Geotechnical Report
Stumpy Point Water Treatment Facility
Stumpy Point, Dare Co., North Carolina
Project No. 01-106G
Prepared For: Hobbs, Upchurch &
Associates, P. A.

G
E
T
Solutions, Inc.

Geotechnical
Environmental
Testing
Solutions, Inc.

March 20,2001

TO: **Hobbs, Upchurch & Associates, P. A.**
2522 S. Croatan Highway, Suite 2A
Nags Head, NC 27959

Attn: Mr. Eric Weatherly, P. E.

RE: Report of Subsurface Investigation and Geotechnical Engineering
Stumpy Point Water Treatment Facility
Stumpy Point, Dare County, North Carolina
GET Project No: 01-106G

Dear Mr. Weatherly:

In compliance with your instructions, we have completed our Geotechnical Engineering Services for the referenced project. The results of this study, together with our recommendations, are presented in this report.

Often, because of design and construction details that occur on a project, questions arise concerning subsurface conditions. G E T Solutions, Inc. would be pleased to continue its role as Geotechnical Engineer during the project implementation.

We trust that the information contained herein meets your immediate need, and we would ask that you call this office with any questions that you may have.

Respectfully Submitted,
G E T Solutions, Inc.

Camille A. Kattan

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TABLE OF CONTENTS

1.0 PROJECT INFORMATION

1.1 Project Authorization:

G E T Solutions, Inc. has completed our Geotechnical Engineering study for the proposed Stumpy Point Water Treatment Facility project. The geotechnical engineering services were conducted in general accordance with G E T Proposal No. P01G-105, dated January 23, 2001. Verbal authorization to proceed with the Geotechnical Engineering services was obtained from Mr. Eric Weatherly, P. E., dated January 29, 2001.

1.2 Project Description:

The proposed development at this site is planned to consist of building a spherical 75,000-gallon elevated steel water tank. The tank will have a 30-foot diameter base, and will be approximately 140 feet in height. The structural loads associated with this tank will be furnished by the manufacturer at a later date, and are now assumed to be the weight of the water plus 50% for the tank structure, in addition to the wind loads estimated at 40 pounds per square foot of directional tank area.

Also, a single story block building is planned to be built adjacent to the tower to house associated water treatment equipment. The building will have a finished floor elevation at about 8' MSL, with exterior grades built-up to elevation 4' MSL.

If any of the noted information is incorrect or has changed, please inform G E T Solutions, Inc. so that we may amend the recommendations presented in this report, if appropriate.

1.3 Purpose and Scope of Services:

The purpose of this study was to obtain information on the general subsurface conditions at the proposed project site. The subsurface conditions encountered were then evaluated with respect to the available project characteristics. In this regard, engineering assessments for the following items were formulated:

1. General assessment of the soils revealed by the borings performed at the proposed development.

2. General location and description of potentially deleterious material found in the borings that may interfere with construction progress or structure performance, including existing fills or surficial/subsurface organics.

3. Soil subgrade preparation, including stripping, grading and compaction. Engineering criteria for placement and compaction of approved structural fill material.

4. Construction considerations for fill placement, subgrade preparation, and foundation excavations.

5. Feasibility of utilizing a shallow foundation system for support of the proposed tank structure. Design parameters required for the foundation system, including foundation sizes, allowable bearing pressures, foundation levels and expected total and differential settlements. Also, alternate prestressed concrete pile design was evaluated for the tank structure.

6. Feasibility of utilizing a deep foundation system (piles) for support of the proposed building. Design parameters required for the foundation system, including pile type and length, allowable bearing capacity, and expected total and differential settlements.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic material in the soil, bedrock, surface water, groundwater or air, on or below or around this site. Any statements in this report or on the boring logs regarding odors, color, unusual or suspicious items or conditions are strictly for the information of the client. Prior to development of this site, an environmental assessment is advisable.

2.0 FIELD AND LABORATORY PROCEDURES

2.1 Field Exploration:

In order to explore the general subsurface soil types and to aid in developing associated foundation design parameters, 4 Standard Penetration Test (SPT) borings (designated as B-1 and B-2 in the tank area, and B-3 and B-4 in the building area) were drilled by G E T Solutions, Inc.. The borings in the building area were drilled to depths of 30 feet below the existing site grades, and the tank borings were drilled to depths of 75 feet below the site grades. The SPT borings were performed with the use of a power drill rig using mud (rotary wash) drilling procedures. The soil samples were obtained with a Split-Spoon Sampler in general accordance with the Standard Penetration Test (SPT) ASTM D-1586. These samples were taken continuously from the ground surface to a depth of 10 feet, and at 5-foot intervals thereafter.

The boring locations were established, and were located and staked in the field by the client. The approximate boring locations are shown on the "Boring Location Plan" (Figure 1) attached to this report (Appendix I). This sketch was developed based on the plan provided to us by Hobbs, Upchurch & Associates, P. A.

2.2 Laboratory Testing:

Representative portions of all soil samples collected during drilling were sealed in glass jars, labeled and transferred to our Kitty Hawk laboratory for classification and analysis. The soil classification was performed by a Geotechnical Engineer in accordance with ASTM D2488.

A total of 3 representative split spoon soil samples were selected and subjected to natural moisture and -200 sieve wash analysis in order to corroborate the visual classification. These test results are presented on the "Log of Boring" sheets (Appendix II), included with this report.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Location and Description:

The project site is located on the north side of NCSR 1100, about 500 feet east of its intersection with US Highway 264, at the outskirt of the town of Stumpy Point, North Carolina. This property is currently level, low lying with heavy brush undergrowth, with existing grade elevations estimated to be near 1.5′ to 2′ MSL. Some standing water was observed to occur within the proposed project area during at the time of our field services.

3.2 Subsurface Soil Conditions:

The results of our field exploration program indicated the presence of approximately 5 feet of superficial muck and organic silt (OH) at the boring locations. The organic material thickness could vary between boring locations. Underlying the superficial muck layer, a 2-foot thick deposit of very soft silty clay (CH) was revealed in the borings, containing some organics.

Extending below a depth of 7 feet, the subsurface soils were generally uniform and consistent at the boring locations(granular soils), comprised of gray silty sand (SM) to a depth of 22.5 feet, followed by light gray to gray slightly silty sand (SP-SM). Some fine to coarse sand was found to occur between the depths of 22.5 and 27.5 feet.

The Standard Penetration Test (SPT) results, N-values, recorded within the granular soil layers ranged from 2 to 15 blows-per-foot (BPF) to a depth of 22.5 feet, indicating a very loose to medium dense relative density. The materials occurring at depths greater than 22.5 feet (to the boring termination depths of 30 to 75 feet) generally exhibited more dense characteristics, with the recorded N-values varying between about 25 and 100+ BPF. As an exception, a loose silty clayey sand deposit (SM-SC) was found between the depths of 37.5 and 47.5 feet.

The above subsurface description is of a generalized nature provided to highlight the major soil strata encountered. The records of the subsurface exploration included in Appendix II (Log of Boring sheets) and the Generalized Soil Profile presented in Appendix III should be reviewed for specific information as to individual borings. The stratifications shown on the records of the subsurface exploration represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations. The stratifications represent the approximate boundary between subsurface materials and the transition may be gradual.

3.3 Groundwater Information:

The groundwater was measured at each boring location during drilling operations. The depth to the groundwater at the boring locations was recorded at depths of 0.5 feet below the existing grades.

Groundwater conditions will vary with environmental variations and seasonal conditions, such as the frequency and magnitude of rainfall patterns, as well as man-made influences, such as existing swales, drainage ponds, underdrains and areas of covered soil (paved parking lots, side walks, etc.). It is estimated normal seasonal high groundwater level will occur at or above the existing ground surface (as noted at the time of our field services). We recommend that the contractor determine the actual groundwater levels at the time of the construction to determine groundwater impact on this project.

4.0 EVALUATION AND RECOMMENDATIONS: Water Treatment Building

On the basis of the results of our soil test borings, and considering the presence of the superficial organic deposit, it is our opinion that the proposed treatment building and its first floor slab should be supported by means of a pile foundation system. This option is considered viable and more economical, as performing subgrade improvement construction activities (undercut of organics and backfill with structural fill materials) is considered impractical given the shallow location of the groundwater table and the required depth of undercut (7 to 8 feet).

Accordingly, the employment of a deep foundation system, comprised of driven round timber piles, will be considered more economical. The pile design criteria are presented in the following sections of this report. It is noted that fill placement to establish exterior grades to about 4.5 feet MSL (an matching interior grades beneath floor creating a crawl space) should initially be performed to facilitate pile installation activities.

4.1 Timber Piles:

In accordance with the above discussion, it is anticipated that driven round treated timber piles will be selected to support the building frame and first floor slab structures.

It is recommended that the timber piles meet the requirements of ASTM D-25 for round timber tip bearing piles. The piles should be clean peeled and pressure treated in accordance with the requirements of AWPA C3. The timber pile design stresses should be established in accordance with ASTM D-2899 and the local applicable Building Codes. Additionally, we recommend the timber piles be treated with ACA (ammoniated copper arsenate) and CCA (copper chrome arsenate) due to the location of the proposed structure in a temperature zone coastal environment. Prior to driving, it is recommended that timber piles be relatively free of defects and have a water content greater than approximately 20 percent (to minimize "breaking") and less than about 50 percent (to minimize "brooming").

Driven timber piles will derive their long term capacity from shaft friction and end bearing support via embedment within the predominately clayey soils encountered at this site. These soil materials typically exhibit time-dependent strength characteristics, consequently shaft friction and end bearing support tend to increase from initial installation through a process termed "soil setup". Essentially, the dynamics of driving piles through these sandy materials will cause excess pore pressures to develop, thereby decreasing driving resistance during initial pile installation. The pile capacities developed during driving are usually much lower than the design values. However, once driving is complete, these pore pressures dissipate with time (and soil setup occurs) and the bearing capacity of the pile increases. Based upon our experience with similar projects in the area, 12 to 24 hours is usually required for the pore pressures to dissipate.

For the reasons described previously, it may not be possible to confirm pile capacities with a simple driving criteria such as number of hammer blows per foot of advanced pile. Instead, driving criteria will likely consist of a target tip elevation and/or certain embedded length in a bearing material with specified driving resistance. In order to confirm the required tip elevations, we recommend conducting a Test Pile Program prior to ordering production piles, as will be discussed later in this report.

4.2 Timber Pile Design Recommendations:

The following conclusions and recommendations are based on the previously discussed project information, our observations at the site, our interpretation of the field and laboratory data obtained during the exploration, and our experience with similar subsurface conditions. Subsurface conditions in unexplored locations may vary from those encountered at specific boring locations. If the proposed construction scheme should vary from that previously described, we request the opportunity of reviewing our recommendations. We conducted static capacity analyses on the timber piles using equations for shaft friction and end bearing that incorporated our experience and that of published information on displacement piles driven. Our individual pile axial capacity estimates versus pile embedment lengths are tabulated below:

Pile Type & Dimensions	Installation Method	Embedment Depth* (feet)	Allowable Capacity in Compression (tons)	Allowable Capacity in Tension (tons)	Allowable Lateral Capacity* (tons)
8″ Min. tip Diam. roundTimber	Driving	30	20	5	0.5

- Embedment depths determined from elevation 4 MSL.
- Lateral capacity computed for a lateral load applied at free pile butt level, at ground level for a maximum butt deflection of 1 inch. Batter piles would enhance stability.

The above shown pile embedment are required to achieve the allowable capacities. Any reduction in the length of embedment will correspond to a reduction in the allowable design capacities, unless otherwise directed by the geotechnical engineer after the pile testing program.

In order to minimize driving difficulties due to densification of the soils and the reduction in capacity due to group action of the piles, it is recommended that the piles be driven with a center-to-center spacing of at least 3 feet.

The piles should be advanced by driving with an impact hammer to their design tip elevations. If for some reason during construction, pile driving "refusal" is encountered before piles reach their design tip elevations, the Geotechnical Engineer should be retained to review driving records and field reports before assuming the pile can adequately support the design capacity. If the contractor's hammer is insufficiently sized, it may reach a "false" practical refusal before reaching desired tip elevations.

4.3 Pile Settlements:

Based on the results of load tests performed on piles driven in similar soils conditions, it is anticipated that the total butt settlements will not exceed about ½ inch, which is the settlement necessary to mobilize the soil/pile capacity.

4.4 Test Pile Program:

We recommend that a test pile program be implemented for the purpose of assisting in the development of final tip elevations and to confirm that the contractor's equipment and installation methods are acceptable. The test program should involve test or "indicator" piles to provide an indication of various driving and/or installation conditions. We recommend at least 2 indicator piles. It is important to note the relationship between the required testing and our design assumptions. We chose safety factors based upon the recommended pile testing program. We expect that the pile testing program will include primarily dynamic evaluation (WEAP). A static pile load test is not considered necessary at this site. The geotechnical Engineer should monitor the driving of these test piles. Depending on the resistance to penetration during initial driving, the geotechnical engineer may require some or all of the test piles to be re-tapped following a waiting period of a 12 to 24 hours to check for pile/soil setup.

The test pile lengths and locations should be selected by the geotechnical engineer. The piles should be driven using the drive system submitted by the contractor and approved by the geotechnical engineer. Test pile lengths should be 5 feet longer than the estimated design length to ensure that the required capacity is developed, to allow for refinement of estimated capacities, and for dynamic and static testing reasons. The indicator piles installed during the Test Pile Program which satisfy the geotechnical engineer's requirements for proper installation may also be used as permanent piles.

4.5 Production Pile/ Acceptance Criteria:

Utilizing the driving records for the tested piles and the results of the dynamic analysis of the driving conditions (WEAP), the pile capacity versus driving resistance (BPF) may be more accurately estimated; and depending on the results of these analysis an additional load test or some adjustment to the design capacity or depth may be necessary.

The piles should be installed to an embedment depth of 30 feet below the graded surface (elevation +/- 4 feet MSL), or as refined by the geotechnical engineer following the pile test program. These piles should be driven to the dynamic driving resistance as determined in the field by the Geotechnical Engineer, or to practical refusal, whichever comes first.

Based on our experience with pile installation, it is essential that driving be terminated immediately if refusal (i.e. 4 blows per inch or 48 blows per foot) is reached to prevent damage to the piles.. The pile driving hammer should have a minimum rated energy on the order of 15,000 foot-pounds per blow.

The installation of all deep foundation systems should be in accordance with the local Building Code requirements. In addition, the installation of all piles should be monitored by a qualified Geotechnical Engineer or representative. General pile installation guidelines are provided below.

1. The piles should be evaluated and designed for the axial stresses by the project structural engineer. All of the piles in a foundation element should be of the same type and of equal load carrying capacity.

2. The pile driving hammer should be capable of driving the pile to at least 3 times the design load without over-stressing the pile in tension or compression. At all times, the hammer should be operated at the chamber pressure, speed, etc. as recommended by the manufacturer.

3. Pile driving should be as continuous an operation as possible and should proceed without stopping over the last 3 feet of penetration. During pile installation, the contractor should exercise caution as not to over stress the piles. Piles shall not be driven beyond practical refusal (as defined in the VDOT Standard Specifications and in this report).

4. The contractor should exercise caution in pile driving so that the effects of heave are minimized. Careful monitoring by the

contractor for possible heave during driving should be exercised. All piles that heave ¼ inch or more should be re-driven unless otherwise instructed by the Geotechnical Engineer.

5. Piles should be installed under continuous monitoring by a qualified Senior Engineering Technician or Geotechnical Engineer in order to make field judgments of pile penetration and construction and to check for satisfactory foundation support conditions. Driven piles should be monitored for penetration, blow counts during driving and hammer action.

4.6 Adjacent Structures:

When considering the suitability of a driven pile foundation, consideration should be given to the integrity of nearby structures. Due to the large amount of energy required to install driven deep foundations, vibrations of considerable magnitude are generated. These vibrations may affect nearby structures. These structures can, due to their proximity, be detrimentally affected by the construction unless proper protection measures are taken. In addition, experience has shown that these construction features will often lead adjacent property owners to conclude that damage to their property has taken place, even though none has occurred. It is therefore recommended that a thorough survey of the adjacent property be made prior to starting construction. This will help to better evaluate real claims and refute groundless nuisance claims. The survey should include, but not be limited to, the following:

1. Visually inspect adjacent structures, noting and measuring all cracks and other signs of distress. Take photographs as needed.
2. Visually inspect adjacent pavements, noting and measure any significant cracks, depressions, etc. Take photographs as needed.
3. Establish several bench marks along foundation walls on adjacent structures. Both vertical and horizontal control should be employed.
4. Determine if equipment in any adjacent building is sensitive to vibration, and if so, establish proper control and monitoring system.

5.0 EVALUATION AND RECOMMENDATIONS: Elevated Water Tank

On the basis of the results of our soil test borings and the loading conditions of the proposed tank, it is our opinion that a mat foundation support system will be suitable for the elevated water tank. Should the design parameters presented herein (construction procedures, bearing capacity, and/or settlements) be deemed prohibitive for the tank, then the alternate foundation design schemes consisting of driven, 12-in square presetressed concrete piles should be applied (section **5.3** of this report).

5.1 Mat Foundation Design Recommendations:

Our analysis indicates that a mat foundation bearing at a depth of 8 to 9 feet below the existing site grades will develop an allowable soil bearing capacity of 2000 pounds per square foot (PSF). This determination is based on a 30-foot diameter round mat, with the ground water level at the finished surface.

In preparing the site, the foundation should extend laterally at least 5 feet beyond the perimeter of the mat radius (i. e. mat radius + 5 feet), and to a minimum depth of 2 feet below the mat design level. Subsequently, the mat design elevation should be re-established by means of backfilling the 2-foot zone of over excavation with well compacted #57 stone. A temporary dewatering system will be required to accomplish these activities. The structural design and sizing of the mat is the responsibility of the contractor/ manufacturer.

The mat foundation design should take sliding and overturning into consideration, thus properly sizing and embedding the mat to achieve proper safety factors against these failures. Additionally, during temporary wind loading conditions, the maximum bearing capacity may be allowed to reach 3000 psf, so long that the resultant reaction to the eccentric load is maintained within the middle third of the mat with no allowance for negative (uplift) pressures.

5.2 Mat Foundation Settlements:

The total settlements at the center of the mat are determined to reach a maximum of 1.25 inches; while the total settlements at the edges of the mat are expected to be on the order of about 1 inch. Accordingly, differential movement on the order of 0.25 inches should be expected to occur between center and edge of mat. Additionally, under full wind loading conditions, settlements at the receiving edge will reach 1.4 inches in magnitude. All settlements will be the result of granular (elastic) compression, and will fully occur as the loads are being applied.

5.3 Alternate Pile Foundations:

The elevated tank may alternately be designed to be supported by means of a deep foundation system comprised of driven concrete piles. This option will eliminate the need for dewatering and excavating to a depth of about 10 feet, and will result in a system that will yield minimal settlements. In this area, the most economical and the most common type of deep foundation is the driven pile. Typically, precast, prestressed concrete piles are the pile of choice for economic reasons.

The following paragraphs describe the pile capacity analyses and provide our recommendations for static axial compressive pile capacities, pile testing program, and pile construction criteria. In addition, we have provided estimates of settlement.

5.3.1 Axial Compression Capacity Recommendations:

We conducted pile capacity analyses using static formulas with coefficients recommended by Geoffrey Myerhoff and George Sowers. The analyses include the contributions of shaft friction and end bearing to the pile capacity. The piles are expected to derive the majority of their capacity from end bearing in a very dense sand layer generally between the depths of 27 and 32 feet below the existing grades.

The subsurface soils at the project site consists predominately of very loose to very dense silty sands from a depth of 7 feet to the boring termination depths of 75. These soil materials typically exhibit time-dependent strength characteristics, consequently shaft friction and end bearing support tend to increase from initial installation through a process termed "soil setup". Essentially, the dynamics of driving piles through these materials will cause excess pore pressures to develop, thereby decreasing driving resistance during initial pile installation. The pile capacities developed during driving are usually much lower than the design values. However, once driving is complete, these excess pore pressures dissipate with time (and soil setup occurs) and the bearing capacity of the pile increases. Based upon our experience with similar projects in the area, 48 to 72 hours is usually required for the pore pressures to dissipate and soil setup to occur.

For the reasons previously described, it will not be possible to confirm pile capacities with a simple driving criteria such as number of hammer blows per foot of advanced pile. Instead, driving criteria will likely consist of a target tip elevation and/or certain embedded length in a bearing material with specified driving resistance. The specified driving resistance should be based on a Wave Equation Analysis of the contractor's selected hammer, preferably with soil response data from dynamic monitoring with a Pile Driving Analyzer (PDA). In order to confirm the required tip elevations, we recommend conducting a Test Pile Program prior to ordering production piles.

The following table provides our recommended pile type and tip elevations for the foundations for the tank. The allowable capacity for the piles includes a safety factor of at least 2.5 to allow for a pile load test program that relies primarily on dynamic testing. The capacity of a group of piles spaced at least 3 pile diameters apart, center to center, can be taken as the sum of the individual capacities with no reduction factor. If closer pile spacings are anticipated, the geotechnical engineer should be contacted to evaluate the efficiency of the specific pile group. The final order lengths and tip elevations will be adjusted based on the results of the pile load test program.

Pile Type & Dimensions	Installation Method	Embedment Depth* (feet)	Allowable Capacity in Compression (tons)	Allowable Capacity in Tension (tons)	Allowable Lateral Capacity* (tons)
12" Square Prestressed Concrete	Driving	30	50	10	0.5

- Embedment depths determined from elevation 4 MSL.
- Lateral capacity computed for a lateral load applied at free pile butt level, at ground level for a maximum butt deflection of 1 inch. Batter piles would enhance stability.

Based on our interpretation of subsurface conditions, we do not recommend using jetting or predrilling techniques to aid installation of the piles. These techniques can have the effect of reducing shaft friction and end bearing and may adversely affect pile performance.

The piles should be advanced by driving with an impact hammer to their design tip elevations. If for some reason during construction, pile driving "refusal" is encountered before the piles reach their design tip elevations, the Geotechnical Engineer should be retained to review driving records and field reports to determine whether the pile can adequately support the design loads. If the pile driving hammer is not properly matched to the pile type, size and subsurface conditions, it may reach practical refusal before the pile reaches the design tip elevation, or the required capacity.

5.3.2 Pile Group Settlement:

Based on the results of load tests performed on piles driven in similar soils conditions, it is anticipated that the total butt settlements will not exceed about ½ inch, which is the settlement necessary to mobilize the soil/pile capacity.

5.3.3 Test Piles:

We recommend that a test pile program be implemented for the purpose of assisting in the development of final tip elevations and to confirm that the contractor's equipment and installation methods are acceptable. The test program should involve test piles to provide an indication of various driving and/or installation conditions. We recommend at least 3 test piles. It is important to note the relationship between the required testing and our design assumptions. We chose safety factors based upon the recommended pile testing program. We expect that the pile testing program will include primarily dynamic evaluation with a Pile Driving Analyzer (PDA).

The test pile lengths and locations should be selected by the geotechnical engineer. The piles should be driven using the drive system submitted by the contractor and approved by the geotechnical engineer. Test pile lengths should be at least five feet longer than anticipated production pile lengths to ensure that the required capacity is developed, to allow for refinement of estimated capacities, and for dynamic and static testing reasons. The indicator piles installed during the Test Pile Program which satisfy the geotechnical engineer's requirements for proper installation may also be used as permanent project piles.

The contractor should include in his equipment submittal a Wave Equation Analyses (using GRLWEAP™ software) modeling the behavior of the test piles during driving, or what is termed by GRL as a "Driveability Study." The primary intent of the Wave Equation Analyses is to estimate the feasibility of the contractor's proposed pile driving system with respect to installing the piles. Since the results of the Wave Equation Analyses are dependent on the chosen hammer, the pile type and length, and the subsurface conditions, it is likely that at least one Wave Equation Analysis per bridge will be required. Depending on the difference in size between the abutment and pier piles for a given bridge, and the subsurface conditions, separate Wave Equation Analyses may be required.

Pile driving equipment should not be mobilized for the test piles until the Wave Equation Analyses have been submitted and approved by the geotechnical engineer. If the contractor's proposed pile driving system is rejected, subsequent submittals of alternative drive systems should also include appropriate Wave Equation Analyses that are subject to the approval of the geotechnical engineer. The Wave Equation Analyses are also used to estimate:

- Compressive and tensile stresses experienced by the modeled pile during driving
- The total number of blows required to install the pile
- Driving resistance (in terms of blows per foot) within the various soil strata the pile is embedded in
- Driving time

The results of the WEAP analyses are highly dependent on the many input parameters related to the soil conditions, static pile capacity estimates, as well as specific characteristics associated with different makes and models of pile driving hammers.

5.3.4 Dynamic Testing:

Dynamic testing was developed as a method of improving upon the reliability of the wave equation and other dynamic predictions by actually measuring the acceleration and strain of a pile during driving. This technique was developed in the mid-1960's and has been continually refined. The use of dynamic pile testing has permitted the possibility of checking the driving stresses in the pile and the hammer performance during pile driving. It is also possible to estimate the static capacity of the pile based upon the strain and acceleration measurements taken during pile driving.

The test pile installation should be monitored by the Geotechnical Engineer using the PDA, an electronic device that records driving stresses and pile/soil interactions, among other things. The PDA results will confirm that the pile driving system (hammer type/energy, cushion type/ thickness, etc.) can successfully install the piles without over stressing them in compression or tension. We recommended the owner retain the services of the Geotechnical Engineer to perform the dynamic testing, not the contractor, to avoid possible conflicts of interest.

No sooner than 48 hours after installation, each test pile should be re-struck while being monitored with the PDA. This test establishes the "static capacity" of the pile. The initial hammer blow during re-strike activities is critical to the quality of dynamic data with respect to capacity interpretation. The contractor should make every effort to insure an initial high-energy blow of the hammer. After several blows during re-strike activities, pore pressures increase, soil setup diminishes, and ultimately pile capacities (as recorded by the PDA) decrease. Loss of estimated static capacity following repeated hammer blows is the reason the initial blows are critical.
The dynamic data recorded by the PDA during restrike testing should be further refined by using CAPWAP® analysis. CAPWAP® analysis, not the initial assessment of capacity determined by the PDA, should be the basis of static pile capacity estimates. Interpretation of CAPWAP® data, in the context of the soils subsurface conditions and previous static pile capacity estimates, should allow the Geotechnical Engineer to estimate ultimate pile capacities and recommend appropriate production pile lengths.

Our previous experience with the PDA indicates that a significant cost savings may be realized if the PDA is properly utilized to monitor the installation of test piles, confirm pile capacity in production installations, and monitor potentially damaging stresses during driving. The use of the PDA permits the confirmation of allowable compression and uplift capacities and pile integrity on several piles for a cost similar to or less than that of a single full-scale static load test.

5.3.5 Establishing Pile Driving Criteria:

Prior to driving production piles, the geotechnical engineer should establish the criteria for pile installation. The criteria will be based on the PDA data collected during dynamic monitoring of the initial drive of the test piles. The intent of establishing driving criteria is to facilitate installation of the production piles without damage and to provide a means of establishing when piles have achieved the design capacities. The driving criteria may include: hammer type, hammer energy, ram weight, pile cushion and thickness, hammer cushion type and thickness, required tip elevations and driving resistance necessary to achieve capacities, and possibly predrilling/jetting recommendations (if the test pile results warrant the need).

5.3.6 Allowable Driving Stresses:

Guidelines from the Prestressed Concrete Institute (PCI), American Society of Civil Engineers (ASCE), and the Association of State Highway Transportation Officials (AASHTO) indicate that maximum compressive stresses, imposed on driven precast concrete piles during installation, should be less than the following equation: 0.85 x f'_c (concrete compressive strength, psi) - f_{pe} (effective pre-stressing after losses from relaxation). The three groups differ on the maximum tensile stresses. PCI recommends 6 x square root of $f'_c + f_{pe}$; ASHTO and ASCE recommend 3 x square root $f'_c + f_{pe.}$ We recommend using the consensus value for the maximum compressive stress, and the ASCE/AASHTO recommended value for the maximum tensile stress.

5.3.7 Hammer Types and Energies:

In comparing hammers of equal energy, the Prestressed Concrete Institute (PCI) states that hammers with heavier rams and lower impact velocities are less likely to cause damaging stresses in concrete piles. Hammers with proportionally higher ram weights and short stroke heights (low impact velocities) are usually air or steam driven, and not diesel fueled. It has been our experience that air/stream hammers are more appropriate for the installation of precast concrete piles than similarly sized (in terms of energy) diesel hammers. We recommend that the contractor use an air or stream driven hammer whose ram weight is roughly equal to 0.5 to 1.0 times the weight of the pile itself. The actual determination of an acceptable ram weight should be determined through the results of the Test Pile Program. If the contractor elects to use a diesel hammer, we recommend a critical, detailed review of the contractor's

Wave Equation Analysis prior to driving the test piles.

5.3.8 Driven Pile Installation Monitoring:

The geotechnical engineer should observe the installation of the test piles and all production piles. The purpose of the geotechnical engineer's observations is to determine if production installations are being performed in accordance with the previously derived Pile Driving Criteria. Continuous driving and installation records should be maintained for all driven piles. Production piles should be driven utilizing the approved system established as a result of the Test Program.

The field duties of the geotechnical engineer (or a qualified engineer's representative) should include the following:

1. Being knowledgeable of the subsurface conditions at the site and the project-specific Pile Driving Criteria.

2. Being aware of aspects of the installation including type of pile driving equipment and pile installation tolerances.

3. Keeping an accurate record of pile installation and driving procedures.

4. Documenting that the piles are installed to the proper depth indicative of the intended bearing stratum. Also documenting that appropriate pile splicing techniques are used, if necessary.

5. Recording the number of hammer blows for each foot of driving.

6. Generally confirming that the pile driving equipment is operating as anticipated. Record the energy rating of the hammer.

7. Informing the geotechnical engineer of any unusual subsurface conditions or driving conditions.

8. Notifying the contractor and structural engineer when unanticipated difficulties or conditions are encountered.

9. Confirming from visual appearance that the piles are not damaged during installation and observing the piles prior to installation for defective workmanship. The geotechnical engineer should review all driving records prior to pile cap construction.

6.0 SITE WORK

This section of the report addresses the earthwork activities to be conducted at this site, including the roadway construction.

6.1 Subgrade Preparation:

It is recommended that the existing subgrade be lined with geotextile fabric (Mirafi 600X, or equivalent), prior to sand fill placement. A minimum of 2 feet of sand should be allowed to be placed over the fabric in order to provide a suitable bridging effect over the organic subgrade material. It is noted that fabric should not be placed in the treatment building area, where piles are planned to be driven.

6.2 Structural Fill and Placement:

Any material to be used for backfill or compacted fill should be evaluated and tested by G E T Solutions, Inc. prior to placement to determine if they are suitable for the intended use. Suitable structural fill material should consist of sand or gravel containing less than 20% by weight of fines (SP, SM, SW, GP, GW), having a liquid limit less than 20 and plastic limit less than 6, and should be free of rubble, organics, clay, debris and other unsuitable material. On-site sand materials are suitable for this purpose.

All structural fill should be compacted to a dry density of at least 100 percent of the standard Proctor maximum dry density (ASTM D698). In general, the compaction should be accomplished by placing the fill in maximum 10-inch loose lifts and mechanically compacting each lift to at least the specified minimum dry density. A representative of G E T Solutions, Inc. should perform field density tests on each lift as necessary to assure that adequate compaction is achieved.

Backfill material in utility trenches within the construction areas should consist of structural fill (as described above), and should be compacted to at least 100 percent of ASTM D698. This fill should be placed in 4 to 6 inch loose lifts when hand compaction equipment is used.

Care should be used when operating the compactors near existing structures to avoid transmission of the vibrations that could cause settlement damage or disturb occupants. In this regard, it is recommended that the vibratory roller remain at least 25 feet away from existing structures; these areas should be compacted with small, hand-operated compaction equipment.

Soils exposed in the bases of all satisfactory foundation excavations should be protected against any detrimental change in condition such as from physical disturbance, rain or frost. Surface run-off water should be drained away from the excavations and not be allowed to pond. If possible, all footing concrete should be placed the same day the excavation is made. If this is not possible, the footing excavations should be adequately protected.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Drainage and Groundwater Concerns:

It is expected that dewatering will be required for excavations that extend deeper than 2 feet below existing grades. Dewatering to a depth of 2 feet could probably be accomplished by pumping from sumps. Dewatering at depths below the groundwater level may require well pointing.

It would be advantageous to construct all fills (building area) early in the construction. If this is not accomplished, disturbance of the existing site drainage could result in collection of surface water in some areas, thus rendering these areas wet and very loose. Temporary drainage ditches should be employed by the contractor to accentuate drainage during construction.

7.2 Excavations:

In Federal Register, Volume 54, No. 209 (October, 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This ocument was issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that all excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new (OSHA) guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's responsible person, as defined in 29 CFR Part 1926, should evaluate the soil exposed

in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. G E T Solutions, Inc. is not assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

8.0 REPORT LIMITATIONS

The recommendations submitted are based on the available soil information obtained by G E T Solutions, Inc. and the information supplied by Hobbs, Upchurch & Associates, P. A for the proposed project. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, G E T Solutions, Inc. should be notified immediately to determine if changes in the foundation recommendations are required. If G E T Solutions, Inc. is not retained to perform these functions, G E T Solutions, Inc. can not be responsible for the impact of those conditions on the geotechnical recommendations for the project.

The Geotechnical Engineer warrants that the findings, recommendations, specifications or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete the Geotechnical Engineer should be provided the opportunity to review the final design plans and specifications to assure our engineering recommendations have been properly incorporated into the design documents, in order that the earthwork and foundation recommendations may be properly interpreted and implemented. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of Hobbs, Upchurch & Associates, P. A and their consultants for the specific application to the Proposed Stumpy Point Water Treatment Facility in Stumpy Point, North Carolina.

APPENDICES

I. BORING LOCATION PLAN

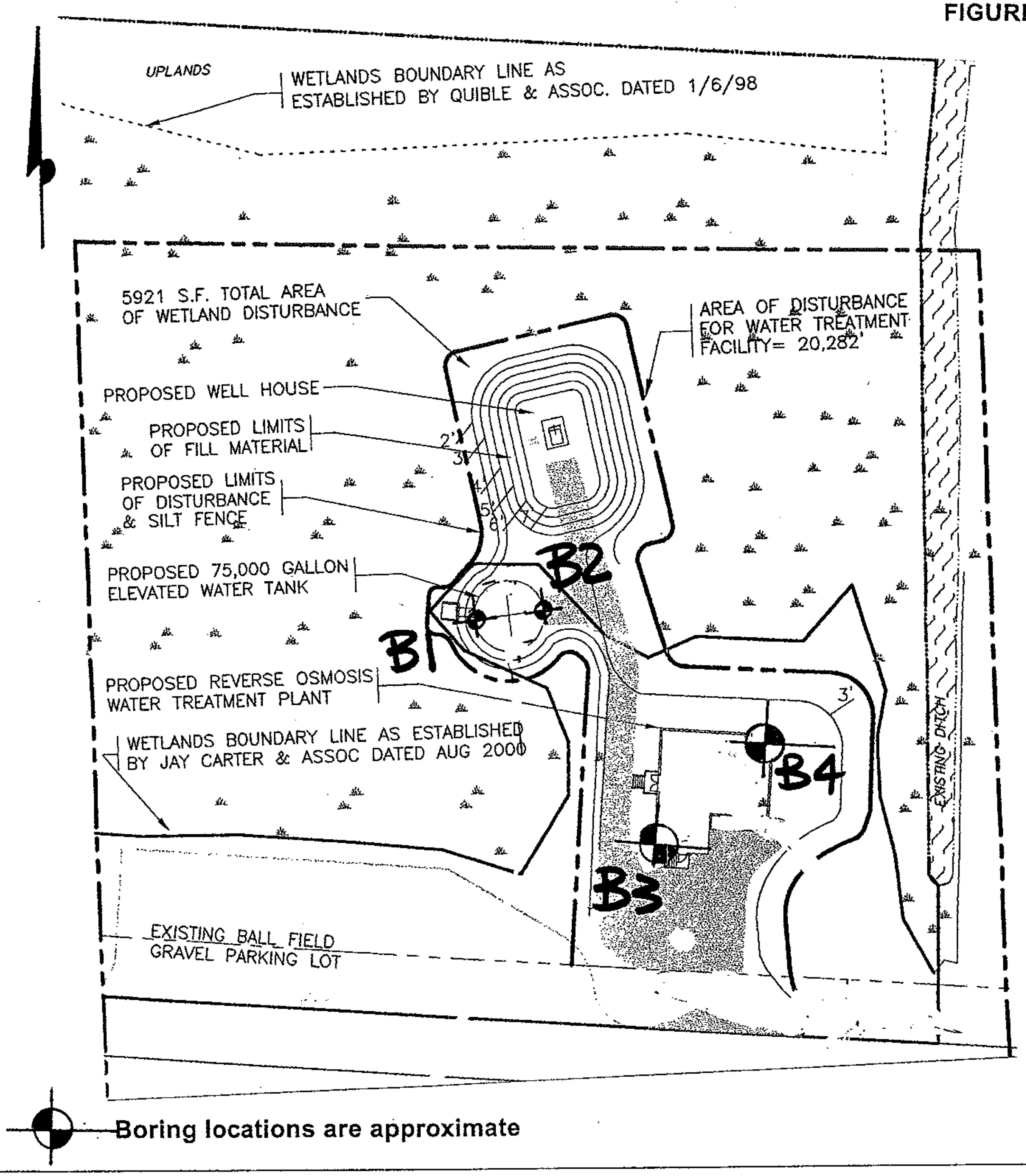
II. LOG OF BORINGS

III. GENERALIZED SOIL PROFILE

APPENDIX I -

BORING LOCATION PLAN

FIGURE 1



BORING LOCATION PLAN

PROJECT: Stumpy Point Water Treatment Facility
Stumpy Point, Dare County, NC
PROJECT NO: 01-106G
CLIENT: Hobbs, Upchurch & Associates, P. A.

SCALE: N/A

DATE: 03/21/2001

APPENDIX II -

LOG OF BORINGS

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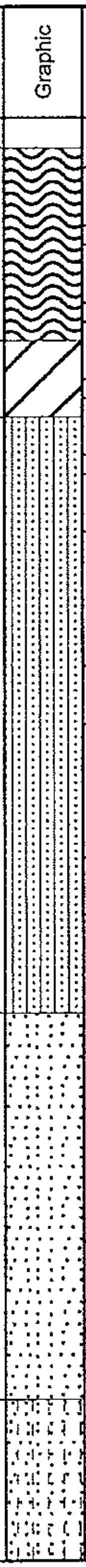
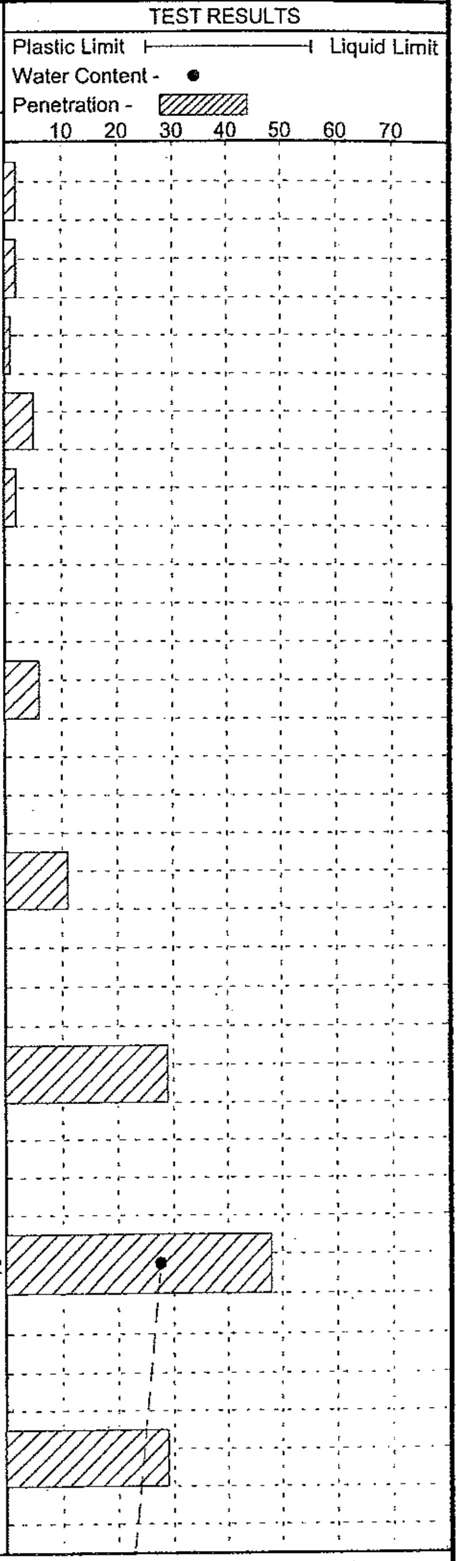
LOG OF BORING
No. B-1

PROJECT: Stumpy Point Water Treatment Facility **PROJECT NO.:** 01-106G
CLIENT: Hobbs, Upchurch & Associates, P. A.
PROJECT LOCATION: Stumpy Point, Dare County, North Carolina
LOCATION: Tank Foundation area, See "Boring Location Plan" Figure 1. **ELEVATION:** Grade
DRILLER: Fishburne **LOGGED BY:** CAK
DRILLING METHOD: Rotary Wash **DATE:** 2/05/2001
DEPTH TO - WATER> INITIAL: ▽ 0.5 feet **AFTER 24 HOURS:** ▽ 0.5 feet **CAVING>** C

Depth (feet)	Description	Sample No.	Blow Counts	% < #200
▽	Black, saturated, MUCK & organic clay (OH), very soft	1	2 1 1	
		2	1 1 1	
5	Gray, saturated, silty clay (CH) with some organics, very soft	3	1 0 1	
7	Gray, saturated, silty sand (SM), trace marine shell fragments, very loose to medium dense	4	1 3 2	
		5	1 1 1	
	Slightly silty sand (SP-SM) at 13 feet	6	2 4 2	
		7	5 6 5	
22.5	Light gray, saturated, fine to coarse sand (SP), trace marine shell fragments, medium dense to very dense	8	10 13 16	
		9	24 25 23	9.2
32.5	Gray, saturated, slightly silty sand (SP-SM), trace marine shell fragments, medium dense	10	8 14 15	





This information pertains only to this boring and should not be interpreted as being indicitive of the site.

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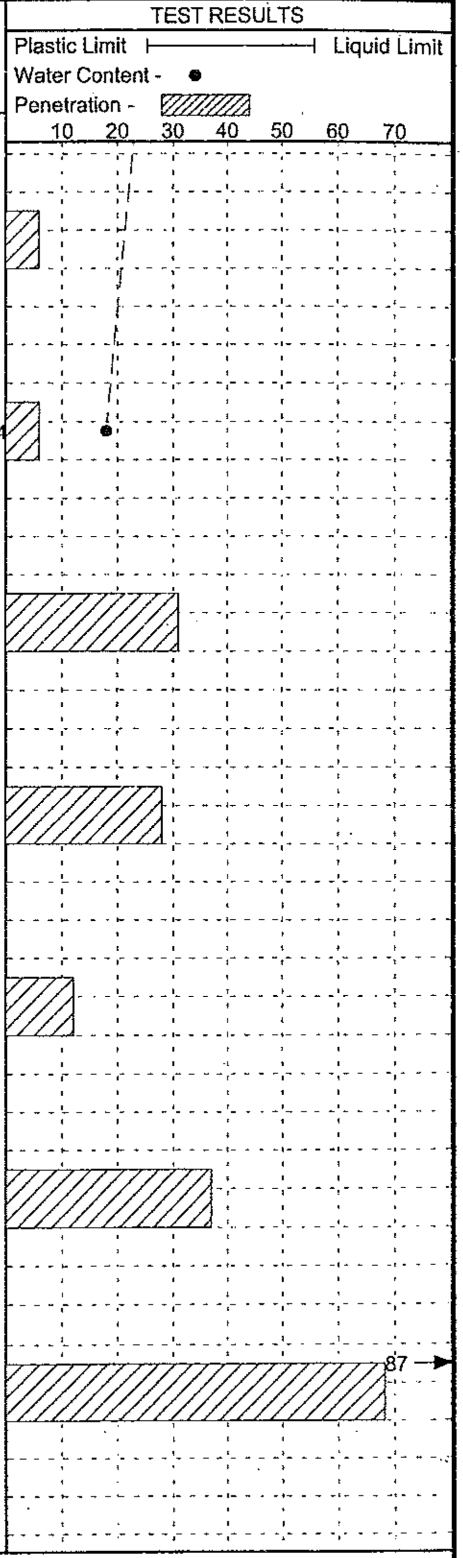
LOG OF BORING
No. B-1

PROJECT: Stumpy Point Water Treatment Facility **PROJECT NO.:** 01-106G
CLIENT: Hobbs, Upchurch & Associates, P. A.
PROJECT LOCATION: Stumpy Point, Dare County, North Carolina
LOCATION: Tank Foundation area, See "Boring Location Plan" Figure 1. **ELEVATION:** Grade
DRILLER: Fishburne **LOGGED BY:** CAK
DRILLING METHOD: Rotary Wash **DATE:** 2/05/2001
DEPTH TO - WATER> INITIAL: ▽ 0.5 feet **AFTER 24 HOURS:** ▼ 0.5 feet **CAVING>** C

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Depth (feet)	Description	Sample No.	Blow Counts	% < #200
37.5	Gray, saturated, silty clayey sand (SM- SC), trace marine shell fragments, loose			
40		11	3 3 3	
45		12	2 3 3	10.4
47.5	Greenish gray, saturated, silty sand (SM) to slightly silty sand (SP-SM), medium dense to very dense			
50		13	12 16 15	
55		14	9 12 16	
60		15	4 5 7	
65		16	15 18 19	
70		17	37 44 43	



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LOG OF BORING
No. B-1

PROJECT: Stumpy Point Water Treatment Facility **PROJECT NO.:** 01-106G
CLIENT: Hobbs, Upchurch & Associates, P. A.
PROJECT LOCATION: Stumpy Point, Dare County, North Carolina
LOCATION: Tank Foundation area, See "Boring Location Plan" Figure 1. **ELEVATION:** Grade
DRILLER: Fishburne **LOGGED BY:** CAK
DRILLING METHOD: Rotary Wash **DATE:** 2/05/2001
DEPTH TO - WATER> INITIAL: ▽ 0.5 feet **AFTER 24 HOURS:** ▼ 0.5 feet **CAVING>** C

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Depth (feet)	Description	Graphic	Sample No.	Blow Counts	% < #200	TEST RESULTS Plastic Limit ├──┤ Liquid Limit Water Content - ● Penetration - ▨ 10 20 30 40 50 60 70
		░	18	40 48 50/5"		▨ 98
75	Boring terminated at 75 ft..					
80						
85						
90						
95						
100						
105						
110						

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LOG OF BORING
No. B-2

PROJECT: Stumpy Point Water Treatment Facility **PROJECT NO.:** 01-106G

CLIENT: Hobbs, Upchurch & Associates, P. A.

PROJECT LOCATION: Stumpy Point, Dare County, North Carolina

LOCATION: Tank Foundation area, See "Boring Location Plan" Figure 1. **ELEVATION:** Grade

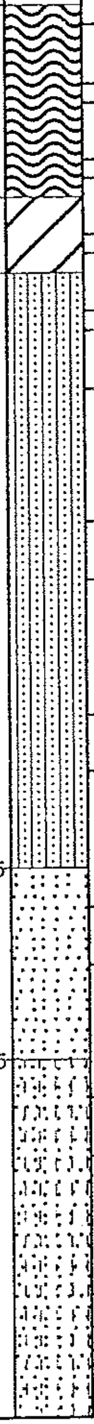
DRILLER: Fishburne **LOGGED BY:** CAK

DRILLING METHOD: Rotary Wash **DATE:** 2/05/2001

DEPTH TO - WATER> INITIAL: ▽ 0.5 feet **AFTER 24 HOURS:** ▽ 0.5 feet **CAVING>** C

Depth (feet)	Description	Sample No.	Blow Counts	% < #200
▽	Black, saturated, MUCK & organic clay (OH), very soft	1	1 1 1	
		2	0 1 0	
5	Gray, saturated, silty clay (CH) with some organics, very soft	3	1 1 1	
7	Gray, saturated, silty sand (SM), trace marine shell fragments, very loose to medium dense	4	2 2 1	
		5	2 3 2	
10				
15		6	2 3 3	
20		7	6 7 6	
22.5	Light gray, saturated, fine to coarse sand (SP), trace marine shell fragments, medium dense to very dense	8	11 12 9	
25				
27.5	Gray, saturated, slightly silty sand (SP-SM), trace marine shell fragments, medium dense	9	20 23 22	
30				
35		10	9 12 15	

Graphic

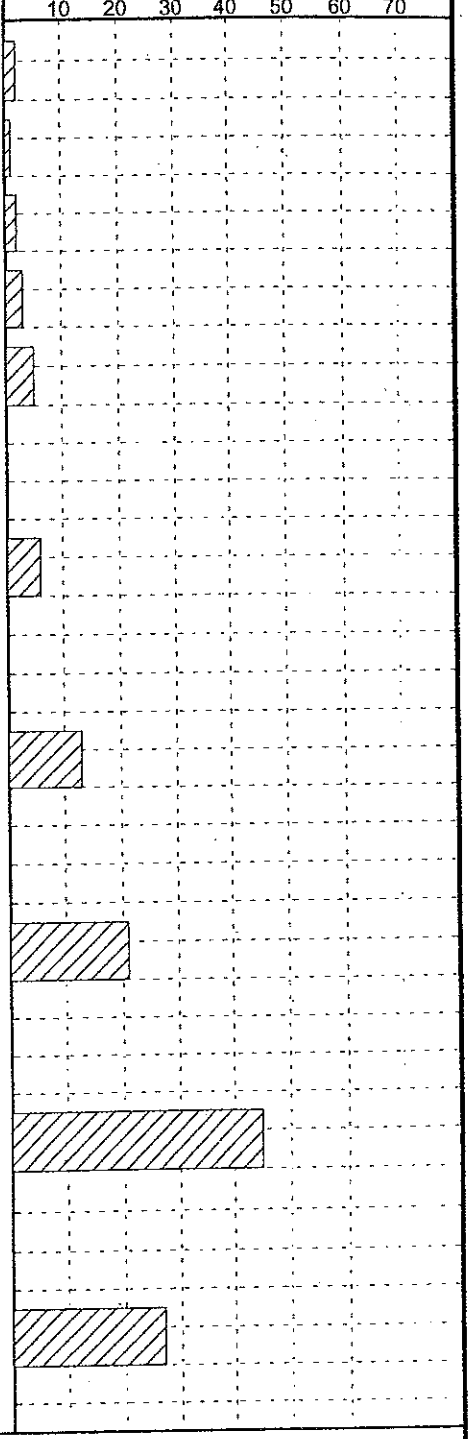


TEST RESULTS

Plastic Limit ├──┤ Liquid Limit

Water Content - ●

Penetration - ▨



This information pertains only to this boring and should not be interpreted as being indicitive of the site.

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Figure PAGE 1 of 3

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LOG OF BORING
No. B-2

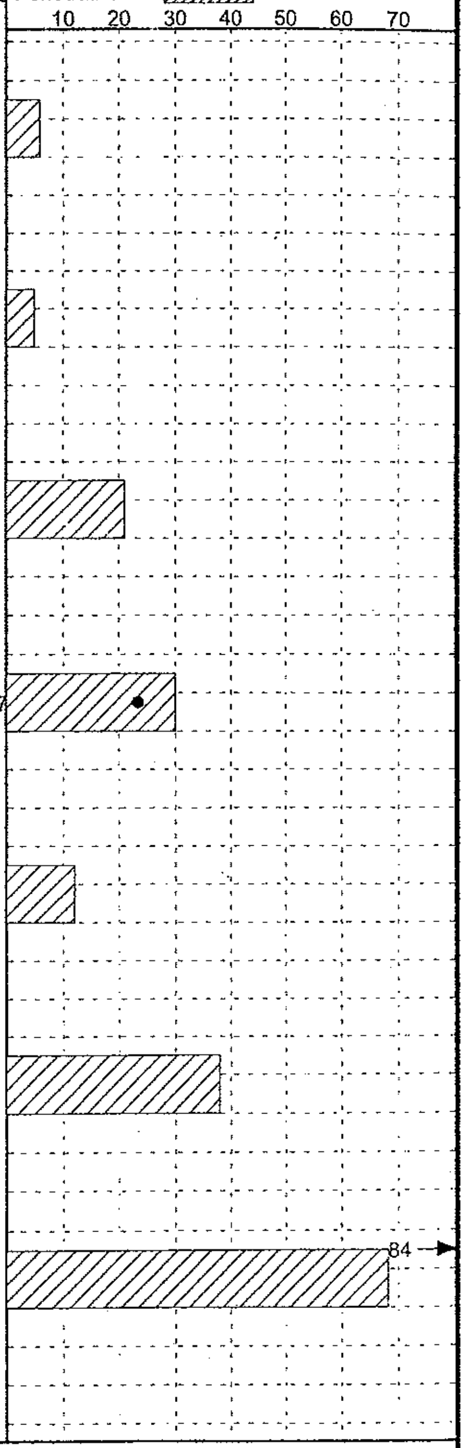
PROJECT: Stumpy Point Water Treatment Facility **PROJECT NO.:** 01-106G
CLIENT: Hobbs, Upchurch & Associates, P. A.
PROJECT LOCATION: Stumpy Point, Dare County, North Carolina
LOCATION: Tank Foundation area, See "Boring Location Plan" Figure 1. **ELEVATION:** Grade
DRILLER: Fishburne **LOGGED BY:** CAK
DRILLING METHOD: Rotary Wash **DATE:** 2/05/2001
DEPTH TO - WATER> INITIAL: ⊻ 0.5 feet **AFTER 24 HOURS:** ⊻ 0.5 feet **CAVING>** C

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Depth (feet)	Description	Sample No.	Blow Counts	% < #200
37.5	Greenish gray, saturated, silty sand (SM) to slightly silty sand (SP-SM), medium dense to very dense	11	3 3 3	
		12	2 3 2	
		13	4 9 12	
		14	9 13 17	10.7
		15	5 5 7	
		16	14 19 19	
		17	34 40 44	

40
45
50
55
60
65
70

TEST RESULTS
Plastic Limit ├──┤ Liquid Limit
Water Content - ●
Penetration - ▨



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Geotechnical
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LOG OF BORING
No. B-2

PROJECT: Stumpy Point Water Treatment Facility **PROJECT NO.:** 01-106G

CLIENT: Hobbs, Upchurch & Associates, P. A.

PROJECT LOCATION: Stumpy Point, Dare County, North Carolina

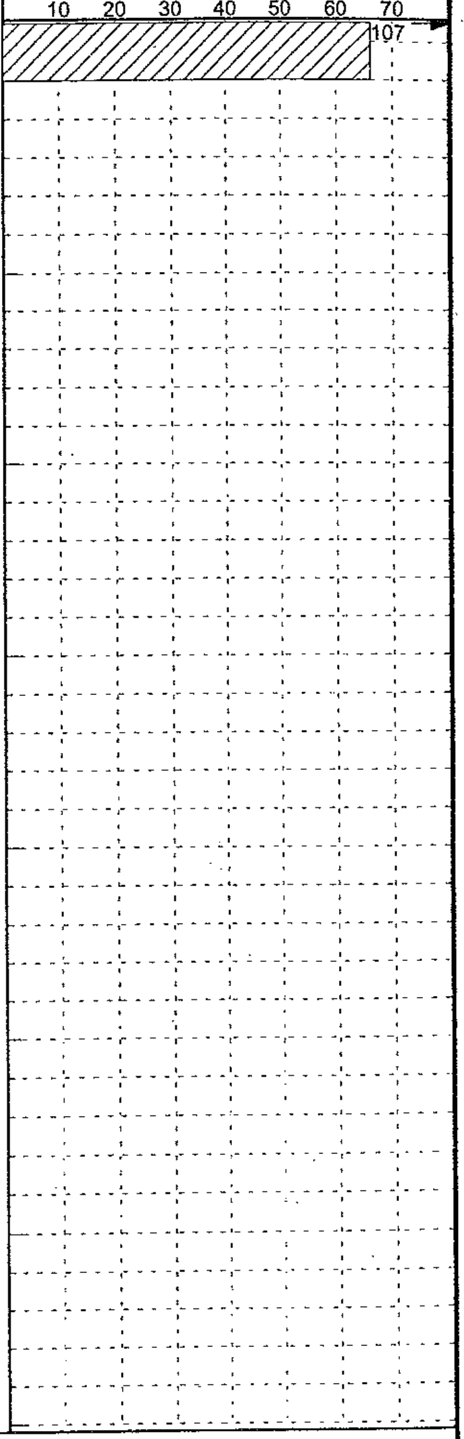
LOCATION: Tank Foundation area, See "Boring Location Plan" Figure 1. **ELEVATION:** Grade

DRILLER: Fishburne **LOGGED BY:** CAK

DRILLING METHOD: Rotary Wash **DATE:** 2/05/2001

DEPTH TO - WATER> INITIAL: ∇ 0.5 feet **AFTER 24 HOURS:** ▼ 0.5 feet **CAVING>** C

Depth (feet)	Description	Graphic	Sample No.	Blow Counts	% < #200	TEST RESULTS Plastic Limit ├──┤ Liquid Limit Water Content - ● Penetration - ▨ 10 20 30 40 50 60 70
75			18	39 57 50/4"		107
	Boring terminated at 75 ft.					
80						
85						
90						
95						
100						
105						
110						



This information pertains only to this boring and should not be interpreted as being indicitive of the site.

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LOG OF BORING
No. B-3

PROJECT: Stumpy Point Water Treatment Facility
PROJECT NO.: 01-106G
CLIENT: Hobbs, Upchurch & Associates, P. A.
PROJECT LOCATION: Stumpy Point, Dare County, North Carolina
LOCATION: Building area, See "Boring Location Plan" Figure 1.
ELEVATION: Grade
DRILLER: Fishburne
LOGGED BY: CAK
DRILLING METHOD: Rotary Wash
DATE: 2/05/2001
DEPTH TO - WATER> INITIAL: ▽ 0.5 feet **AFTER 24 HOURS:** ▼ 0.5 feet **CAVING>** C

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Depth (feet)	Description	Sample No.	Blow Counts	% < #200
0	Black, saturated, MUCK & organic clay (OH), very sot	1	1 2 1	
	solid wood	2	13 12 10	
5	Gray, saturated, silty clay (CH) with some organics, very soft	3	1 1 1	
7	Gray, saturated, silty sand (SM), trace marine shell fragments, very loose to medium dense	4	1 2 3	
		5	3 3 3	
		6	3 2 2	
		7	8 7 8	
22.5	Light gray, saturated, fine to coarse sand (SP), trace marine shell fragments, medium dense to very dense	8	9 14 21	
27.5	Gray, saturated, slightly silty sand (SP-SM), trace marine shell fragments, medium dense	9	12 14 13	
30	Boring terminated at 30 ft.			

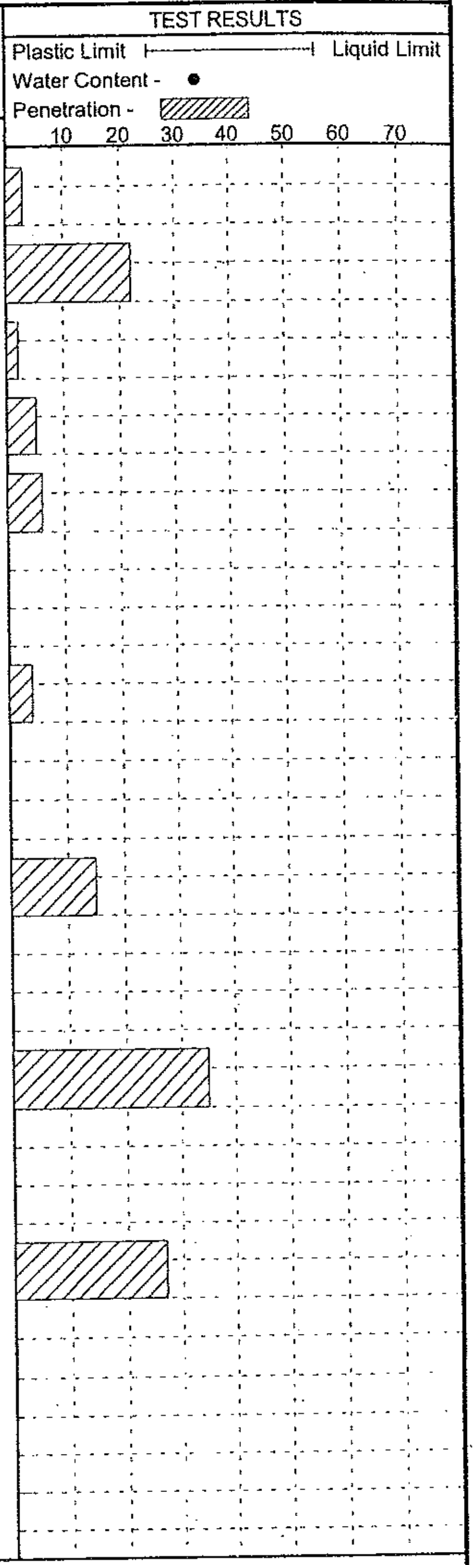

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Figure

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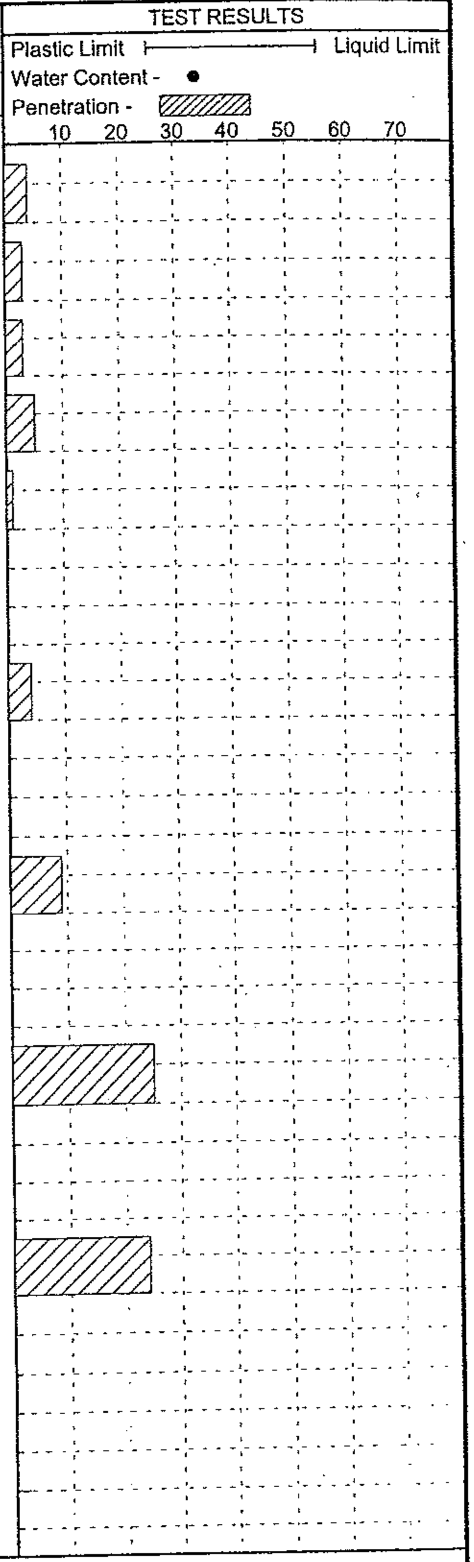
LOG OF BORING No. B-4

PROJECT: Stumpy Point Water Treatment Facility
PROJECT NO.: 01-106G
CLIENT: Hobbs, Upchurch & Associates, P. A.
PROJECT LOCATION: Stumpy Point, Dare County, North Carolina
LOCATION: Building area, See "Boring Location Plan" Figure 1.
ELEVATION: Grade
DRILLER: Fishburne
LOGGED BY: CAK
DRILLING METHOD: Rotary Wash
DATE: 2/05/2001
DEPTH TO - WATER> INITIAL: ▼ 0.5 feet AFTER 24 HOURS: ▼ 0.5 feet CAVING> ⊑

Depth (feet)	Description	Graphic	Sample No.	Blow Counts	% < #200
▼	Black, saturated, MUCK & organic clay (OH), very sot		1	2 2 2	
	solid wood		2	2 1 2	
5	Gray, saturated, silty clay (CH) with some organics, very soft		3	1 1 2	
7	Gray, saturated, silty sand (SM), trace marine shell fragments, very loose to medium dense		4	2 2 3	
10			5	3 0 1	
15			6	2 1 3	
20			7	4 4 5	
22.5	Light gray, saturated, fine to coarse sand (SP), trace marine shell fragments, medium dense to very dense		8	10 13 12	
25					
27.5	Gray, saturated, slightly silty sand (SP-SM), trace marine shell fragments, medium dense		9	10 11 13	
30	Boring terminated at 30 ft.				
35					



This information pertains only to this boring and should not be interpreted as being indicitive of the site.

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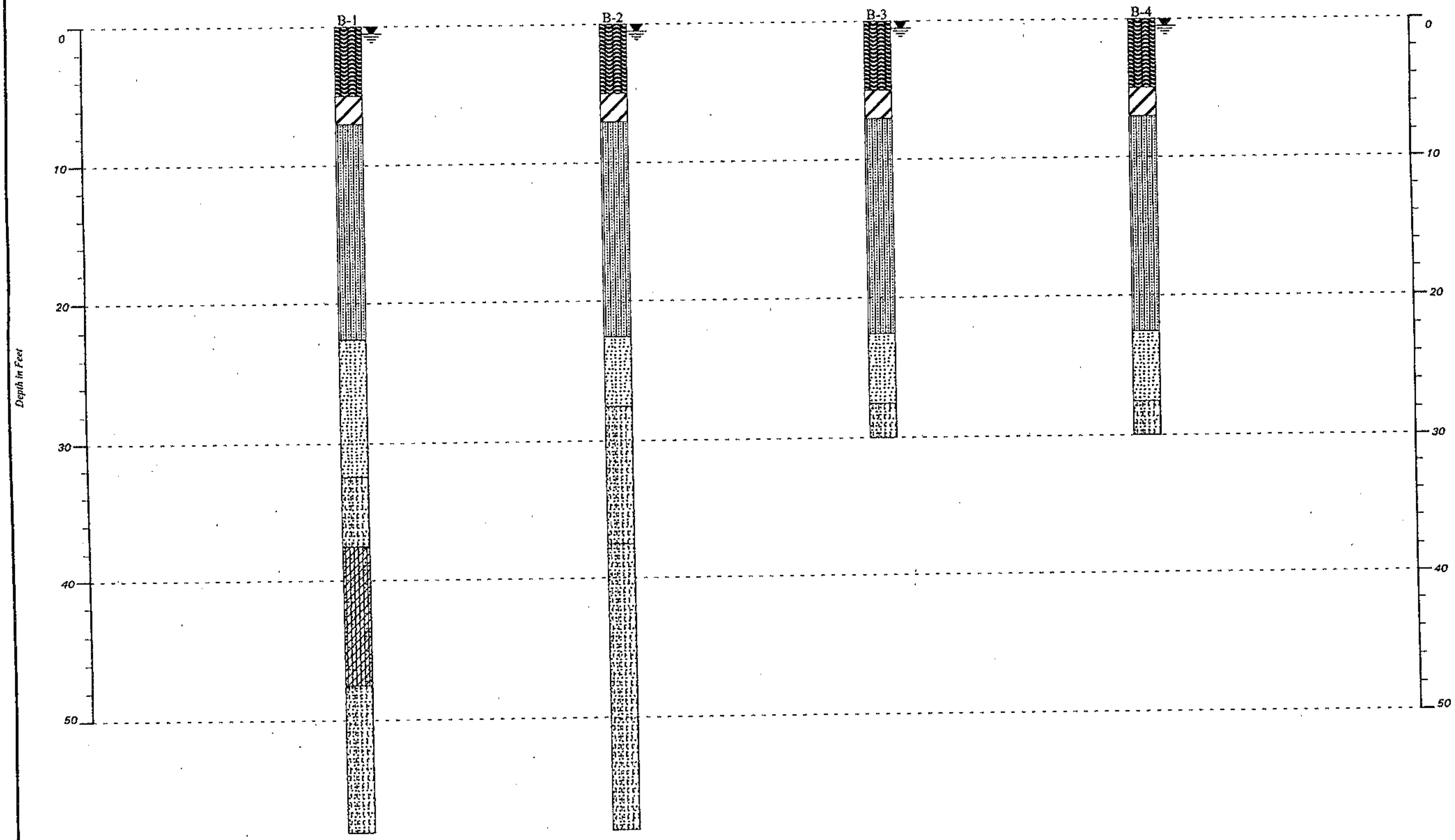
Figure

APPENDIX III -

GENERALIZED SOIL PROFILE



Strata symbols

- ▩ High plasticity organic clays
- ▨ High plasticity clay
- ▥ Silty sand
- ░ Poorly graded sand
- ░ Poorly graded sand with silt
- ▨ Poorly graded clayey silty sand

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GENERALIZED SOIL PROFILE

HORIZONTAL SCALE:	DRAWN BY/APPROVED BY	DATE DRAWN
VERTICAL SCALE: 1"=10'	CAK	3/22/01

Stumpy Point Water Treatment Facility

PROJECT NO. 01-106G	FIGURE NUMBER 1