

**EUSTIS ENGINEERING COMPANY, INC.**

3011 28TH STREET
METAIRIE, LOUISIANA 70002-6019
PN 504-834-0157 / FN 504-834-0354
EMAIL: INFO@EUSTISENG.COM / SITE: WWW.EUSTISENG.COM



5 November 2002

E.C.O. Builders Inc.
900 Old Spanish Trail
Slidell, Louisiana 70458

Attention Mr. Elwin Ordogne
Email eco@ecobuildersinc.com

Gentlemen:

Geotechnical Investigation
Abek Project
Oak Harbor Boulevard
Slidell, Louisiana
Eustis Engineering Project No. 17604

Transmitted is one bound copy of our engineering report covering a geotechnical investigation performed for the subject project. A copy will be sent via email in .pdf format. A bound copy will be transmitted to Mr. Pete Dammon representing Dammon Engineering Inc.

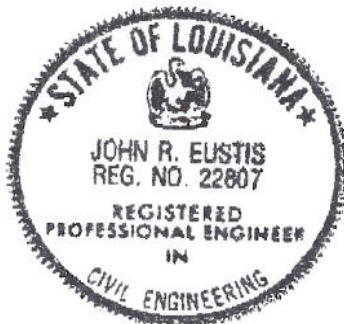
Thank you for asking us to perform these services.

Yours very truly,

EUSTIS ENGINEERING COMPANY, INC.

JOHN R. EUSTIS, P.E.

ANL:ken/mcp



GEOTECHNICAL INVESTIGATION
ABEK PROJECT
OAK HARBOR BOULEVARD
SLIDELL, LOUISIANA
EUSTIS ENGINEERING PROJECT NO. 17604

FOR
E.C.O. BUILDERS INC.
SLIDELL, LOUISIANA

DAMMON ENGINEERING INC.
SLIDELL, LOUISIANA

By
Eustis Engineering Company, Inc.
Metairie, Louisiana

5 NOVEMBER 2002

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GEOTECHNICAL INVESTIGATION
ABEK PROJECT
OAK HARBOR BOULEVARD
SLIDELL, LOUISIANA
EUSTIS ENGINEERING PROJECT NO. 17604

INTRODUCTION

1. This report contains the results of a geotechnical investigation performed for the proposed Abek Project on Oak Harbor Boulevard in Slidell, Louisiana. The investigation was performed in accordance with Eustis Engineering Company, Inc.'s proposal dated 18 September 2002, which was accepted on 19 September 2002 by Mr. Elwin Ordogne, Vice President, E.C.O. Builders Inc., Slidell, Louisiana. The consulting engineers for the project are Dammon Engineering Inc., Slidell, Louisiana.
2. This report has been prepared in accordance with generally accepted geotechnical engineering practice for the exclusive use of E.C.O. Builders and their designated representatives for specific application to the subject sites. In the event of any changes in the nature, design, or location of the proposed structures, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are modified and verified in writing. Should these data be used by anyone other than E.C.O. Builders and their designated representatives, they should contact Eustis Engineering for interpretation of data and to secure other information that may be pertinent to this project.
3. Recommendations and conclusions contained in this report are to some degree subjective. The report in its entirety should not be included in the contract plans and

specifications. However, the results of the soil borings and laboratory tests contained in the Appendix of this report may be included in the plans and specifications.

4. The analyses and recommendations contained in this report are based in part on data obtained from the soil borings. The nature and extent of variations in subsoil conditions that may exist between and away from the boring locations may not become evident until construction. If variations then appear, it will be necessary to reevaluate the recommendations contained in this report.
5. Eustis Engineering's scope of work does not include the investigation or detection of the presence of any biological pollutants in or around the proposed structures. The term "biological pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and the byproducts of any such biological organisms.

SCOPE

6. This investigation included the drilling of soil test borings to determine subsoil conditions and stratification and to obtain samples of the various strata encountered. Soil mechanics laboratory tests performed on samples obtained from the borings were used to evaluate the physical properties of the subsoils. Engineering analyses, based on the soil borings and laboratory tests, were performed to determine recommendations regarding allowable pile load capacities, estimated settlement, and general construction recommendations.

SOIL BORINGS

7. Two undisturbed soil test borings, each 40 feet in depth, were drilled on 8 October 2002 at the locations designated by Dammon Engineering. Boring 1 was made on Lot 3 and Boring 2 was made on Lot 4. The approximate boring locations are

shown on Figure 1. The borings were made with a truck mounted rotary type drill rig and mats were required to gain access to the boring locations. Upon completion of drilling operations, the borings were backfilled and/or grouted with cement-bentonite grout in accordance with current regulatory requirements. Detailed descriptive logs of the borings are shown in both tabular and graphical form in the Appendix.

8. Sampling Methods. Undisturbed samples of cohesive or semi-cohesive subsoils were obtained at close intervals or changes in strata using a 3-in. diameter thinwall Shelby tube sampler. The samples were immediately extruded from the sampler, inspected, and visually classified by Eustis Engineering's soil technician. Pocket penetrometer tests were performed on the soil samples to give a general indication of their shear strength or consistency. The results of these tests are shown on the boring logs under the column heading "PP." Representative samples were placed in moisture proof containers and sealed for preservation of their natural moisture content.

LABORATORY TESTS

9. A series of soil mechanics laboratory tests was performed on samples obtained from the borings. Included in these laboratory tests were natural water content, unit weight, and either unconfined compression shear (UC) or unconsolidated undrained triaxial shear (OB). An Atterberg liquid limit determination was also performed on a representative cohesive sample to aid in classification. The Atterberg limit and natural water content determinations aid in the classification of the subsoils and provide an indication of their relative compressibility. The results of the laboratory tests are tabulated on the boring logs in the Appendix.

DESCRIPTION OF SUBSOIL CONDITIONS

Site Reconnaissance

10. Observations by our soil technician indicate the two adjacent sites are gently sloping undeveloped lots. Surface vegetation consists of grass and small trees. Both sites are bounded by an existing pond.

Stratigraphy

11. Reference to the logs of Borings 1 and 2 indicates the initial 6 inches consists of medium stiff brown clay with roots. Beneath these surficial soils, medium stiff to very stiff brown, light gray, and tan clay and sandy clay continue to depths ranging from 6 to 9 feet, followed by loose to medium dense brown, light gray, and tan clayey sand to the 15-ft depth. Medium stiff to stiff gray, tan, and reddish-gray clay extends to the 35-ft depth, overlying soft gray and reddish-gray sandy clay in which the borings terminate at 40 feet below the existing ground surface.

Ground Water

12. To determine the ground water conditions at the time of the field investigation, observations were made in auger borings located near Borings 1 and 2. The auger borings were made without the addition of water to the 12-ft depth and upon completion of drilling, both bore holes were dry to the 12-ft depth. The depth to ground water will vary with climatic conditions, drainage improvements, the water level in Lake Pontchartrain, and other factors. The ground water level should be determined by those persons responsible for construction immediately prior to beginning work.

FOUNDATION ANALYSES

Furnished Information

13. Drawings and information furnished by Dammon Engineering indicate the proposed two-story buildings will each have 9,000 square feet of floor space and a 5,000 square foot footprint. The proposed building on Lot 3 will have approximate plan dimensions of 78' x 62'. The plan dimensions for the proposed building on Lot 4 were not provided. No additional fill will be placed on the site to reach finished grade beneath the buildings.

Foundation Recommendations

14. The proposed buildings should be supported on deep foundations comprised of driven treated timber pile foundations. All structural loads from the new building (floor, walls, and columns) should be supported on piles having the same tip embedment below the existing ground surface in order to minimize differential settlement. Pile caps should be poured integrally with the floor slab and grade beams.

Site Preparation

15. Drainage During Construction. The initial step to prepare the site for construction should be to establish adequate temporary and permanent drainage to prevent ponding of water and ensure immediate runoff of all rainfall. It is strongly recommended the contractor maintain adequate surface drainage away from all foundations during and after construction. This may be accomplished by setting grades to ensure positive drainage of water away from the foundation areas and providing adequate surface and subsurface drainage structures as required.

16. Permanent Drainage. The near surface soils are subject to a reduction in shear strength and excessive settlement if the moisture content of these soils increases (naturally or as a result of construction operations). It is strongly recommended adequate permanent drainage be provided to collect all rainfall away from the building foundations after completion of construction. All downspouts draining rainfall from the building roofs should be connected to pipes which discharge into a drainage system. Water should not be allowed to collect near the foundations.
17. Clearing. The existing ground surface beneath the buildings should be stripped of all vegetation, debris, stumps, organic matter, and any other deleterious materials. Stripping should be to a depth necessary to remove weak surficial soils and reach firm undisturbed soil. The exact depth of stripping should be determined during construction.
18. Subgrade Preparation. After the stripping and clearing operations, the exposed surface should be crowned and sealed with several passes of a smooth steel drum roller compactor or bulldozer. Any depressions or weak areas identified should be thoroughly cleaned out to the surface of undisturbed soil and backfilled with a select structural fill material placed and compacted under controlled conditions. All clearing and compaction operations should be performed during periods of dry weather only.
19. Structural Fill. A select granular material, such as sand or clayey sand, should be used as backfill and/or fill required to reach design grade. Sand fill should be non-plastic and free of roots, clay lumps, and other deleterious materials with no more than 10% by weight of material passing a U.S. Standard No. 200 mesh sieve. This material will expedite construction and drain freely after inclement weather. As an alternate to sand fill, clayey sand may be used but will require more effort to process and compact. However, this material is moisture sensitive and could delay

construction if installed during periods of wet weather. Clayey sand fill should have a liquid limit of 25 or less and a plasticity index no greater than 6.

20. Compaction. Structural fill used as form fill beneath pile supported floor slabs should be spread in loose lifts of 6 to 8 inches. Each lift should be compacted to 95% of the maximum dry density within $\pm 2\%$ of optimum water content in accordance with ASTM D 698. All clearing and compaction operations should be accomplished during periods of dry weather only.

Pile Foundations

21. Allowable Pile Load Capacities. Analyses were performed to determine the estimated allowable single pile load capacities for various sizes and embedments of treated ASTM D 25 quality timber piles for the project. Our analyses assume the timber piles are driven vertically and provide for a nominal 2-ft cutoff for the pile cap. The results of these analyses are tabulated below.

| SIZE OF TREATED ASTM D 25 QUALITY TIMBER PILE | PILE TIP EMBEDMENT BELOW EXISTING GROUND SURFACE IN FEET | ESTIMATED ALLOWABLE SINGLE COMPRESSIVE PILE LOAD CAPACITIES IN TONS FACTOR OF SAFETY = 2 |
|-----------------------------------------------------|-------------------------------------------------------------------|---------------------------------------------------------------------------------------------------|
| 6-In. Tip with Natural Taper to Butt | 25 | 9* |
| 6-In. Tip 8-In. Butt | 30 | 12* |
| 8-In. Tip with Natural Taper to Butt | 25 | 13 |
| 8-In. Tip 12-In. Butt | 30 | 17 |

*Pile capacities for small timber piles are normally limited to 8 tons by local building codes.

22. These allowable pile load capacities contain an estimated factor of safety of approximately 2 against failure of a single pile through the soil. The analyses for pile

capacities are based on a soil-pile relationship only. Therefore, the structural capacity of the piles, and any connections to transmit these loads, should be determined by a structural engineer.

23. Pile Group Capacity and Spacing. Timber piles will derive their supporting capacity through skin friction. Skin friction piles used in groups or rows should be evaluated on the basis of group perimeter shear by the formula shown on Figure 2. The minimum center to center spacing between individual timber piles within a pile group or row should be determined using the equation given on Figure 2.
24. Estimated Settlement Due to Structural Loads. We estimate settlement of foundations due to sustained loads and supported on vertical piles driven in small groups or isolated rows to tip embedments of at least 20 feet will be $\frac{1}{4}$ to $\frac{1}{2}$ inch. This estimate does not include elastic deformation of the piles which should be added to the settlement estimate. Elastic deformation of the piles may be estimated as 67% of the static column strain of a pile acting as a column. Concrete for pile caps should be placed integrally with the concrete for grade beams and floor slabs. All piles should be driven to the same tip embedment in order to minimize differential foundation settlement.
25. Our estimates of settlement are for piles used individually or in small groups and are based on the following assumptions:
- the largest group dimension will be no greater than 20% of the pile length,
 - the center to center spacing between groups will be no closer than twice the largest group dimension,
 - the center to center spacing between rows of single piles will be no closer than 8 feet, and
 - no more than 1 foot of fill will be placed to raise existing grade.

In the event any of our assumptions are not met, Eustis Engineering should be contacted to evaluate the potential settlement of pile foundations.

26. Pile Driving. Close field supervision should be maintained by experienced personnel to ensure proper procedures are followed and accurate records are kept during all pile driving operations. The driving record should include hammer model, driving energy, pile type, overall length, tip and butt diameters, depth and diameter of predrill, and the number of blows per foot of penetration. An accurate driving record is especially important to verify all piles are installed to the required tip embedment and to give an indication of any unusual driving characteristics which may indicate pile breakage.

27. Predrilling. Based on the soil conditions developed at the boring locations, predrilling may be necessary for the installation of piles below 15 feet in depth to penetrate the loose to medium dense sand to this depth. Based on the borings, predrilling should extend to a depth of 15 feet for all piles. Piles should then be driven to a resistance indicative of their respective compressive load capacities. ***Predrilling should terminate no deeper than 5 feet above the final tip embedment.*** Predrilling should be accomplished by wet rotary drilling methods using a bit having a diameter no larger than 6 inches for small timber piles and 8 inches for larger piles. These requirements for predrilling should be reevaluated based on information from field verification of installation and design load capacities during a test pile program.

28. Hammers. Treated ASTM D 25 quality timber piles, having butt diameters of 12 inches and tip diameters of 8 inches, may be driven with a single acting air hammer with a manufacturer's rated energy of 15,000 ft-lbs per blow. Smaller timber piles with 6-in. diameter tips and 8-in. diameter butts may be driven with a drop hammer or single acting air hammer delivering 7,500 ft-lbs of energy per blow. If a drop hammer is used, the ram weight should not exceed 2,500 to 3,000 pounds and the

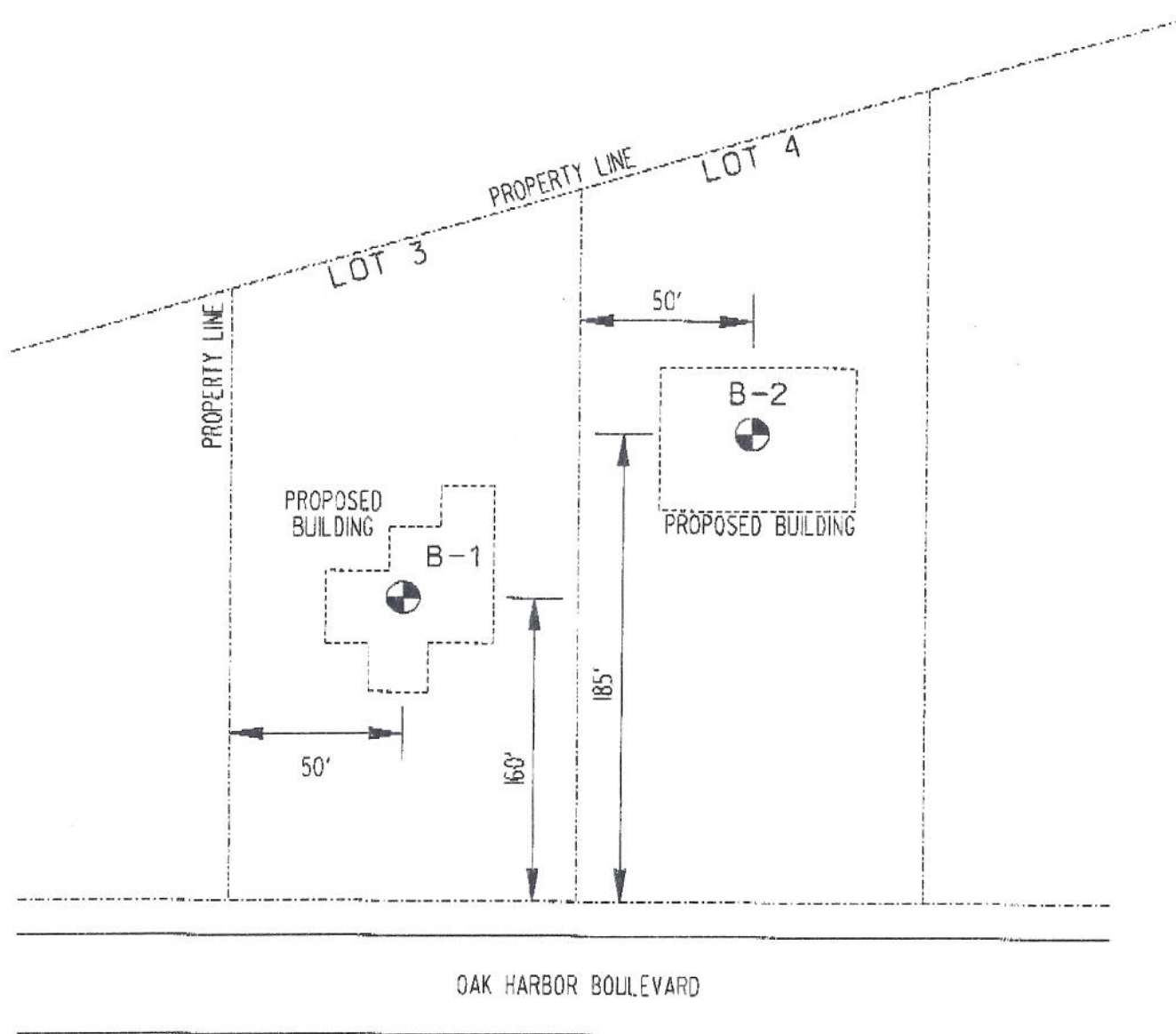
drop should not exceed 3 feet. Using these driving energies, piles should be driven no harder than 25 blows per foot in order to minimize structural damage to the piles.

29. Test Piles and Pile Load Tests. Eustis Engineering considers the test pile program and the pile load test as an extension of our geotechnical investigation. Therefore, Eustis Engineering should be retained to observe installation of the test piles and perform and evaluate the load test.
30. Once the design pile type and length have been selected, it is recommended at least one test pile be driven within the area of each of the proposed buildings to be pile supported to determine predrilling requirements and to verify capacities recommended in this report. The test piles should be installed with the same equipment and methods which are used to drive the job piles. The test pile showing the least resistance to driving should be allowed to "set" for a period of 14 days and then should be load tested to failure in accordance with ASTM D 1143.
31. Vibrations. Pile driving operations will cause vibrations which may affect nearby structures and utilities. Pile driving should be monitored with a seismograph at any structure of concern during the test pile program and the driving of job piles to record their magnitude of vibrations. Peak particle velocities of 0.25 in./sec as measured by the seismograph are generally regarded as a vibration level uncomfortable to human perception. Peak particle velocities in excess of 0.25 in./sec may densify cohesionless soils near this site. Densification can result in settlement of pavements, structures, or utilities founded in or over these deposits. Exceeding peak particle velocities of 0.5 in./sec may induce vibratory damage to the structures, pavements, and utilities. If sustained peak particle velocities exceed 0.25 in./sec, Eustis Engineering should be contacted, pile driving suspended, and consideration given to altering installation criteria.

ADDITIONAL GEOTECHNICAL SERVICES


32. To provide continuity between the investigation, design, and construction phases, Eustis Engineering should be retained to provide additional services. These services may include field density tests on compacted fill, pile inspection and logging, performance and evaluation of a pile load test, vibration monitoring, and any other soils or materials testing services which will provide quality control during construction and conformance to design specifications.

33. In summary, Eustis Engineering should be retained to monitor all geotechnical related work performed by the contractor. This permits the geotechnical engineer to be available quickly, evaluate unanticipated conditions, conduct additional tests if required, and formulate alternative solutions to problems when necessary. This is recommended to avoid construction cost overruns or disputes on the project.



⊕ DENOTES UNDISTURBED BORINGS DRILLED:
8 OCTOBER 2002

NOT TO SCALE

| | | |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------|-----------------------------------------------|
|  EUSTIS ENGINEERING COMPANY, INC. GEOTECHNICAL ENGINEERS 3011 28TH STREET METAIRIE, LOUISIANA | | |
| LOCATION OF BORINGS ABEK PROJECT OAK HARBOR BOULEVARD SLIDELL, LOUISIANA | | |
| DRAWN BY: D. LAFONT CHECKED BY: J.R.E. | PLOT DATE: 4 NOV. 02 JOB NO.: 17604 | CADD FILE: FIGURE 1.CGN FIGURE 1 |

CAPACITY OF PILE GROUPS

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

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

In Which:

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

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

SPACING WITHIN PILE GROUPS

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

In Which:

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





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

APPENDIX



**LEGEND AND NOTES FOR
LOG OF BORING AND TEST RESULTS**

PP Pocket penetrometer resistance in tons per square foot
 TV Torvane shear strength in tons per square foot
 SPT Standard Penetration Test. Number of blows of a 140-lb. hammer dropped 30 inches required to drive 2-in O.D., 1.4-in. I.D. sampler a distance of one foot into the soil, after first seating it 6 inches

SPLR Type of Sampling  Shelby  SPT  Auger  No Sample

SYMBOL Clay Silt Sand Humus Predominant type shown heavy;
 Modifying type shown light
   

DENSITY Unit weight in pounds per cubic foot

USC Unified Soil Classification

TYPE UC Unconfined compression shear
 OB Unconsolidated undrained triaxial compression shear on one specimen confined at the approximate overburden pressure
 UU Unconsolidated undrained triaxial compression shear
 CU Consolidated undrained triaxial compression shear
 DS Direct shear
 CON Consolidation
 PD Particle size distribution
 k Coefficient of permeability in centimeters per second
 SP Swelling pressure in pounds per square foot

ϕ Angle of internal friction in degrees

c Cohesion in pounds per square foot

Other laboratory test results reported on separate figure

Ground Water Measurements  Initial  Final

GENERAL NOTES

- (1) At the time the borings were made, ground water levels were measured below existing ground surface. These observations are shown on the boring logs. However, ground water levels may vary due to seasonal and other factors. If important to construction, the depth to ground water should be determined by those persons responsible for construction, immediately prior to beginning work.
- (2) While the individual logs of borings are considered to be representative of subsurface conditions at their respective locations on the dates shown, it is not warranted that they are representative of subsurface conditions at other locations and times.

ABEK PROJECT
OAK HARBOR BOULEVARD
SLIDELL, LOUISIANA

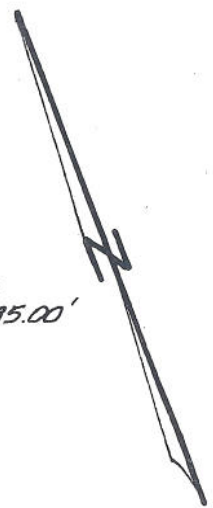
Boring: 2 Date Drilled: 10/08/02 Job No.: 17604 Gr. Water Depth: See Text Datum: See Text

| Scale In Feet | PP | SPT | S P L R | Symbol | Visual Classification | USC | Sample Number | Depth In Feet | Water Content Percent | Density | | Shear Tests | | | Atterberg Limits | | | Other Tests |
|---------------|----|-----|------------------|--------|--------------------------------------------------------------|-----|---------------|---------------|-----------------------|---------|-----|-------------|---|------|------------------|----|----|-------------|
| | | | | | | | | | | Dry | Wet | Type | σ | C | LL | PL | PI | |
| 0 | | | | | | | 1 | 0-0.5 | | | | | | | | | | |
| 3.00 | | | | | Medium stiff brown clay w/roots | CH | 2 | 2-3 | 17 | 112 | 131 | UC | - | 2045 | 50 | | | |
| 2.00 | | | | | Very stiff tan & brown clay w/sand pockets & shell fragments | CH | 3 | 5-6 | 21 | | | | | | | | | |
| 0.50 | | | | | Very stiff brown & tan sandy clay w/shell lenses & roots | CL | 4 | 8-9 | 27 | 90 | 114 | UC | - | 520 | | | | |
| 0.25 | | | | | Medium stiff tan & light gray clay w/roots & sand layers | CH | 5 | 11-12 | 16 | | | | | | | | | |
| 0.25 | | | | | Medium dense brown clay w/roots | SC | 6 | 14-15 | 16 | 118 | 136 | OB | - | 590 | | | | |
| 1.00 | | | | | Medium stiff light tan & gray clay w/organic matter | CH | 7 | 19-20 | | | | | | | | | | |
| 2.25 | | | | | Medium stiff tan & light gray clay w/shelly sand lenses | CH | 8 | 24-25 | 33 | 88 | 118 | UC | - | 880 | | | | |
| 2.00 | | | | | Stiff light gray & tan clay | CH | 9 | 29-30 | | | | | | | | | | |
| 1.00 | | | | | Stiff brown & light gray clay w/risures | CH | 10 | 34-35 | 46 | 75 | 109 | UC | - | 1125 | | | | |
| 0.50 | | | | | Soft gray sandy clay | CL | 11 | 39-40 | | | | | | | | | | |

Comments:

GOLF COURSE HOLE 7
 588°49'27"E - 211.88
 105.94' | 105.94'

LAKE



FLOOD ZONE
 "A-10" BFE 10'
 FLOOD ZONE
 "A-10" BFE 11'

SET 1/2" REF.
 REBAR @ 285.00'

LOT 2

SET 1/2" REF.
 REBAR @
 145.00'

LOT 3

LOT 4

LOT 5

BUILDING SETBACK:
 FRONT - 25'
 SIDES - 10'
 REAR - 25' FROM
 WATER

LEGEND:

- 1/2" REF. REBAR SET
- 5/8" REBAR FOUND
- ⊕ FIRE HYDRANT
- ⊗ WATER VALVE

BENCHMARK:
 "X" ON CURB.
 ELEV. 6.51'

N70°26'48"E - 331.59'

S20°26'48"W - 401.51'

N69°33'12"W

10' PLANT & UTIL. SERV.
 25' BLDG SETBACK

OAK HARBOR

