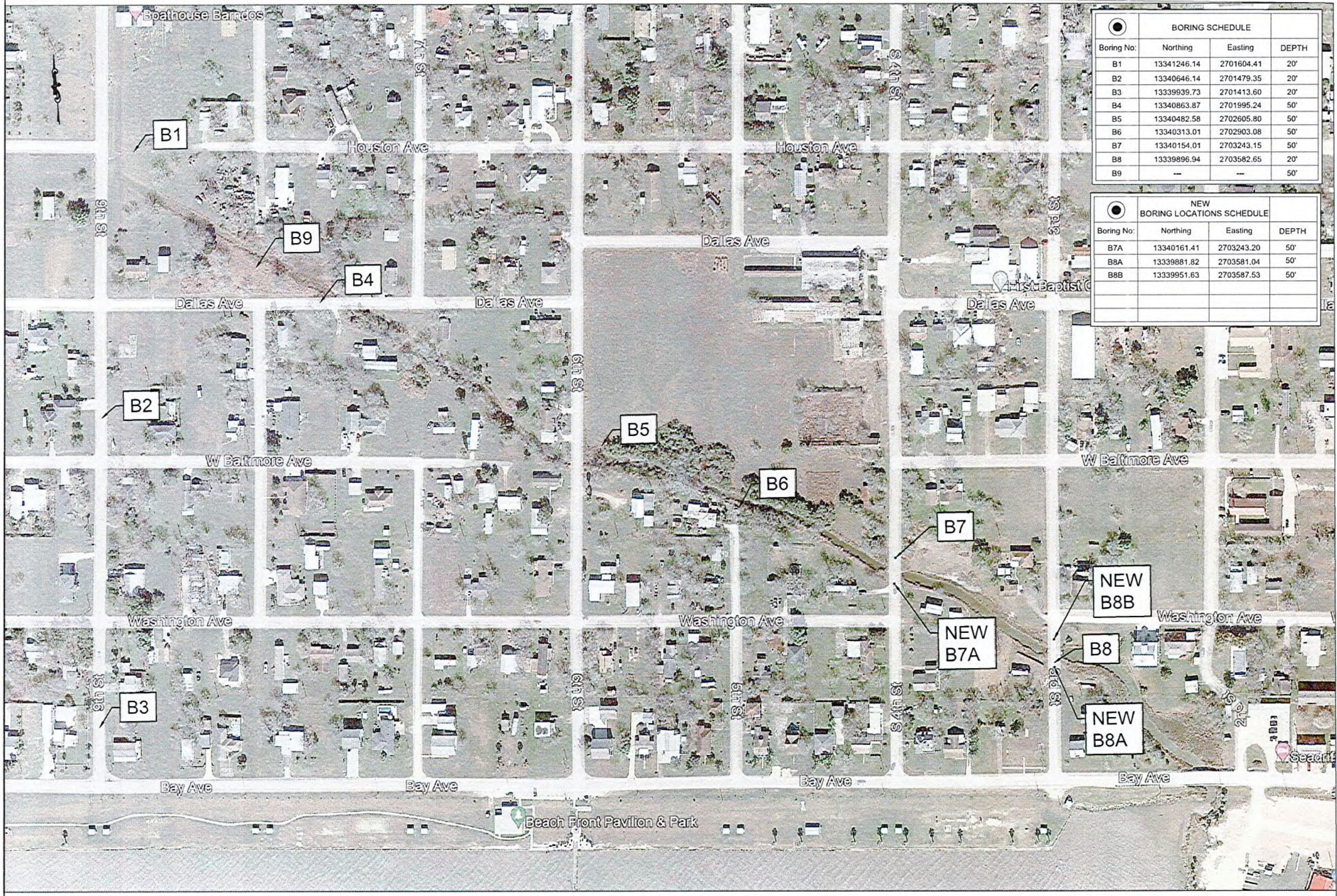
Log of Borings

Laboratory Test Results

Symbols and Terms Used on Boring Log

Field and Laboratory Testing Procedures

Important Information About Your
Geotechnical Engineering Report



BORING SCHEDULE			
Boring No:	Northing	Easting	DEPTH
B1	13341246.14	2701604.41	20'
B2	13340646.14	2701479.35	20'
B3	13339939.73	2701413.60	20'
B4	13340863.87	2701995.24	50'
B5	13340482.58	2702605.80	50'
B6	13340313.01	2702903.08	50'
B7	13340154.01	2703243.15	50'
B8	13339896.94	2703582.65	20'
B9	---	---	50'

NEW BORING LOCATIONS SCHEDULE			
Boring No:	Northing	Easting	DEPTH
B7A	13340161.41	2703243.20	50'
B8A	13339881.82	2703581.04	50'
B8B	13339951.63	2703587.53	50'

REV.	DATE	DESCRIPTION	BY	CHK.
B	02/16/23	ADDED NEW BORING LOCATIONS	BD	MJ
A	12/22/22	30% SET FOR REVIEW ONLY	BD	MJ

Texas Registered Engineering Firm F-04188

G & W ENGINEERS, INC.

ENGINEERING SURVEYING CONSULTING PLANNING

205 W. Live Oak St. Port Lavaca, Texas 77979 (361) 552-4509

1801 7th St., Suite 260 Bay City Texas 77414 (979) 323-7100

DRAINAGE IMPROVEMENTS PROJECT

GLO-CDBG-MIT NO. 22-085-014-D245

CALHOUN COUNTY, TEXAS

BORING LOCATION MAP

THIS DOCUMENT IS PRELIMINARY AND WAS RELEASED FOR THE PURPOSE OF REVIEW UNDER THE AUTHORITY OF MARLA M. JASEK, P.E., TEXAS NO. 83938 ON FEBRUARY 15, 2023.

IT IS NOT TO BE RELIED UPON AS A COMPLETE ENGINEERING DOCUMENT

FOR REVIEW ONLY
NOT FOR CONSTRUCTION

PROJECT NO. 5310.013A

SCALE: 1"=100'-0"

DRAWN BY: STC

CHK'D BY: SM

DATE: 06/16/2022

SHT. NO. B1.0

Log of Boring

PROJECT: Calhoun County-GLO-CDBG-MIT
3rd & 4th Street
Seadrift, TX

CLIENT: G&W Enginneers

BORING NO.: **B8-A/**B-1
PROJECT NO.: V-231109
DATE: 04/04/23
SURFACE ELEV.: N/A
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-50.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 8.0'

FIELD DATA		LABORATORY DATA								
SAMPLE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Atterberg Limits % Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
Disturbed Sample										CLAYEY SAND - with gravel, dark brown (SC)
Split Spoon Sample	N=6									- dark gray (SC)
Steel Tube Sample	P=3.5	23	101	34	14	20	48	1.1	15.0	
Steel Tube Sample	P=2.5	16		33	16	17	36			- brown (SC)
Steel Tube Sample	P=2.5	26		17	12	5	22			SILTY CLAYEY SAND - dark gray (SC-SM)
Split Spoon Sample	N=5									CLAYEY SAND - light gray (SC)
Split Spoon Sample	N=7									- dark gray (SC)
Split Spoon Sample	N=24									- brown (SC)
Split Spoon Sample	N=47	24		54	18	36	70			SANDY FAT CLAY - brown and gray (CH)
Split Spoon Sample	N=71									
Split Spoon Sample	N=73									
Split Spoon Sample	N=70	25		42	16	26	70			SANDY LEAN CLAY - brown and gray (CL)
Split Spoon Sample	N=82									- brown (CL)
										Boring terminated at 50.0'

DEPTH (FEET): 0, 8, 16, 24, 32, 40, 48, 56

Steel Tube Sample
Split Spoon Sample
Disturbed Sample

REMARKS:

TSI
Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County-GLO-CDBG-MIT
3rd & 4th Street
Seadrift, TX

CLIENT: G&W Enginneers

BORING NO.:B8-B/B-2
PROJECT NO.: V-231109
DATE: 04/04/23
SURFACE ELEV.: N/a
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-50.0'

GROUNDWATER INFORMATION:
Groundwater was encountered at 12.0'

DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT, T : INCH/100 BLOWS, P : TONS/SQ. FT., R : PERCENT, RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Atterberg Limits % Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0	Disturbed			7		26	12	14	18			CLAYEY GRAVEL - with sand, brown and dark brown (GC)
	Steel Tube		P=3.5									SANDY LEAN CLAY - dark brown (CL)
	Steel Tube		P=3.0	25	96	39	14	25	53	1.1	8.0	- light gray (CL)
	Steel Tube		P=2.5									
8	Split Spoon		N=3	27		17	17	0	12			SAND - with silt, light gray (SP-SM)
	Split Spoon		N=4	28		19	19	0	16			SILTY SAND - light gray (SM)
16												
	Split Spoon		N=8									
24	Split Spoon		N=25	23		27	13	14	65			SANDY LEAN CLAY - reddish brown (CL)
	Split Spoon		N=46									FAT CLAY - reddish brown (CH)
32												
	Split Spoon		N=67									
	Split Spoon		N=68									LEAN CLAY - with sand, reddish brown (CL)
40												
	Split Spoon		N=70									
48	Split Spoon		N=80									- brown (CL)
												Boring terminated at 50.0'
56												

Steel Tube Sample

Split Spoon Sample

Disturbed Sample

REMARKS:

TSI Laboratories, Inc.

Log of Boring

PROJECT: Calhoun County-GLO-CDBG-MIT
3rd & 4th Street
Seadrift, TX

CLIENT: G&W Enginneers

BORING NO.:B7-A/B-3
PROJECT NO.: V-231109
DATE: 04/04/23
SURFACE ELEV.: N/a
LAB. NO.: L-001

DRILLING METHOD(S) : Dry Auger 0-50.0'

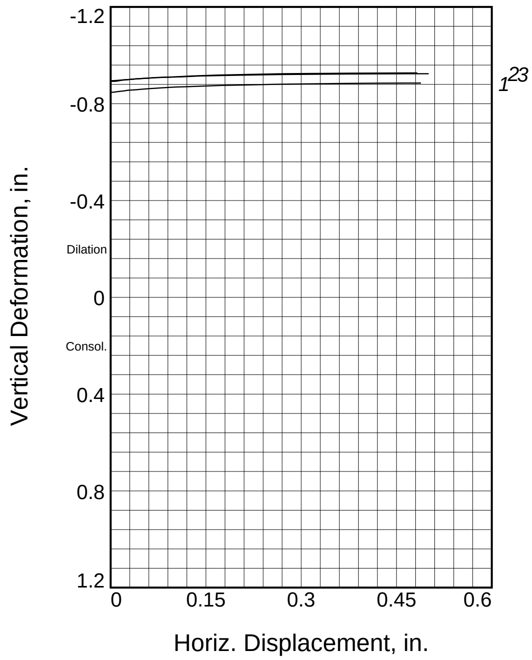
GROUNDWATER INFORMATION:
Groundwater was encountered at 12.0'

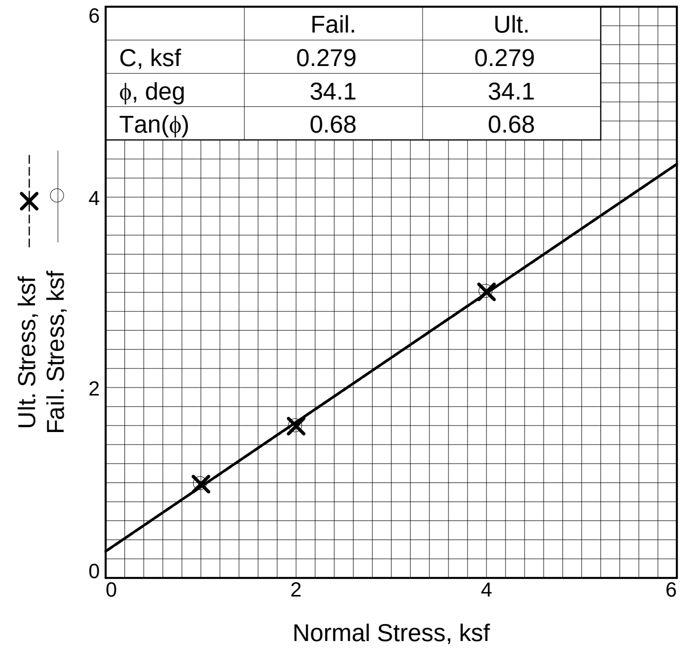
DEPTH (FEET)	SAMPLE	SOIL TYPE	N : BLOWS/FT T : INCH/100 BLOWS P : TONS/SQ. FT. R : PERCENT RQD : RATIO	MOISTURE CONT. %	DRY DENSITY pounds/ft. 3	Atterberg Limits % Liquid Limit	Plastic Limit	Plasticity Index	MINUS No. 200 SIEVE (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN %	DESCRIPTION OF STRATUM
0	Disturbed											CLAYEY SAND - with gravel and shell, dark gray (SC)
	Steel Tube		P=3.5	11	115	29	13	16	32			
	Steel Tube		P=3.5									
	Steel Tube		P=3.5	17		20	15	5	28			SILTY CLAYEY SAND - light brown (SC-SM)
8	Steel Tube		P=3.0									- brown (SC-SM)
	Split Spoon		N=4	26		35	15	20	33			CLAYEY SAND - light gray (SC)
16												
	Split Spoon		N=7	27		18	18	0	13			SILTY SAND - light gray (SM)
24	Split Spoon		N=23									- reddish brown (SM)
	Split Spoon		N=41									SANDY LEAN CLAY - reddish brown (CL)
32												
	Split Spoon		N=62	23		37	14	23	67			
40	Split Spoon		N=66									
	Split Spoon		N=71									CLAYEY SAND - with gravel, brown (SC)
48	Split Spoon		N=81	27		44	17	27	43			
												Boring terminated at 50.0'
56												

Steel Tube Sample
Split Spoon Sample
Disturbed Sample

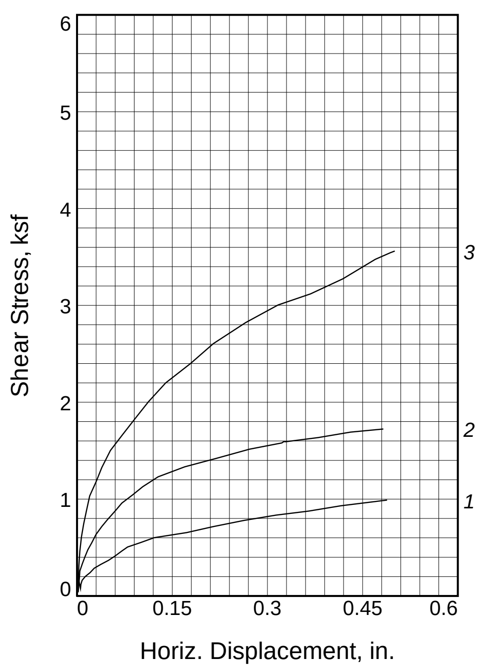
REMARKS:

TSI Laboratories, Inc.





	Fail.	Ult.
C, ksf	0.279	0.279
φ, deg	34.1	34.1
Tan(φ)	0.68	0.68



	Sample No.	1	2	3
Initial	Water Content, %	38.6	38.6	38.6
	Dry Density, pcf	82.9	84.5	84.0
	Saturation, %	100.9	104.9	103.5
	Void Ratio	1.0323	0.9937	1.0070
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	34.6	33.0	30.0
	Dry Density, pcf	87.6	89.2	93.6
	Saturation, %	101.1	100.4	101.1
	Void Ratio	0.9242	0.8890	0.8013
	Diameter, in.	2.500	2.500	2.500
	Height, in.	0.947	0.948	0.897
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	0.985	1.594	3.005
	Displacement, in.	0.482	0.326	0.317
	Ult. Stress, ksf	0.985	1.594	3.005
	Displacement, in.	0.482	0.326	0.317
	Strain rate, in./min.	0.001	0.001	0.001

Sample Type: Laboratory molded
Description: Dark gray Clay w/ Gray sand

Assumed Specific Gravity= 2.70
Remarks:

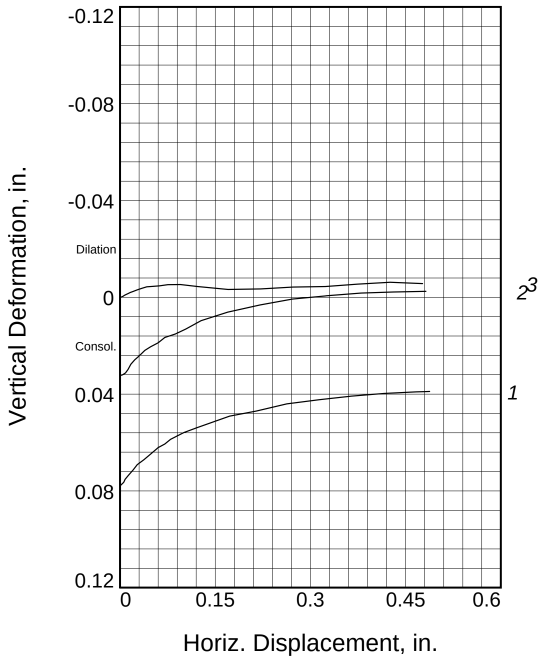
Figure ________

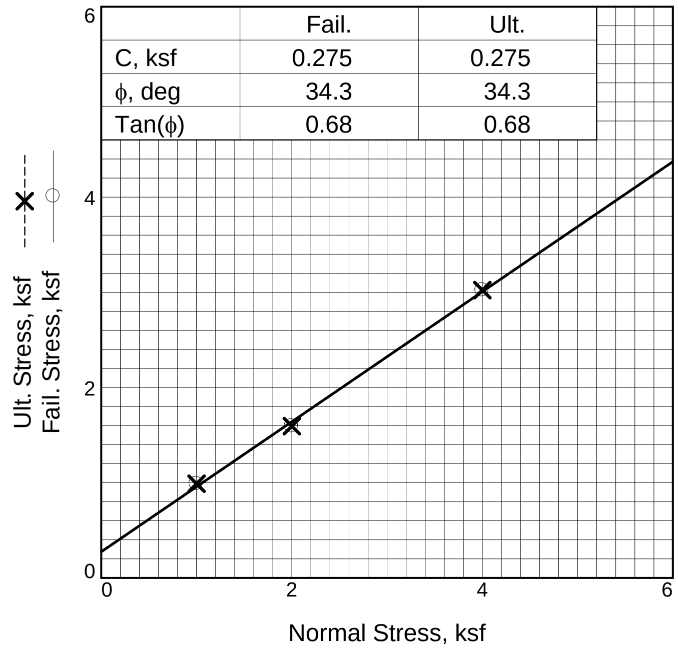
Client: TSI Laboratories, Inc.

Project: V-231109 Calhoun Co Glo Bridges

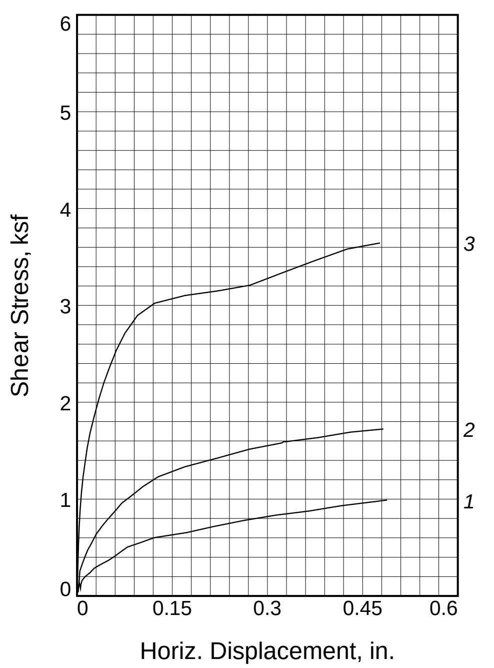
Location: B8-A/B-1
Depth: 18-20 ft.
Proj. No.: 92225176 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Terracon Consultants, Inc.
Houston, TX





	Fail.	Ult.
C, ksf	0.275	0.275
ϕ, deg	34.3	34.3
Tan(ϕ)	0.68	0.68



	Sample No.	1	2	3
Initial	Water Content, %	25.4	25.4	25.4
	Dry Density, pcf	93.8	94.8	94.6
	Saturation, %	85.9	88.0	87.6
	Void Ratio	0.7971	0.7783	0.7813
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.000	1.000	1.020
At Test	Water Content, %	24.2	28.4	24.7
	Dry Density, pcf	93.8	94.8	94.6
	Saturation, %	82.0	98.4	85.2
	Void Ratio	0.7971	0.7783	0.7813
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.000	1.000	1.020
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	0.989	1.594	3.022
	Displacement, in.	0.488	0.326	0.122
	Ult. Stress, ksf	0.989	1.594	3.022
	Displacement, in.	0.488	0.326	0.122
	Strain rate, in./min.	0.005	0.005	0.005

Sample Type: Laboratory molded
Description: Gray Sand w/clay lumps

Assumed Specific Gravity= 2.7
Remarks:

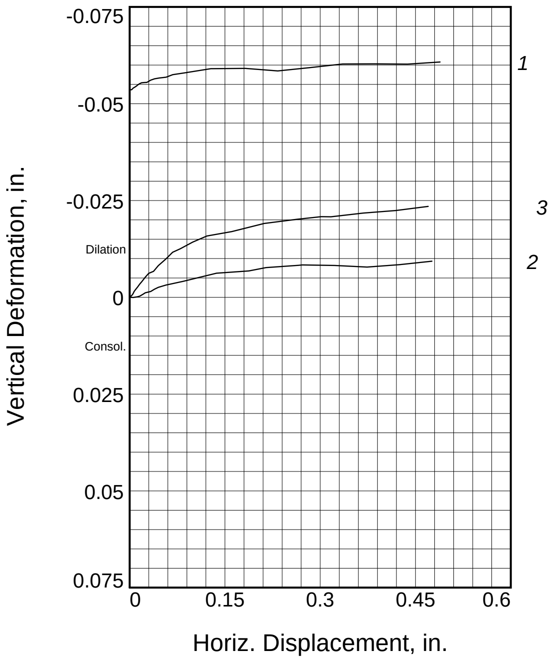
Figure ________

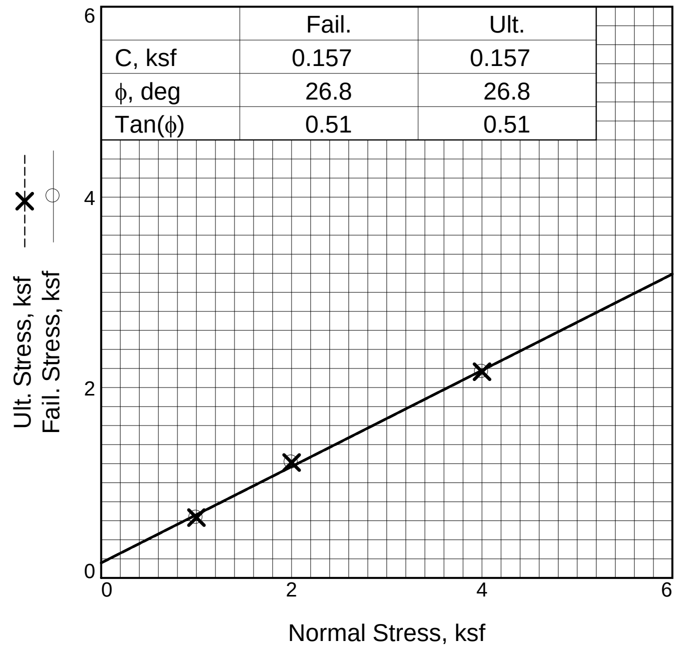
Client: TSI Laboratories, Inc.

Project: V-231109 Calhoun Co Glo Bridges

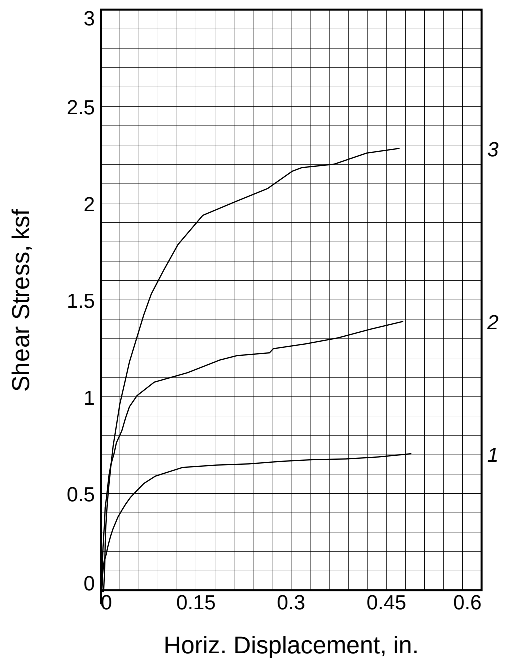
Location: **B8-B/**B-2
Depth: 18-20 ft. **Proj.**
No.: 92225176 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Terracon Consultants, Inc.
Houston, TX





	Fail.	Ult.
C, ksf	0.157	0.157
ϕ, deg	26.8	26.8
Tan(ϕ)	0.51	0.51



	Sample No.	1	2	3
Initial	Water Content, %	24.2	24.2	24.2
	Dry Density, pcf	102.5	101.6	100.8
	Saturation, %	98.8	96.5	94.8
	Void Ratio	0.6743	0.6900	0.7028
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.000	1.000	0.970
At Test	Water Content, %	25.5	25.9	24.4
	Dry Density, pcf	102.5	101.6	100.8
	Saturation, %	104.1	103.2	95.6
	Void Ratio	0.6743	0.6900	0.7028
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.000	1.000	0.970
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	0.634	1.212	2.165
	Displacement, in.	0.128	0.215	0.302
	Ult. Stress, ksf	0.634	1.212	2.165
	Displacement, in.	0.128	0.215	0.302
	Strain rate, in./min.	0.001	0.001	0.001

Sample Type: Laboratory molded
Description: Reddish brown Fat Clay w/sand

Assumed Specific Gravity= 2.75
Remarks:

Figure ________

Client: TSI Laboratories, Inc.

Project: V-231109 Calhoun Co Glo Bridges

Location: **B7-A/**B-3
Depth: 23-25 ft.
Proj. No.: 92225176 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Terracon Consultants, Inc.
Houston, TX

KEY TO SYMBOLS

Symbol	Description
	Strata symbols
	Low plasticity clay
	High plasticity clay
	Poorly graded clayey silty sand
	Clayey sand
	Soil Samplers
	Steel Tube Sample
	Split Spoon Sample
	Disturbed Sample

Consistency of Sands & Gravels	
Consistency	**Penetration Resistance (N)* Blows Per Foot**
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	Over 50

Consistency/Strength of Clays & Silty Clays		
Consistency	**Undrained Shear Strength, tsf**	**Pocket Penetrometer (p)**
Very Soft	Less than 0.125	0 – 0.5
Soft	0.125 – 0.25	0.5 – 1.0
Firm	0.25 – 0.50	1.0 – 1.75
Stiff	0.50 – 1.0	1.75 – 3.5
Very Stiff	1.0 – 2.0	3.5 – 4.5
Hard	Over 2.0	Over 4.5

*N=Number of Blows from 140 lb. hammer falling 30”to drive a 1-3/8” ld. split barrel sample (ASTM D-1586)

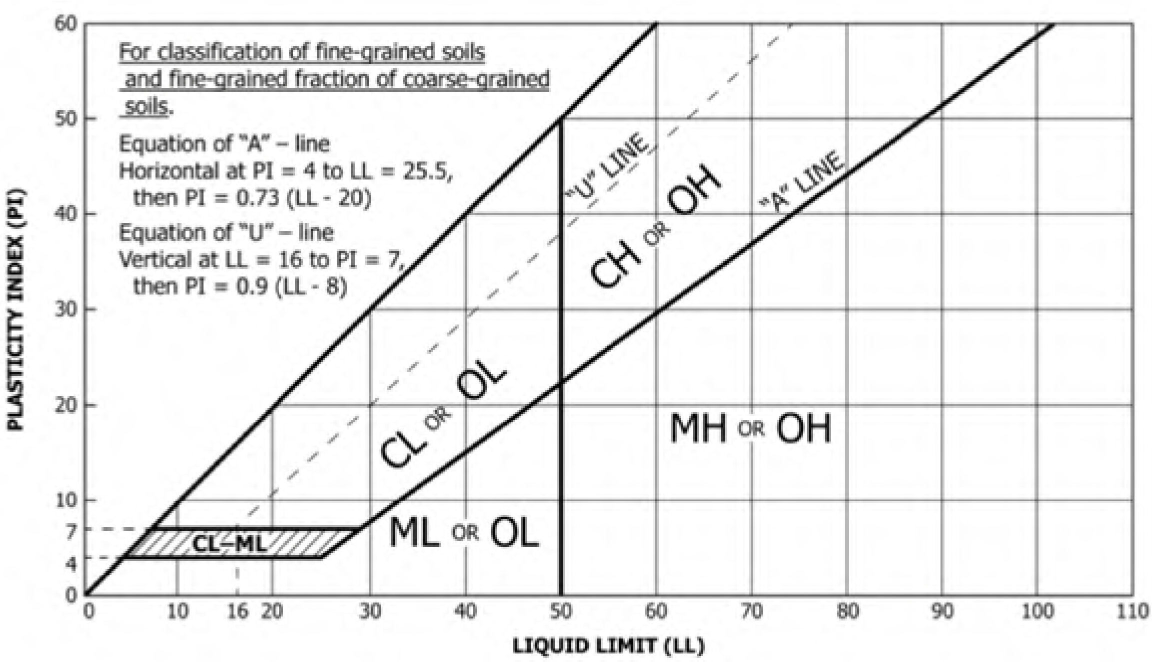
Soil Grain Analysis
US Standard Sieves

6” 3” ¾” #4 #10 #40 #200

Boulders	Cobbles	Gravel		Sand			Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine		

152 76.2 19.1 4.76 2.0 0.420 0.074 0.002

Soil Grain Size in Millimeters
ASTM D-2488



FIELD AND LABORATORY TESTING PROCEDURES
(TEST PROCEDURES ARE PRESENTED FOR INFORMATIONAL PURPOSES)

FIELD TESTING

A. Boring Procedure Between Samples

The borehole is extended downward, between samples, by continuous flight, hollow or solid stem augers or by rotary drilling techniques using bentonite drilling fluid or water.

B. Penetration Test and Split-Barrel Sampling of Soils
ASTM D-1586

This sampling method consists of driving a 2-inch outside diameter split barrel sampler using a 140 pound hammer freely falling through a distance of 30 inches. The sampler is first seated 6 inches into the material to be sampled and then driven an additional 12 inches. The number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance. Recovered samples are first classified as to color and texture by the driller. Later, in the laboratory, the driller's field classification is reviewed by the soils engineer who examines each sample.

C. Thin-Walled Tube Geotechnical Sampling
ASTM D-1587

This method consists of pushing thin walled steel tubes, usually 3 inches in diameter, into the soils to be sampled using hydraulic or other means. Cohesive soils are usually to be sampled in this manner and relatively undisturbed samples are recovered.

D. Soil Investigation and Sampling by Auger Borings
ASTM D-1452

This method consists of augering a hole and removing representative soil samples from the auger flight or bit at 5 foot depth intervals or with each change in substrata. Disturbed sampled are obtained and this method is, therefore, limited to situations where it is satisfactory to determine the approximate subsurface profile.

E. Diamond Core Drilling for Site Investigation
ASTM D-2113

This method consists of advancing a hole into hard strata by rotating a single or double tube core barrel equipped with a cutting bit. Diamond, tungsten carbide, or other cutting agents may be used for the bit. Wash water is used to remove the cuttings and cool the bit. Normally, a 2 inch outside diameter by 1-3/8 inch inside diameter (NX) coring bit is used unless otherwise noted. The rock or hard material recovered within the core barrel is examined in the field and in the laboratory and the cores are stored in partitioned boxes. The core recovery is the length of the material recovered and is expressed as a percentage of the total distance penetrated.

F. Visual – Manual Soil Classification Procedure
ASTM D-2488

This procedure is a visual – manual soil classification methodology for the description of soil for engineering purposed when precise soils classification is not required.

LABORATORY TESTING

A. Atterberg Limits: Liquid Limit, Plastic Limit and Plasticity Index of Soils
ASTM D-4318, TEX 104-E, 105-E and 106-E

Atterberg Limits determine the soil's plasticity characteristics. The soil's Plasticity Index (PI) is representative of this characteristic and is the difference between the Liquid Limit (LL) and the Plastic Limit (PL). The LL is the moisture content at which the soil will flow as a heavy viscous fluid. The PL is the moisture content at which the soil begins to lose its plasticity. The test results are presented on the boring logs adjacent to the appropriate sampling information.

B. Particle Size Analysis of Soils
ASTM D-422 and TEX 110-E

Grain size analysis tests are performed to determine the particle size and distribution of the samples tested. The grain size distribution of the soils coarser than the Standard Number 200 sieve is determined by passing the sampled through a standard set of nested sieves.

C. Laboratory Determination of Water (Moisture) Content of Soil and Rock
ASTM D-2216 and TEX 103-E

The moisture content of soil is defined as the ratio, expressed as a percentage, of the weight of water in a given soil mass to the weight of solid particles. It is determined by measuring the wet and oven dry weights of a soil sample. The test results are presented on the boring logs.

D. Unconfined Compressive Strength of Cohesive Soil
ASTM D-2166

The unconfined compressive strength of soil is determined by placing a section of an undisturbed sample into a loading frame and applying an axial load until the sample fails in shear. The test results are presented on the boring logs adjacent to the appropriate sampling information.

E. California Bearing Ratio (CBR) of Lab Compacted Soils
ASTM D-1883

The CBR test is performed by compacting soil in a 6 inch diameter mold at the desired density, soaking the sample for four days under a surcharge load approximating the pavement weight and then testing the soils in punching shear. A 2 inch diameter piston is forced into the soil to determine the resistance to penetration. The CBR is the ratio of the actual load required to produce 0.1 inches of penetration to that producing the same penetration in a standard crushed stone.

F. Swell Test
ASTM D-4546

The swell test is performed by confining a 1 inch thick specimen in a 2-1/2 inch diameter stainless steel ring and loading the specimen to the approximate overburden pressure. The test specimen is then inundated with distilled water and allowed to swell for 48 hours. The volumetric swell is measured as a percentage of the total volume and is converted mathematically to linear swell.

G. Compaction Tests
ASTM D-698, D-1557, TEX 113-E or 114-E

The compaction test is performed by compacting soil in a steel mold at varying moisture contents. Layers are compacted using a hammer weight and number of blows per layer which vary with the different test procedures. ASTM D-698, D-1557, TEC 113-E and 114-E. The data is plotted and the maximum weight and optimum moisture content is determined.

H. Classification of Soils for Engineering Purposes
Unified Soil Classification System, D-2487

This standard describes a system for classifying mineral and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index and shall be used when precise classification is required.

RECOMMENDED SPECIFICATIONS FOR PLACEMENT OF SELECT FILL

1. General

The soils engineer shall be the owners representative to control the placement of compacted fill. The soils engineer shall approve the subgrade preparation, the fill materials, the method of placement and compaction, and shall give written approval of the completed fill.

2. Preparation of Existing Ground

All topsoil, plants and other organic material shall be removed. The exposed surface shall be scarified, moistened if necessary, and compacted in the manner specified for subsequent layers of fill.

3. Select Fill Material

Fill shall have a liquid limit of less than 35 and a Plasticity Index between 8 and 18. The fill shall contain no organic material or other perishable material, and no stones larger than 6 inches. Fill material shall be approved by the soils engineer.

4. Placing Select Fill

Fill materials shall be placed in horizontal layers not exceeding 8 inches thickness after compaction. Successive loads of material shall be dumped so as to secure even distribution, avoiding the formation of layers of lenses of dissimilar materials. The contractor shall route hill hauling equipment to distribute travel evenly over the fill area.

5. Compaction of Select Fill

a. Moisture Control: The moisture content of the fill material shall be distributed uniformly throughout each layer of the material. The allowable range of moisture content during compaction shall be within plus two (+2) and minus two (-2) percentage points of the optimum moisture content. The contractor may be directed to add necessary moisture to the material either in the borrow area or upon the fill surface or to dry the material, as directed by the soils engineer. The drying of cohesive soils between lifts to moisture contents less than 70% of optimum before the placement of subsequent lifts shall be avoided or the fill reworked at the proper moisture content.

b. Compaction: The material in each layer shall be compacted to obtain proper densities. Compaction by the hauling equipment alone will not be considered sufficient. Structural fills, including pavement subgrade, subbase, and base, shall be compacted to densities equivalent to the percentages of the Standard Proctor (ASTM D-698) or Modified Proctor (ASTM D-1557) maximum dry density listed in the table below. The Texas Department of Highways and Public Transportation Method TEX 113-E or TEX 114-E compaction test, which varies the compactive effort with soil type, may be substituted for the Standard or Modified Proctor methods and percentages listed in the table below.

	PERCENT COMPACTION	
Area	Fine Grained Soils ASTM D-698 (Standard) or TEX 114-E	Coarse Grained Soils ASTM D-1557 (Modified) or TEX 113-E
Within five (5) feet of building lines, under footings, floor slabs, slab-on-grade foundation and structures attached to the building (i.e. walls, patios, steps)	95	95+
More than five (5) feet beyond building lines, under walks, and fill area to be landscaped	90	90
Pavement subgrade and subbase, including lime treated soils	95	95+

Soils classified as coarse grained soils are those with more than 50%, by weight, retained on the No. 200 Standard Sieve.

6. Comparison Testing

Field density tests for the determination of the compaction of the fill shall be performed by TSI Laboratories, Inc. in accordance with recognized procedures for making such tests. A representative number of tests shall be made in each compacted lift at locations selected by the soils engineer or his/her representative. For general structural and paving fills, we suggest one test per 3,000 square feet per lift with a minimum of three tests per lift.

IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

The following observations and suggestions are provided to help you better utilize your geotechnical engineering report and to reduce construction problems and delays related to the soil and groundwater conditions.

REPORT IS BASED UPON SPECIFIC SITE AND PROJECT

A geotechnical report is based on a subsurface exploration conducted on a specific site and planned using specific project information. The project information typically includes structure size and configuration, type of construction, and general location on the site. Limitations, such as existing buildings or utilities, specific foundation requirements for structures, budget limitations, and the level of risk assumed by the client may affect the scope of the exploration.

Since the report applies to a specific structure and site, the geotechnical report should not be used in the following circumstances unless the geotechnical engineer has reviewed the changes and concurs in the use of the report.

- When the nature of the proposed structure is changed, such as an office building instead of a warehouse or parking garage, or a refrigerated warehouse instead of one which is not refrigerated
- When the size, configuration, or floor elevations is changed
- When the location of the structure on the site is changed
- When there is a change of ownership

FINDINGS ARE PROFESSIONAL ESTIMATES

The actual subsurface conditions are determined only at the boring locations and only at the time the samples are taken. The information is extrapolated by the geotechnical engineer who then renders professional opinions regarding the characteristics of the subsurface materials, the behavior of the soils during construction, and appropriate foundation designs. No exploration, however complete, can be assured of sampling the entire range of soil conditions. The soils may vary between or beyond the borings and stratum transitions may be more gradual or more abrupt, and all types of oils and rock existing on the site may not be found in the borings. The geotechnical engineer is often retained during construction to evaluate variances and recommend solutions to problems encountered on the site.

SUBSURFACE CONDITIONS CAN CHANGE

Grading operations on or close to the site, floods, groundwater fluctuations, utility construction, and utility leaks are among the events that can change the subsurface conditions. The geotechnical engineer should e kept apprised of such events.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

A geotechnical report may have been made to evaluate foundation alternatives only, for preliminary site evaluation, or for other limited purposes. The exploration may also have been limited by the direction of the client, budget limitations, or the level of risk assumed by the client. Therefore, no one other than the original client should use the report for its intended purpose or other purposes without conferring with the geotechnical engineer.

GEOTECHNICAL REPORTS ARE SUBJECT TO MISINTERPRETATION

Geotechnical reports are based on the project information available at the time the report was made and the judgment and opinions of the geotechnical engineer. This specialized information is subject to misinterpretation by other design professionals, contractors and owners. The geotechnical engineer should be retained during the design process to interpret the recommendations and review the adequacy of the plans and specifications relative to geotechnical issues. The boring logs should no be separated from the geotechnical report, but, rather the entire report should be made available to the contractors and others needing this information.