Report of Subsurface Exploration
and Geotechnical Evaluation Services
Proposed Southern Oaks @ Telecom Park
Office Building Site
Tampa Telecom Business Park
Telecom Parkway East & Hollow Stump Road
Temple Terrace, Florida
HSA Project Number: 502-5358-00
November 19, 2007

HSA
ENGINEERS & SCIENTISTS
A member of the CRA Family of Companies

November 19, 2007

Ryan Companies, US, Inc.
101 East Kennedy Blvd., Suite 2450
Tampa, Florida 33602

<u>Via Electronic and Regular Mail</u>

Attn.: Mr. Brian Smith, Senior Project Manager

Subject: **Report of Subsurface Exploration**
and Geotechnical Evaluation Services
Proposed Southern Oaks @ Telecom Park Office Building Site
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Tampa, Florida
HSA Project Number: 502-5358-00

Dear Mr. Smith:

Pursuant to your authorization given on September 4, 2007, and in general accordance with our Proposal Number 502-5358-98, dated August 21, 2007, **HSA Engineers & Scientists (HSA)** has completed the exploration of subsurface soil conditions beneath the proposed Southern Oaks commercial office development, at the referenced site in Tampa Telecom Business Park. Authorization for the services in this study was provided by Mr. Brian Smith, Senior Project Manager of Ryan Companies, US, Inc., who instructed us to proceed with this study by a notice-to-proceed E-mail message to our Mr. Joseph A. Eduardo, P.E. on September 4, 2007. This notice-to-proceed E-mail message was confirmed by the issuance and mutual execution of Contract Number 1799-001-50-501165 K, a copy of which is included herein, as **Appendix D**.

The purposes of this exploration were to provide a design basis delineation of the stratification and engineering properties of subsurface soils, and to develop recommendations which address geotechnical issues related to the design and installation of foundations, pavements and utility infrastructure. In addition, this study was also performed to delineate recommended earthwork procedures and operations, which should be implemented during the construction period. This study addresses subgrade soil strata which lie well within the depth of influence of the expected magnitude of the loads that will be applied by the elements of this proposed development. In addition, this study

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included exploration of the surface of the underlying carbonate bedrock, in support of our assessment of the potential for the development of sinkhole subsidence incidents on the property.

EXECUTIVE SUMMARY

Our studies indicate that the proposed buildings, pavement, infrastructure utility lines, and other improvements associated with the proposed office buildings property may be installed on the property, provided that discovered sinkhole conditions are mitigated, where these occur below the proposed building areas and below other settlement-sensitive features of the development. Mitigation should consist of the injection of low to moderate slump cementicious grout into discovered sinkhole prone areas.

In performing this study, we have assumed that foundation loads may range from light (2,000 pounds per lineal foot, 50,000 pounds, and 100 pounds per square foot for linear, concentrated and area wide loads, respectively) to moderately heavy (10,000 pounds per lineal foot, 400,000 pounds, and 200 pounds per square foot for linear, concentrated and area wide loads, respectively). Preliminarily, we recommend that shallow foundations be used to support the proposed buildings and minor structures, which are designed using a net bearing pressure of 6,000 pounds per square foot, where earthwork will include undercutting of deep loose sand deposits, followed by conventional compaction of the undercut areas and replacement of the undercut volume with controlled compacted fill. It is expected that the soil that is removed in the undercut operation will be reused in the backfill operation. A net bearing pressure of 6,000 pounds per square foot can also be used if the near surface soils are improved using deep vibro-compaction techniques or high impact (dynamic) compaction techniques. The use of piles or drilled shafts to support building loads is not expected to be necessary.

Subsurface conditions were found to be favorable for the installation of economical pavement, to create the packing lots and the circulation paths that will be needed. Both asphalt and concrete pavements may be used, without any undue restriction, because of the discovered generally free-draining sand deposits that are expected to form the pavement subgrade, and the generally deep position of the ground water surface. Your selection may be based primarily on the basis of your need to control cost, promote durability, and increase longevity of the paved areas.

Please review the attached report where detailed discussions are contained, which describe the methods of exploration, the tests that were performed and the results of our geotechnical exploration, as well as more detailed conclusions and other important details not included in this summary. The report should be read in its entirety to obtain a more complete understanding of the information provided and to aid in any decisions made, or actions taken, based on this information.

PROJECT INFORMATION

Project Authorization

Authorization to proceed with this project was received from Mr. Brian Smith of Ryan Companies US, Inc., by his execution of Exhibit 1 of Contract Number 9020.779.999.221010, which refers to HSA Proposal No. 502-5358-98.

Project Location and Conditions

Location – The proposed site occupies approximately 18.4 acres, situated within a tract of land that lies in the southwest one-quarter of the northwest one-quarter of Section 12, Township 28 South, Range 19 East, Hillsborough County, Florida. More specifically, the subject property is a rectangular tract of land that occupies the northeast quadrant of the intersection of the proposed eastward extension of Telecom Parkway East at Hollow Stump Road, which is at the east end of the Tampa Telecom business park. The properties that flank the north and east boundaries of this site consist of residential developments. The property that flanks the south side of the site consists of low-density residential properties that appear to predate the installation of planned subdivisions, in this locality. The approximate location of the property, with respect to the local street map of this area, is depicted on **Figure 1**.

Topography – According to land surface altitude data that was gathered by King Engineering, Inc., and is shown on **Figure 3** herein, the land surface on this property descends due east from a high of about 48 feet NGVD at the Hollow Stump Road western boundary, to a low of about 28 feet NGVD, near the center of a ±4-foot deep, ±250-foot wide by ±600-foot long approximately elliptical depression in the land surface that occupies the area located below land surface altitudes of 33 feet NGVD. This feature occupies part of the western side of the proposed storm water management basin, and is deepest near the southeast corner of the parking lot, near the location of boring AB-19. The land surface rises radially from the edges of the depression area.

Vegetation – Nearly all of this property is vegetated with a moderately-dense to dense mature woodland, whose dominant species is oak. The under-story vegetation consists of common shade tolerant native shrubs, vines, and sapling trees, whose density is dependant on the light that is allowed to pass through the overlying tree canopy covers.

Project Description

As described in our August 21, 2007 proposal, we understand that the proposed development includes the construction of two four-story office buildings, each providing 110,000 square feet of office space and occupying a rectangular footprint of about 27,500 square feet. The development will also include the installation of a ±2.8-acre storm water management basin at the east end of the property. Most of the balance of the property will be paved to accommodate employee and visitor vehicles. Lastly, the development will include the installation of standard power, water, wastewater, communications, and other miscellaneous standard drainage and utility infrastructure. In performing this study, we understand that foundation loads may range from light (2,000 pounds per lineal foot, 50,000 pounds, and 100 pounds per square foot for linear, concentrated and area wide loads, respectively) to moderately heavy (10,000 pounds per lineal foot, 300,000 pounds, and 200 pounds per square foot for linear, concentrated and area wide loads, respectively).

PURPOSE AND SCOPE OF SERVICES

The purpose of our study was to evaluate general subsurface soil conditions located both within the depth of significant load influence of shallow spread footings and into the surface of the bedrock within the proposed building areas; to evaluate general subsurface soil conditions within areas that will be paved; and to evaluate general subsurface soil conditions within the proposed storm water management basin. The subsurface materials encountered were reviewed with respect to the available project characteristics. Engineering assessments of the following items have been formed:

1. Evaluation of the property, with the goal of identifying areas that may be at risk to develop sinkhole subsidence. Assessment of the qualitative risk for future sinkhole subsidence, based on the hydrogeologic conditions on the property and the conditions revealed in the soil borings that were drilled in Ground Penetrating Radar (GPR) anomalies. Preparation of a strategy and recommendations to mitigate sinkhole conditions to prevent the development of sinkhole subsidence below the proposed office buildings.

2. Determination of the feasibility of the use of shallow foundation systems to support the proposed office buildings. Development of foundation design and earthwork preparation recommendations to support the expected foundation loads. Estimation of the in-service performance of foundations that are designed and constructed as recommended.

3. If conventional shallow foundations do not appear to be feasible, determination of feasible alternatives to improve load supporting capability of the soil and/or determination of feasible alternate foundation support systems.

4. Development of pavement design and earthwork preparation recommendations to support the expected vehicular traffic.

5. Development of utility trench design and earthwork recommendations.

6. Assessment of the soils in relation to the impacts on the installation of infrastructure utility lines and recommended limitations of soil that may be excavated during earthwork operations on the property for incorporation into the project.

7. Estimation of the position of the seasonal high ground water level, in each soil boring that is performed in the proposed storm water management basin.

8. Report of infiltration test results on soils located within the proposed storm water management basin.

The following services have been provided in order to achieve the preceding objectives:

1. Initial reconnaissance of the site by a geotechnical engineer to document the condition of the site at the time of our exploration. The reconnaissance included recording the type and condition of vegetation, the existence and general condition of any structures on the site and adjacent to the site, the inclination and direction of site topography, site drainage features, and other particulars about the site which may be pertinent to this study;

2. Transect Pathway Clearing for Ground Penetrating Radar (GPR) Survey – Prior to the initiation of the gathering of data in the GPR survey, a bush hog mower was used to clear the under-story ground cover vegetation, along the selected GPR alignment paths, to allow the survey equipment to traverse the transect alignments, as necessary.

3. Ground Penetrating Radar Survey – A GPR survey of the entire property was performed to assess the lateral continuity of subsurface conditions on the property, and to detect the position and lateral extent of anomalous conditions that may be related to the presence of conditions conducive to the development of sinkhole subsidence on this property.

4. Assessment of the GPR profile cross sections, identification of suspect features, and rating of suspect features of interest. Selection of soil boring locations based on the compiled data and other site considerations.

5. Pathway Clearing for Drill Rig Access – Prior to the initiation of the soil borings, a bush hog mower was used to widen selected paths in the under-story ground cover vegetation, to allow the drill rigs to travel to the selected SPT boring locations, as necessary.
6. Flagging of proposed soil boring locations and notification of Sunshine State One-Call Center of the location, date, and nature of our proposed soil drilling operations. Resolution of any conflicts between the proposed location of the soil borings and the marked location of buried utility lines, as necessary, to maintain a margin of at least five feet between marked utility lines and the soil boring locations;
7. Performance of a series of twelve (12) Standard Penetration Test (SPT) borings (SPT-7 through SPT-18) to delineate the stratification and engineering properties of subsurface soils below the proposed location of the office buildings.
8. Performance of a series of six (6) Standard Penetration Test (SPT) borings (SPT-101 through SPT-106) to delineate the stratification and engineering properties of subsurface soils below the identified GPR anomaly locations, within the proposed footprint of the building areas.
9. Performance of a series of five (5) Standard Penetration Test (SPT) borings (P-1 through P-5) to delineate the stratification and engineering properties of subsurface soils within the confines of the proposed storm water management basin area.
10. Performance of a series of six, (6) Double-Ring Infiltration (DRI) tests within soils in the storm water management basin through which water will pass.
11. Excavation of soil within the DRI test pits to collect a series of four, (4) representative samples of soil that may be used as fill in the earthwork operations on the project site.
12. Performance of a series of nine (9) hand auger (AB) borings to delineate the stratification and engineering properties of near-surface subsurface soils, below pavement-covered areas on the property. Performance of Hand Cone Penetrometer (HCPT) soundings, combined with and in advance of the hand auger (AB) borings, to augment the stratification information collected in the borings;
13. Review of each soil sample obtained in our field testing program by a geotechnical engineer in our laboratory for verification of classification and assignment of laboratory tests, required.
14. Performance of laboratory soils classification tests on selected soil samples to confirm our visual soil classification of the site soils and to provide pertinent test data to estimate soil engineering parameters.
15. Analysis of the existing undeveloped wooded commercial property's soil and ground water conditions as they relate to the proposed development.

16. Preparation of this report to document the results of our field testing program, engineering analysis, pavement design recommendations, foundation design recommendations, and site earthwork recommendations.

The results of the explorations described herein have been used in the geotechnical engineering analyses and the formulation of recommendations. The results of the subsurface exploration, including the recommendations and the data on which they are based, are presented in this written report prepared by a Florida licensed engineer specializing in geotechnical engineering who is familiar with the local soil conditions.

INCORPORATION OF PREVIOUS STUDY INFORMATION

It should be understood that, in performing this study, we have incorporated the data that was gathered in a due-diligence study of this property (HSA Project No. 502-4466-00), as documented in our July 2, 2007 report, entitled *Report of Due-Diligence Subsurface Exploration and Geotechnical Evaluation Services*. This data included the performance of a GPR survey of the property, performance of series of seven (7) SPT borings, and performance of series of ten (10) AB borings, together with laboratory index testing of selected soil samples.

PUBLISHED DOCUMENT STUDY

As part of our study, we have reviewed the following documents:

- July 1, 1978, Temple Terrace, Florida 7.5-Minute Quadrangle Map, published by the United States Geological Survey (USGS)
- The May 1989 issue of the Soil Survey of Hillsborough County, Florida, published by the United States Department of Agriculture
- The 2005 issue of the Floridan Aquifer potentiometric surface maps, published by the United States Geological Survey (USGS) and Southwest Florida Water Management District (SWFWMD), for the months of May and September.

FIELD EXPLORATION

Ground Penetrating Radar Survey

HSA completed a supplemental Ground Penetrating Radar (GPR) survey within the limits of the proposed building footprints that compliments our previous study of the property, which was completed in July 2007 are approximately depicted in **Figure 2**. As before, the purposes of the GPR survey were to assess the stratigraphy below this property, and identify the position and lateral extent of subsurface anomalies that, in our experience, have frequently been indicative of the presence of underlying sinkhole-related karst features. The methodologies and equipment used in performing this GPR survey, as well as the detailed results and limitations of the survey, are described in the attached **Appendix A**.

Soil Borings

Our field operations, including the exploration performed in the due-diligence study, consisted of conducting a series of thirty (30) SPT borings using procedures similar to those outlined in ASTM D 1586. In addition, a series of nineteen (19) Hand Auger (AB) borings, and concurrent Hand Cone Penetrometer (HCPT) soundings in advance of the AB borings, were performed using procedures similar to those outlined in ASTM D 1452. The borings were performed at the locations indicated on the attached **Figure 3**. The test locations and depths of the borings were specified by our firm, as described in Proposal No. 502-5358-98, dated August 21, 2007. The borings were performed to determine the stratification and engineering properties of the subsurface soils below the proposed improvements planned for the Southern Oaks development. Borehole depths ranged from a minimum depth of 10 feet to a maximum depth of 65 feet below the existing ground surface. A continuous drilling and sampling procedure was performed within the upper 10 feet of the SPT boring and within depth intervals where the SPT N value is less than 4 blows per foot to detect subtle changes in soil stratigraphy and pertinent engineering properties within this critical depth.

The borings that lie within the building area footprints were located in the field by our drilling crew by visual reckoning using a site plan provided by King Engineering, Inc., having a scale of 1 inch = 100 feet and by tape measurement from 25-foot offset building corner stakes that were set by representatives of King Engineering, Inc. land survey staff. The remaining borings were located in the field by our drilling crew by visual reckoning using a recent aerial photograph, referenced to the Florida State Plane and UTM coordinate systems, having a scale of 1 inch = 100 feet and, selectively, by tape measurement from staked GPR transect end stakes. Land surface elevations at the soil boring locations were estimated by our interpolation of the land surface elevation contour lines that were developed by

King Engineering, Inc., with respect to the plotted positions of the soil borings on the property, as shown on **Figure 3**.

The accuracy of the boring locations and land surface altitudes is that implied by the measurement and estimation methods that were used. Upon completion, each auger borehole was filled in with local soil and the SPT boreholes were sealed with cement slurry, as required, in accordance with Southwest Florida Water Management District (SWFWMD) regulations. A brief summary of the drilling and testing procedures used in the borings is included in the attached **Appendix B**.

Double-Ring Infiltration Tests

In addition to the soil borings, this exploration has included the performance of a series of six (6) Double-Ring Infiltration (DRI) tests at the locations shown on the attached **Figure 3**. The purpose of these tests was to determine the rate of water infiltration through unsaturated granular soils. The DRI tests were performed using the procedures outlined in the current edition of ASTM Standard D 3385. Some minor modifications to those procedures may have been applied in performing these tests, due to field conditions which, in our judgment, have no effect on the results of the tests. The reported infiltration rate corresponds to the time rate linear slope of the test results at the end of the test, indicating a stabilized infiltration rate. The following table summarizes the results of the DRI tests, and pertinent data of the stratigraphy at each test location.

DRI Test Summary							
Test Number	**Soil Boring Profile**	**Test Depth Below Grade (Feet)**	**Estimated Land Surface Altitude (Feet NGVD)**	**Estimated Restrictive Layer Surface Altitude/ Classification (Feet NGVD)**	**Estimated Seasonal High Ground Water Depth/Altitude (Feet)**	**Total Elapsed Test Time (Minutes)**	**Stabilized Infiltration Rate (Inches Per Hour)**
DRI-1	P-1	3.0	34.1	22.1/SC	6.0/28.1	240	12.0
DRI-2	P-2	4.0	32.6	20.6/SC	4.5/28.1	240	4.08
DRI-3	P-3	3.0	32.0	28.0/SM	3.5/28.5	240	5.06
DRI-4	P-4	1.0	32.9	29.9/SC	3.0/29.9	240	12.4
DRI-5	P-5	5.0	32.0	25.5/SC	4.0/28.0	240	0.78
DRI-6	SPT-4B	1.0	34.6	32.1/SC	1.5/33.1	240	8.75

Bulk Soil Sampling

During the performance of the Double-Ring Infiltration (DRI) tests, a series of four (4) 75 to 100 pound bulk samples of soil were excavated by hand, placed into sample containers (5-gallon buckets), and transported to our Tampa, Florida laboratory. Specifically, samples were collected from the following locations and depth zones.

BULK SAMPLING OF POTENTIAL STRUCTURAL FILL MATERIALS				
Boring No.	**DRI No.**	**Sample Depth Interval (Feet)**	**Sample Description**	**USCS Classification**
P-1	DRI-1	1 – 2	Grayish-Brown Fine SAND	SP
P-2	DRI-2	4-1/2 – 5	Light Brown Fine SAND	SP
P-3	DRI-3	2 – 2-1/2	Grayish-Brown Fine SAND	SP
P-5	DRI-5	2 – 2-1/2	Grayish-Brown Fine SAND	SP

These samples were prepared and used in the performance of Modified Proctor (ASTM D 1557) moisture-density relationship testing, which is further described later in this report.

LABORATORY SERVICES AND TESTING

Visual and Tactile Review of Disturbed Soil Samples

The field soil boring logs and recovered soil samples were transported to our Tampa office from the project site. Following the completion of the field exploration activities, visual and tactile examination of each soil sample was performed by a geotechnical engineer in our Tampa, Florida soils laboratory, who assigned an engineering classification to the soil and rock samples that were retrieved in the field exploration. Based on this review, samples were selected and pertinent laboratory soil classification tests were assigned to these soil samples, in order to provide data with which to determine the accuracy of the assigned classifications. The visual classification of the samples was performed in accordance with the current Unified Soil Classification System (ASTM D 2487).

Moisture Content and Percent Fines Tests

A series of fifty (50) disturbed soil samples, which were collected during the performance of the SPT and AB soil borings (both studies), were selected, during the visual and tactile examination operations, and subjected to determination of their moisture content, using test procedures specified in ASTM

Standard D 2216. These same samples were retained, after determination of their moisture content, and subjected to further testing, to determine the percentage of soil that was capable of passing through a U.S. No. 200 sieve (percent fines content), using the soil washing procedures described in ASTM Standard D 1140. The results of these tests are documented on the attached soil boring profiles, in **Figures 4** through **12**, each plotted at the position where the sample was retrieved within the soil borings. The results of these tests are useful in confirming the engineer's visual classification of the soil, and they may also assist in predicting the behavior of these soils, when subjected to added stress and/or the addition or withdrawal of water.

Organic Content Tests

One (1) sample was selected from boring SPT-3 and subjected to testing procedures described in ASTM D 2974, for determination of its percentage of organic constituents. The result of the test is useful in confirming our visual classification of the soil, and in evaluating its compressibility. The results of the organic content tests are plotted, at the appropriate depth, adjacent to the soil boring profiles on **Figure 8**.

Atterberg Limits Testing

A series of nine (9) soil samples were selected from borings AB-2, SPT-4B, SPT-7, SPT-9, SPT-15, and SPT-16 for determination of their plasticity characteristics using the Atterberg Limits test procedure (ASTM D 4318). The results of these tests include determination of the liquid limit (LL), which is the water content at which the soil will close a standard gap, when subjected to a measured disturbance energy. Essentially, the LL value is associated with a water content above which the soil enters into a soft, flowing condition. The test series also includes determination of the plastic limit (PL) of the soil, which is essentially that moisture content below which the soil is no longer ductile (plastic). The plasticity index (PI) is defined as the difference between the LL and PL values, which is the range where the soil behaves in a ductile manner.

The results of the Atterberg Limits tests are plotted adjacent to the soil boring profile, at the associated sample depth, on the attached **Figures 8, 9**, and **10**. Correlations have been developed which relate these test results to various soil engineering parameters. The results of these tests were used to estimate soil compressibility characteristics, for this study.

Grain-Size Analysis Tests

A series of twenty (20) of the samples, which were subjected to the percent fines determination tests, were retained and tested further, to determine the distribution of sizes of the soil particles that comprised the coarse fraction (i.e., that portion of the sample that did not pass through the U.S. No. 200 sieve) of the samples, using procedures described in ASTM Standards D 421 and D 422. The results of these tests may be useful in the estimation and comparison the permeability characteristics of soils across the site. In addition, the results of these tests may provide guidance to construction professionals, in their selection of earthwork machinery that could be used to compact and/or excavate soils, as called for in the project plans. The results of the grain-size analysis tests are presented, graphically, plotted on the Grain Size Distribution Test Reports graph sheets, which are contained in the attached **Appendix C**.

Moisture Density Relations Tests

A series of four (4) samples were subjected to testing to determine the relationship between internal moisture and dry density, when subjected to measured compaction energy. Each of the four bulk samples that were obtained from the proposed storm water management basin were prepared and tested, using the procedures outlined in ASTM Standard D 1557 (Modified Proctor Moisture Density Test). The graphical relationship delineating the soil dry density with variation of moisture is shown on the curves that are attached in **Appendix C**. Cohesionless granular soil samples were selected for testing, since those materials are most useful as structural fill on this property. The following table summarizes the pertinent results of the testing of the soil samples

MODIFIED PROCTOR (ASTM D 1557) TEST RESULTS SUMMARY					
Boring No.	**Sample Depth Interval (Feet)**	**Sample Description**	**USCS Classification**	**Maximum Dry Density (Pounds per Cubic Foot)**	**Optimum Soil Moisture Content (%)**
P-1	1 – 2	Grayish-Brown Fine SAND	SP	107.2	13.3
P-2	4-1/2 – 5	Light Brown Fine SAND	SP	105.0	13.5
P-3	2 – 2-1/2	Grayish-Brown Fine SAND	SP	106.0	13.5
P-5	2 – 2-1/2	Grayish-Brown Fine SAND	SP	103.8	13.2

Based on the results of the testing, it is evident that the near surface sand deposits will compact readily, using conventional vibratory compactors, in the presence of abundant water. It is recommended that water be liberally applied to these soil deposits, when compacting. Flooding of these low-fines soil deposits with water may be considered, when compaction to depths in excess of 12 inches is desired.

SUBSURFACE CONDITIONS

Regional Surficial Soil Conditions

The U.S. Department of Agriculture – Soil Conservation Service has mapped the shallow soils in this area of Hillsborough County. This information was outlined in a report titled *The Soil Survey of Hillsborough County, Florida* dated May 1989. The aerial photographs used in the mapping were prepared in 1982 and show the proposed office site area to be wooded similar to its present condition. Most of the site appears to be located within the Candler fine sand, 0 to 5 percent slopes, mapping unit (mapping unit 7) and part of the eastern third of the property lies within the Zolfo fine sand (mapping unit 61). The approximate lateral extent of the mapped USDA soils is superimposed on the attached **Figure 3**.

The land surface of Candler fine sand, 0 to 5 percent slopes, in its undisturbed state, is nearly level to gently sloping. This soil is excessively drained. The position of the normal seasonal high ground water table is generally found below a depth of 80 inches. According to the USDA, the following stratigraphic profile is typically encountered in this soil. The surface layer of Candler fine sand consists of dark gray fine sand that is about 6 inches thick, which is underlain by light yellowish brown fine sand, to a depth of 35 inches. From depths of 35 to 72 inches, very pale brown fine sand is found. Below a depth of 72 inches, and extending to a depth of 80 inches or more, a mixture of very pale brown fine sand with strong brown loamy sand lamellae occurs.

In its natural state, the land surface of Zolfo fine sand is nearly level. This soil is somewhat poorly drained. The position of the normal seasonal high ground water table is generally found at a depth of 20 to 40 inches below grade in most years. The following stratigraphic profile is typically encountered in this soil. The surface layer of Zolfo fine sand consists of very dark gray fine sand that is about 3 inches thick, which is underlain by a layer of grayish brown fine sand to a depth of 15 inches. From about 15 inches to 51 inches below grade, a layer of light gray mottled fine sand is found which is underlain by grayish brown fine sand to about 60 inches below grade. Lastly, below 60 inches, a layer of dark brown fine sand is penetrated, which extends below a depth of 80 inches.

The information contained in the USDA Soil Survey is not necessarily representative of the soils on the site. The survey is, however, a good basis for general evaluation of the shallow soil conditions of the area. The soil survey mapping is based on interpretation of physiographic information shown on aerial photographs, which is confirmed by the performance of scattered shallow borings. Accordingly, borders between mapping units are considered to be approximate, and the change from one mapping unit area to the adjoining area may be transitional. Differences may also occur from the typical stratigraphy that is described above, and small areas of other similar and dissimilar soils may occur within the soil mapping unit. Consequently, there may be differences between the soil description published by the USDA and that which was discovered in the soil borings that were performed for this study. In general, the subsurface conditions that were revealed in the soil borings are consistent with those described in the USDA report, with the exception of conditions similar to those that were discovered in boring SPT-4B, where highly plastic clayey sand deposits were discovered starting at a depth of about 2-1/2 feet.

Ground Penetrating Radar Survey Summary

Briefly, and as shown on **Figure 2** herein, our interpretation of the GPR profiles, for this study, identified approximately seven (7) features of interest, most of which were interpreted to represent marker layer discontinuities or significant down-warping of the surface of the marker layer (typically the cohesive soil horizon). In our experience, such features may be created both by erosion forces and by the activation of geologic sinkhole conditions (i.e., the transport of unconsolidated sediments into cavernous bedrock zones). The performance of soil borings may reveal what geomorphic processes have produced these features. Accordingly, borings SPT-101, SPT-102, SPT-103, SPT-104, SPT-105 and SPT-106 were drilled near the center of GPR anomalies GPR-105, GPR-104, GPR-103, GPR-106, GPR-102 and GPR-101, respectively, to reveal the nature of the subsurface conditions where the survey profiles revealed conditions indicative of past sinkhole subsidence, which have not yet resulted in the development of a depression in the land surface.

Stratigraphic Conditions Summary

The delineation of the vertical extent of individual soil strata, the identification of pertinent soil engineering properties, where applicable, and a description of each geologic layer discovered in the course of our exploration, is given in the final soil boring profiles illustrated on the attached **Figures 4** through **12**. The final soil boring profiles were prepared by a geotechnical engineer based upon a combination of his technical review of the field soil boring logs, his visual classification of the recovered soil samples, and his review of the results of laboratory soils tests, performed for this study, as described above. The stratification lines shown are used to indicate a transition from one soil type to another. The actual boundary between the illustrated soil and rock layers may be gradual, or indistinct. Consequently, the stratification boundary lines, shown on the final soil boring profiles, represent our best estimate of the location of the transition between distinct geologic layers, and they are in no way intended to designate a depth of exact geological change.

The recommendations contained in this report are based on the contents of the final soil boring profiles. While the borings are representative of subsurface conditions at their respective locations and vertical reaches, local variations which are characteristic of the subsurface materials of the region, or which may be due to man-made alteration of the native geologic conditions, may be encountered.

The subsurface conditions, based on the data obtained from thirty (30) SPT borings and nineteen (19) AB borings, is generally described below:

> The explorations reveal that this property was underlain by a variably-thick sequence of sedimentary sand, silt and clay deposits that lie upon the surface of a dissected limestone bedrock mass. Initially, the land surface is underlain by generally cohesionless deposits of uniformly-graded, fine-grained sand. Typically, the upper veneer of soil consists of a grayish brown sand with silt deposit, which nominally serves as the vegetation support (topsoil) soil layer. It was present in most of the borings that were drilled in this study. The topsoil is underlain by cohesionless sand deposits, most of which contained minor silt fractions of less than 10 percent. These soils extend to depths ranging from a low of about 2-1/2 feet at boring SPT-4B, to as deep as about 37 feet in boring SPT -2, and include strata numbers 1 through 8 and 10 through 12. The soil layers that are identified as silty sand were generally penetrated near the base of the cohesionless sand zone, and were occasionally incorporated as minor components in the sand deposits with predominantly lower fines contents.
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> Below the cohesionless sand deposits, clayey sand layers were penetrated, whose thickness ranged between about 3 feet in boring SPT-6 to over 18 feet in boring SPT-16, where that boring was terminated at a depth of 30 feet. Hand auger borings AB-2 and AB-4 were terminated at a depth of 10 feet in these clayey sand deposits, while the balance of the auger borings were terminated in the cohesionless soil deposits at a depth of 10 feet. Cohesive soil deposits were absent in boring SPT-11, and boring SPT-18 was terminated at 30 feet without penetrating any cohesive soil layer.
>
> A discontinuous layer of light gray, calcareous clayey elastic silt generally underlies the cohesive soil deposits and overlies the surface of the limestone bedrock mass. Limestone fragments and thin lenses were often included in this material. The consistency of this soil

layer is highly variable, as indicated by the wide range of recorded penetration resistance values in this material. In particular, the penetration resistance of this material in borings SPT-6 and SPT-11 indicates a high level of disturbance, producing a soil layer with nearly no resistance to the penetration of the drilling rods and sampling tools. In borings SPT-1 and SPT-3, a layer of cohesive soil was found to underlie the elastic silt layer.

The surface of the carbonate (limestone) bedrock was penetrated in the SPT soil borings at depths ranging between about 18-1/2 feet in boring SPT-4B and 52 feet in boring SPT-3. The bedrock is a predominantly white limestone that was generally very hard and massive where it was not corroded. Significant corrosion of the bedrock was identified in borings SPT-1, SPT-2, SPT-5 and SPT-6. Elsewhere, in the SPT-100 series soil borings that were performed to explore the identified GPR anomalies, the bedrock mass appeared to be generally intact within the explored depths.

In general, the borings reveal that the near surface deposits of sand are very loose to loose in consistency. The density of the sand deposits, as indicated by the recorded penetration resistance values, commonly increases with depth, below about 8 to 10 feet below grade. Exceptions to this general pattern were discovered in borings SPT-1 and SPT-4B, where recorded penetration resistances indicate that the sand in the upper 8 to 10 feet was medium-dense to dense.

With the exception of the conditions revealed in soil borings SPT-3, SPT-4A, SPT-101, SPT-103, SPT-104, and SPT-106, the borings that were drilled in the identified GPR anomalies (both studies) discovered some highly disturbed soil and/or corroded bedrock zones that are indicative of conditions that are commonly considered to be consistent with past sinkhole activity. However, we note that with the exception of an evident shallow depression of the land surface at boring SPT-4A, no surface expression of the underlying sinkhole conditions has developed below the identified GPR anomaly areas. Moreover, the penetration resistance record in the borings suggests that the soils in the upper regions of the subgrade of this property have not been disturbed to any detectable degree, using the SPT boring procedures. Because no evidence of sinkhole conditions was found in boring SPT-4A, it is our opinion that the GPR anomaly may represent surficial erosion of the marker soil layer. Similarly, it appears that much the same can be said for the other anomaly soil borings, where evidence of very weak, disturbed soil conditions was not found. Lastly, it should be noted that apparently disturbed soil conditions were discovered in borings SPT-11, SPT-13, and SPT-18 were encountered, beyond the limits of the GPR-identified anomaly areas.

In general, the borings in the proposed storm water management basin reveal that the basin is underlain by between about 2-1/2 and 12 feet of low-fines rapidly draining sand deposits, lying upon less permeable deposits of silty, fine grained sand or clayey sand. We expect that the permeability of the silty and clayey sand deposits will be at least one order of magnitude lower than that of the surficial cohesionless sand deposits.

GROUND WATER CONDITIONS

The ground water level readings were measured in the borehole upon completion of testing and at the completion of each day's field work, where possible. The measured borehole ground water levels are plotted adjacent to the final soil profiles. These water level readings may differ from the actual stable ground water table due to variations in the permeability of soil layers. The degree of accuracy of the reported water levels is also related to the time allowed for the borehole water level to come to equilibrium. Consequently, if a water table is not indicated, it does not necessarily mean that ground water does not exist within the vertical reach of the borehole. Finally, it must be noted that fluctuations in the ground water level may occur due to variations in rainfall and other environmental or physical factors at the time measurements are made.

Unconfined (Surficial) Aquifer Conditions

The depth to the phreatic surface of surficial aquifer was detected in soil borings SPT-1 and SPT-2, at depths of about 14 and 15 feet, respectively. Based on indications in the boring logs and the apparent saturation of soil samples, we estimate that the depth to the water surface in borings SPT-3 and SPT-6 was about 15 and 13 feet, respectively. Water level depths were also detected in borings AB-14, AB-16, AB-17, and AB-19. We suspect that the low permeability of the soils in the remaining SPT boreholes prevented a stabilized water body surface from being detected. In general, the hand auger borings performed earlier in the year were likely too shallow to penetrate the surface of the water body below this property. The recently-performed hand auger borings did apparently penetrate saturated soils, indicating that the water in the surficial aquifer had risen in response to the summer rains, suggesting that those water surface levels may have been approaching normal seasonal high ground water levels.

The SPT borings were drilled using mud-rotary methods, which generally preclude determination of the depth to a water surface, after mud is introduced in the bore hole. Furthermore, in compliance with water management drilling regulations, upon completion, the SPT bore holes were filled with cement grout and/or bentonite clay pellets. Consequently, we were unable to measure a stabilized water level, in many of the SPT borings.

The range of fluctuation of the unconfined aquifer water table is largely dependent on the volume of rain water which infiltrates through the soil at any given time during the year. The altitude of the phreatic surface of the surficial aquifer has been found to peak annually in the period from late August to late-September, which is near the end of the three-month to four-month long period when 60 to 70 percent of the yearly precipitation falls. Conversely, the lowest annual piezometric level is commonly recorded in May, just prior to the commencement of the summer season rains. The recorded and estimated ground water levels correspond to conditions recorded in the borings near the end of the arid season and we judge that these water levels may be approaching (or possibly below) the normal seasonal low water level. Fluctuation of the phreatic surface of the surficial aquifer commonly ranges several feet between the high and low level within the thick cohesionless Candler sand deposits. A narrower range of fluctuation is expected to occur in the Zolfo sand deposits.

Confined (Floridan) Aquifer Conditions

All of the SPT borings penetrated through the low-permeability cohesive soil layers that typically separate the waters in the surficial aquifer from the waters in the confined aquifer. Drilling fluid circulation loss events were recorded in all deep borings that penetrated the bedrock surface, with the exception of borings SPT-3 and SPT-8. These loss events occurred most commonly near to the interface of the sediments with the surface of the bedrock.

Based on the potentiometric water surface levels that were published for the Floridan aquifer, in 2005, it is expected that the confined aquifer potentiometric water levels range between about 22 and 25 feet NGVD, corresponding to depths ranging between about 5 and 25 feet below the land surface on this property. Based on the estimated depth to the surface of the unconfined (surficial) aquifer, it appears that the head difference between the surficial and Floridan aquifers may be negative (i.e. artesian conditions) in the east end of the property, and positive where the land surface altitude rises above about 40 feet NGVD, in the western third of the land. This is demonstrated by the apparent ground water surface levels that were recently measured. Below land surface elevation of 40 feet NGVD, the water level measured in borings AB-14, AB-17, and AB-19 indicate a surficial aquifer level ranging between about 23-1/2 and 26-1/2 feet NGVD, which is nearly equivalent to the expected wet season potentiometric surface of the bedrock (Floridan) aquifer. Similarly, the indicated water levels in the SPT borings indicate a water surface in the range of 20 to 25 feet NGVD.

GEOTECHNICAL EVALUATIONS

Sinkhole Subsidence Potential

Sinkhole Formation – There are three distinct types of sinkholes which have developed in Florida. The first type is the classical *collapse sink*, which is generally steep-sided and rocky. It occurs when a cavity can no longer support the weight of the overlying soil and rock. This type of sink generally occurs when the limestone is at or near the surface and solution weathering is still very active. It is unlikely that cavities in older, deeper ancient rocks at a great depth below the surface, which have undergone much more intensive solution weathering in the past, are large enough to cause a deep-seated roof collapse. A cavity which is large enough to have caused a roof collapse would have done so when it was closer to the surface and the beam action or arching effect of the overlying formation was not as great as it is today.

The second type of sink, which is more common, though not as dramatic as the collapse sink, is called a *doline* or *solution sink*. There is no physical disturbance of the soluble rock beneath a doline. Subsidence of the overlying soil occurs due to gradual lowering of the rock surface and/or the gradual dissolution of leaching of calcium carbonate from the calcareous soil and rock which exists between the ground surface and the underlying aquifers. (The Florida Geological Survey estimates that this type of subsidence occurs at the rate of one foot every five to six thousand years). Because the water flows radially to the intersection of vertical joints where the water enters the rock mass, the surface expression of the rock lowering or the leaching of the soluble soil constituents is a shallow depression located over the intersection of the joints. In some cases, the surface depression has the same shape as the original calcareous deposit, as the case of a shell bed which has dissolved or partially dissolved since deposition.

The third type of sinkhole and probably the most common type of sink occurring in Florida is the *erosion sink*. Erosion sinks most frequently occur in an environment with the following characteristics:

- Limestone strata overlain by relatively pervious unconsolidated sediments; i.e., sandy soils;
- Cavity systems present in the limestone;
- A surficial water table higher than the potentiometric surface in the underlying limestone; and/or
- A breach of the limestone into the cavernous zone creating a point of high recharge to the artesian aquifer.

Under these circumstances water moving down into the limestone may take carry amounts of sediment into the cavernous system creating a void in the overlying sediment. When the void in the overlying sediment reaches the size where the roof is no longer stable, the overburden will collapse. In many cases the overburden is visible after the collapse, but some sinks of this type have occurred in which the collapsed overburden disappeared into the cavity system. In other cases the subsidence of the ground surface is only six inches to one foot deep.

Sinkholes that develop in this locality are generally the erosion sinkhole variety, where the sandy deposits are carried into underlying rock cavity systems by the force created by the difference in hydraulic head between the surficial and bedrock aquifers.

Discovered Property Conditions – Our review of the topography of the land surface indicates that this property has undergone little localized past subsidence of the land surface. The eastern half of the property, however, does contain a broad elliptical depression that is about 4 feet deep. A small shallow depression in the land surface was discovered and boring SPT-4A was drilled within that depression area. As discussed in the sections of this report that describe the subsurface conditions, the SPT soil borings that were performed in the identified GPR anomaly areas revealed that the GPR survey may have identified locations where migration of soils and apparent disturbance (raveling) has occurred. Where no significant thickness of disturbed soil was found (such as at borings SPT-3, SPT-4A, SPT-101, SPT-103, SPT-104, and SPT-106), it appears that the GPR has identified locations where the reflective surface had sustained surface erosion. In general, the condition of the soil sediments, located within the upper ± 20 feet, appear to have been unaffected by the presence of the disturbed soils, and the inferred bedrock void zones, located below this property. The discovery of apparent abnormally weak soil conditions at locations that were not identified by the GPR survey is somewhat troublesome, in that it is evident that we are unable to rely entirely on the GPR survey, in order to plan a program to mitigate the apparent sinkhole conditions that underlie portions of the proposed building areas.

Qualitative Sinkhole Development Potential – Regarding the potential for the development of future sinkhole subsidence on this property, based on the discovered geologic and hydrogeologic conditions, it is our opinion that the potential is best described as low. This opinion is based primarily on the demonstrated low or absent hydraulic gradient between the surficial and bedrock aquifers. Based on the recoded and estimated depths to the ground water surface, the altitude of the phreatic surface of the surficial aquifer may be a highly muted reflection of the topography of the land surface, existing mostly within a range of between 20 and 30 feet NGVD, throughout the year. Therefore, in the upland areas, the potential for the development of sinkhole subsidence is generally greater than the potential in areas of lower land surface altitude, because of the generally increasing altitude of the phreatic surface of the surficial aquifer in those areas, as compared to the altitude of the potentiometric surface of the bedrock aquifer. In those areas, the potential for future land surface subsidence, in our opinion, is best described as low to moderate.

The development of the discovered corroded bedrock and overlying highly disturbed soil conditions, found in many of the GPR anomalies, is thought to have occurred during periods when sea levels were much lower causing an increase in the hydraulic gradient to occur in those times. It is our opinion that current time migration of sediments into bedrock void zones will be much less in the foreseeable future.

Possible Effects of Property Development – It should be understood that alterations to the pattern of ground water recharge and flow that may occur as the consequence of the development of this property, may also alter the potential for the development of sinkhole subsidence. For instance, the addition of water to a storm water management basin above the discovered sinkhole conditions may trigger a subsidence event in the basin, because of the artificial elevation of the surficial aquifer water surface in that area. Similarly, pavements and buildings will tend to reduce the recharge to the surficial aquifer below those areas, which may tend to lower the phreatic surface of the surficial aquifer. While this may be beneficial in some instances, the reduction of the water surface level will also act to increase the effective stresses in the soil mass. Such stress increases may promote readjustment of the soil column above the disturbed soil zones and promote sagging of the overlying soil into the discovered weak zones. The consequences of minor localized subsidence on buildings that are constructed above such areas may include costly repair, unless the sinkhole conditions are mitigated prior to construction.

Buildings

Our exploration reveals that the buildings will lie upon predominantly very loose to loose deposits of low-fines cohesionless sand that in turn lie upon generally more competent (denser) deposits of sand and cohesive soil. Without improvement of the discovered loose sand deposits, shallow foundations that bear on these soils are expected to undergo significant deleterious settlement, generally in excess of one inch, at the heavily-loaded columns and walls. If shallow spread footings are installed on the unimproved, very loose to loose deposits of sand, located within the top 10 to 15 feet of the land surface, we judge that settlement of the building's columns and walls may occur which will exceed the normal tolerances for steel structures, due to the expected high initial compressibility of the very loose sand deposits, and to the susceptibility of these very loose deposits to undergo further compression in the future, if they are subjected to ground vibration or to saturation. Ground vibrations may be transmitted, for example, in the event that soils are compacted using a vibratory roller, at a nearby site. Saturation of the foundation subgrade soils may occur in instances where precipitation runoff from the roof is allowed to pond or is directed near the perimeter walls. It is possible to reduce the potential for settlement of this structure by reducing the compressibility of the very loose deposits of sand, located below the bulk of the zone of soil that is stressed by the shallow foundations.

After improvement (densification) of the discovered weak soil deposits to increase their bearing capacity and reduce their compressibility, it is our opinion that the building subgrade soils will be capable of supporting the anticipated loads on a conventionally designed shallow foundation system after a program of site preparation. The following report subsection describes earthwork procedures which were found to be successful in achieving the required soil improvement, at similar sites. A series of three viable soil improvement options are presented, to allow value engineering to be applied in bidding the earthwork. These options consist of:

1. Conventional undercutting of the loose sand deposits and replacement by controlled compacted fill;
2. Densification of the loose sand deposits using impacts of a heavy tamper, commonly called Dynamic Deep Compaction (DDC); and
3. Installation of stone columns to densify and reinforce the loose soils located within the stress influence zone below the foundations.

That section also includes target specifications for in-place soil densities. It has been our experience that the described first option earthwork procedures can be accomplished using conventional earthwork equipment and techniques.

Pavement Areas

Paved areas on this property are expected to lie on existing or fill soils that are composed of predominantly cohesionless deposits of low-fines sand. Prevailing ground water conditions on this property and our review of proposed finished grades indicate that it is unlikely that seasonal fluctuation of the ground water body will bring the water surface within 2 feet of the bottom of anticipated pavement layers. Furthermore, it is expected that the pavement subgrade will be composed of essentially well-drained soil deposits. Lastly, our experience with similar soil deposits indicates that a Limerock Bearing Ratio (LBR) of 15 may be conservatively assumed as a basis of the design of the pavement layer system. This assumption is part of our pavement design recommendations.

Storm Water Management Basins

In general, the borings in the proposed storm water management basin reveal that the basin is underlain by between about 2-1/2 and 12 feet of low-fines rapidly draining sand deposits, lying upon less permeable deposits of silty, fine grained sand or clayey sand. We expect that the permeability of the silty and clayey sand deposits will be at least one order of magnitude lower than that of the surficial cohesionless sand deposits.

Infrastructure Corridors

The exploratory borings encountered low-fines sand deposits, within the upper regions of nearly all of the project site, with the exception of areas in the lower altitudes such as SPT-4B and P-4, and ground water surface altitudes predominantly below 25 feet NGVD. It is expected that most pipelines and other minor below-grade utility lines will be installed in the cohesionless soil deposits. Dewatering of trench excavations may be needed where the invert of the trench lies below 30 feet NGVD. In general, it is expected that trench backfill and bedding materials will consist of the sand deposits that will be excavated during installation of the utility lines. Where cohesive soil deposits are cut, it may be necessary to form bedding surfaces and backfill the trenches with the sand deposits borrowed from locations elsewhere on the property.

SINKHOLE MITIGATION RECOMMENDATIONS

Mitigation of Potential Future Sinkhole Subsidence Events

Mitigation Method – Where present, the discovered potential sinkhole conditions can be mitigated by the injection of grout into the weak soil zones and underlying corroded bedrock zones. This recommended correction (mitigation) of the discovered sinkhole activity conditions is intended to reduce the potential for future subsidence events, by closing the connection between the sediments that lie above the rock surface with void zones within the rock mass, and reinforcing the condition of the sediments that have already been disturbed by past migration of sediments into the void zones. Recommended methods to correct the geologic sinkhole conditions are described below.

Purpose – We recommend that the identified potential sinkhole zones be mitigated by injecting a low to moderate slump (4 to 6 inch slump) cementicious grout, under pressure, into the discovered weak bedrock and/or soil zones. The purpose of the grout injection program is to restrict and/or close breaches in the surface of the limestone that connect the soils that overlie the rock surface to suspected cavernous bedrock zones, and to consolidate the discovered weak soil zones that lie upon the rock surface. While it is possible that some cavernous zones may be filled using this technique, any such filling of the cavernous zones is unintentional, and unnecessary.

Installation of Injection Pipes – Within the proposed building areas, plus a margin of 25 feet beyond the perimeter of the building areas, and within any other area on the property where it is necessary or desirable to prevent the development of future sinkhole subsidence events, we recommend that grout injection pipes be installed through the sand and clay deposits that overlie the bedrock surface, and then that each point penetrate a distance of between 2 and 5 feet into the surface of the first "competent" (as determined by a representative of HSA during the installation operations) rock zone below the soils. The purpose of penetrating the rock surface is to confirm the presence of a competent base on which the grout column will bear, and to determine whether rock void zones underlie the surface of a thin ceiling of the rock over void zones. We recommend that injection points be installed to a depth of not more than 60 feet. Exception to this restriction may be granted by the engineer, who reviews the grout injection operations, in the event that injection point installation records indicate that the surface of the bedrock has not been penetrated, within 60 feet of the land surface.

Recommended Injection Point Location Plan – Based on the information that was revealed in this study, and considering the limitations of the data gathered for this study in the discovery that sinkhole conditions may underlie areas that were not detected in the GPR survey, it is our opinion that subsurface grout injection operations should be performed through injection points that are installed in a 25-foot wide by 25-foot long grid pattern throughout the entire building areas plus the above stated margin. In doing so, the operations will both provide qualitative confirmation that sinkhole activity conditions do or do not underlie the proposed buildings, at frequently checked locations, and will mitigate potential sinkhole activity conditions, where these are found.

Grout Injection Procedures – Grout should be injected first within and along the surface of the limestone, to reduce the potential for the development of future subsoil raveling/migration. Initially, the injection pipe should be withdrawn to the depth where the surface of the competent rock was detected, and sufficient grout should be injected to fill the bore hole that was created in the competent rock. Subsequently, the injection point should be withdrawn to a position that is between 1 and 2 feet above the surface of the bedrock, where injection operations to seal the surface of the bedrock should be performed. At that depth, provided that the grout will flow under pressures less than the threshold injection pressure, sufficient grout should be injected to cover a tributary area of the bedrock surface whose volume is determined by calculating the circular area, whose diameter equals the spacing distance between injection points, multiplied by the thickness of the injected grout (1 to 2 feet). Following completion of the sealing injection volume, injection of grout into the weak zones above the bedrock should then be performed. This injection operation into the weak zones above the bedrock is intended to consolidate and reinforce that weak material, and thereby reduce the potential for mass subsidence (sagging) of the overlying undisturbed soil zones into the weak zones. The volume, per lineal foot, of grout that is injected into the weak soil zones above the rock surface should correspond to the volume necessary to achieve injection pressure threshold limits, but not more than the incremental volume equivalent to the column, whose diameter corresponds to the spacing distance between grout injection points.

Injection Operation Limitations – We recommend that the grout threshold injection pressure be set to be not more than 150 psi above the pressure needed to initiate injection through the pumping and piping assembly, under no resistance at the exit point of the pipe. This threshold pressure should be determined at least once daily (before any grout is injected on any day) and whenever our representative notes significant change in the consistency and slump of the grout, as the operation progresses, in order to allow adjustment of the injection operations. This test should be performed by filling the test assembly (including all hoses, pipes and couplings) and then pumping a trial batch of the grout until such time that the fluid pressure at the peak of the stroke of the pump is consistent for a minimum of 20 strokes of the pump. The testing assembly should include a length of 60 feet of the grout pipe that is being used. This testing should be witnessed and the results verified by a representative of HSA.

Based on the data revealed in this exploration, we recommend that grout injection cease at an altitude no shallower than 20 feet NGVD, at any location, to avoid disturbing the surficial sand deposits that will support the building's foundations and floor slabs. Exception to this restriction may be granted by the engineer, who reviews the grout injection operations, in the event that injection records indicate that very weak soils exist above 20 feet NGVD.

Grouting may be performed at any location and depth until the above described threshold and volume limits are attained and until heave of the land surface of not more than 1/4 inch is detected within 5 feet of the pipe that is being injected with grout. When heave is detected, the pipe should be extracted to the next interval and grouting may resume.

Secondary Injection Points – During HSA's review of the injection operations, it may become evident that zones exist that require additional treatment in order to provide a reasonable level of assurance that the problematic conditions have been mitigated. Where that is indicated, secondary grout injection locations may be selected. Injection of grout at those locations should be performed in the manner described herein for the primary injection points.

Contractor's Grout Injection Plan – It is important that HSA work with the contractor during the planning and execution of the grout injection operations, to provide assistance in the development of the grout injection program that is responsive to the recommendations made herein, to monitor the injection operations, and to record pay quantity items, including injection pipe depths and grout mass consumption. HSA should be engaged by the owner to review and comment on the contractor's injection plan. The contractor should resolve any comments made by HSA to HSA's satisfaction, prior to attaining approval from the owner or other responsible party to initiate grout injection operations.

Remediation Cost Estimates – It is very difficult, if not impossible, to accurately predict the length of injection pipe that will be installed and the quantity of grout that will be necessary to stabilize the discovered sinkhole conditions. Accordingly, the associated cost to perform the remediation operation is also difficult to estimate. Accordingly, it is common practice to proceed with the grout injection operations on a unit rate basis. Detailed pipe installation and injection records should be created, by HSA personnel in the field, to document the injection operations. Based on those records, modifications to the program, as necessary, may be made to address differing and unexpected conditions, below the home. The grout injection program should be observed and monitored by of a representative of our firm to assess the effectiveness of the program and to assign appropriate modification when necessary, as outlined by the grouting plan described herein.

Unintended Damage To Water Nearby Wells – Please note that it is possible that the proposed grout injection operations may include injection of cementicious grout into fractures and void zones in the limestone aquifer. It has been our experience that nearby water wells may be affected by such grouting operations. Some of the effects could include the appearance of cloudy/sandy conditions in water withdrawn from affected wells. In rare instances the well collection zone may become sealed by intrusion of the grout. This operation may affect wells both within and beyond the limits of the subject property, depending on the transmissivity of the rock mass. Several factors influence the severity of this condition. These factors include the extent and severity of fissures and cavernous conditions in the limestone mass in this area, the size and depth of nearby wells, the proximity of the grouting operation to the wells, and whether water is being withdrawn from the wells, during the grout injection operations.

FOUNDATION RECOMMENDATIONS

As discussed above, we recommend that the proposed buildings be supported upon shallow foundation systems, and that the ground floor of the building consist of a conventional slab-on-grade element. We present herein a recommended procedure that may be applied in performing the earthwork that will be necessary to prepare the subgrade soils to receive the foundations.

Site Preparation Recommendations - Foundations

The existing natural surficial soils should be prepared prior to placement of engineered fill and foundation construction on the soils, in accordance with the following site preparation recommendations. The recommended procedures should be covered in the project specifications, and completed prior to construction of the foundation system.

1. The building area, plus a margin of five feet beyond the perimeter of the foundation system, should be cleared and grubbed of any vegetation, stumps, tree root systems, and sod. Organic topsoil should be excavated and removed. Stripped vegetation, roots, debris, and organic soils should be disposed in accordance with the owner's instructions. Any hole larger than three feet in diameter resulting from the removal of any object should be ramped to allow compaction of the bottom and sides with mechanical equipment prior to filling.

2. Mobilize the grout injection contractor and equipment on to the site and perform grout injection operations as outlined in the report section above.

3. After the grout injection operations have been completed, the entire building area, plus a margin of at least 5 feet beyond the foundation limit as measured at the bottom of the cut, should be excavated to an altitude corresponding to a minimum of that determined by subtracting the width of the foundation plus 24 inches from the proposed bearing level of the footings. For example, the cut level of a 5-foot wide foundation bearing at an altitude of 38 feet NGVD would be 38 less 5 feet less 24 inches (31 feet NGVD). The lowest calculated elevation should be the established cut level for that building. The exposed soils within the construction area plus the margin should be thoroughly moisture conditioned with an ample supply of water. This surface should then be compacted, by applying a minimum of six (6) passes of a steel-wheeled, self-propelled vibratory roller, having a minimum drum centrifugal force of 25,000 pounds, to a depth of 12 inches below the exposed grade to attain or exceed an in-place dry density of 95 percent of the Modified Proctor (ASTM D 1557) Maximum Dry Density. Each pass of the vibratory roller should overlap a minimum of 12 inches of the adjacent pass track. The specified density level should be measured by a qualified soils

technician using procedures described by ASTM D 2937 or approved equal, prior to commencement of subsequent procedures. In the event that initial rolling results in unstable yielding or pumping conditions, the soils engineer shall be contacted to determine the cause of the problem and make recommendations for remediation. As a minimum, soft, yielding, excessively wet, or otherwise unsuitable material shall be cut out and replaced with compacted clean granular fill. In the event that applied water does not penetrate sufficiently deep into natural soils to act as a lubricant in the compaction process, it will be necessary to disk or otherwise break up the soils before and during application of water.

4. After steps 1, 2, and 3 are completed, fill necessary to raise the grade to finished floor subgrade, or any interim working grade, should then be placed in one-foot thick layers, moisture-conditioned, and compacted to attain or exceed an in-place dry density of 95 percent of the Modified Proctor (ASTM D 1557) Maximum Dry Density. All fill should consist of clean, natural, deposits of granular soil, or of clean, processed, granular soils, which are free of roots and other organic debris. Inorganic debris, such as crushed concrete particles, may be included, insofar as the largest sized fragment will pass through a US Standard sieve, having a 2-inch mesh opening, and the grain size distribution of the material is not gap-graded. Acceptable fill shall be classified SP, SP-SM, SM, SW, or SW-SM. The use of gravel as fill may be permitted, on a case-by-case basis.

5. Continuous wall footing trenches and individual footing pits should be excavated to footing line and bottom grade. Foundation soils should be saturated with water and compacted with suitable mechanical equipment to achieve the specified level of density to the required depth. Foundation bottom grade should be tested to confirm that a minimum density of 95 percent of the Modified Proctor Maximum Dry Density exists to a depth of 12 inches below footing bottom. If necessary, the bottom of the footing excavation shall be undercut, refilled, and re-compacted with mechanical equipment to achieve the necessary minimum field density to the required depth.

6. Foundation backfill on sides of formed footings, if any, and building slab subgrade fill should then be placed in one-foot thick layers, be moisture-conditioned, and then compacted to attain or exceed an in-place dry density of 95 percent of the Modified Proctor (ASTM D 1557) Maximum Dry Density. All fill should consist of clean, natural, deposits of granular soil, or of clean, processed, granular soils, which are free of roots and other organic debris. Inorganic debris, such as crushed concrete particles, may be included, insofar as the largest sized fragment will pass through a US Standard sieve, having a 2-inch mesh opening, and the grain size distribution of the material is not gap-graded. Acceptable fill shall be classified SP, SP-

SM, SM, SW, or SW-SM. The use of gravel as fill may be permitted, on a case-by-case basis.

7. HSA Engineers and Scientists, Tampa office, should be engaged by the owner prior to site preparation to provide field observation of site preparation steps, compaction operations on natural and fill soils, and conduct field in-place density testing to confirm that the specified requirements are met.

Alternatives to procedure steps 3 and 4, outlined above, consist of the performance of dynamic deep compaction, to densify the soils, or the installation of stone columns, to transfer and distribute loads and reinforce the weak soils. Each of these alternatives is briefly described below.

Dynamic Deep Compaction Recommendations

Description of Work – The work shall consist of densifying the foundation soils by dynamic deep compaction of the areas and to the extent shown on the project drawings. The work shall be performed by a Specialty Contractor who can meet the requirements as outlined below. The Specialty Contractor shall furnish all supervision, equipment (including cranes), labor, and materials necessary or incidental to the completion of the dynamic compaction for this project.

Dynamic compaction is a process whereby a large weight is raised above the ground and allowed to fall from a height (commonly ranging between 40 to 80 feet), producing a high energy impact on the foundation subgrade soils. The height of the drop and the mass of the weight that is used should be specified by the selected DDC contractor to satisfy the soil improvement criteria stated herein. Specifically, the selected DDC contractor should specify the impact mass and the height of the impact that is necessary to compact the soil mass located a distance of twice the width of isolated (column) footings and four times the width of strip (wall) footings, below the base of the footing area being densified. The lateral extent of the densified soil mass shall correspond to the width of the footing, plus a margin of 1/4 of the width of the footing, on all sides of the foundation. The soil within the specified zone shall be compacted to achieve an average penetration resistance value of 20 blows per foot, as determined by SPT soil borings or interpolation of Cone Penetration Testing (CPT) soundings. In addition, SPT values shall be a minimum of 10 blows per foot within this zone.

After completion of the DDC operations on the foundation areas, the exposed soils within the construction area plus the margin should be graded to a level condition, and thoroughly moisture conditioned with an ample supply of water. This surface should then be compacted by applying a minimum of six (6) passes of a steel-wheeled, self-propelled vibratory roller, having a minimum drum centrifugal force of 25,000 pounds, to a depth of 24 inches below exposed grade, to attain or exceed an in-place dry density of 95 percent of the Modified Proctor (ASTM D 1557) Maximum Dry Density. Each pass of the vibratory roller should overlap a minimum of 12 inches of the adjacent pass track. The specified density level should be measured by a qualified soils technician using procedures described by ASTM D 2937 or approved equal, prior to commencement of subsequent procedures. In the event that initial rolling results in unstable, yielding or pumping conditions, the geotechnical engineer shall be contacted to determine the cause of the problem and make recommendations for remediation. As a minimum, soft, yielding, excessively wet, or otherwise unsuitable material shall be cut out and replaced with compacted clean granular fill. In the event that applied water does not penetrate sufficiently deep into natural soils to act as a lubricant in the compaction process, it will be necessary to disk or otherwise break up the soils before and during application of water.

The Specialty Contractor shall coordinate his work with the Contractor. Excavation, fill placement, or dewatering may be required prior to performance of dynamic compaction at the site. The water table should be at least 6 feet below the grade of the work areas prior to performing the work of this Section.

<u>Specialty Contractor's Qualifications</u> – The Specialty Contractor shall be regularly engaged in Dynamic Compaction work and shall be pre-qualified to perform this project. Pre-qualification shall include a minimum of the following information and shall be submitted 20 days prior to bid opening. The Specialty Contractor shall document that they have performed a minimum of the following work in the United States:

> Provided all supervision, labor, material and equipment to successfully densify by dynamic compaction, 25 separate projects involving over two million square feet utilizing energy inputs of between 20 and 40 tons dropped from heights of 40 to 80 feet to improve soil for bearing capacity using free fall, and single and double lines. Five of the documented, successfully completed projects shall be similar to this project in type of soil to be densified, depth to be densified, energy input required and the type of modified cranes to be used.

<u>Testing and Inspection Services</u>

HSA shall perform monitoring and testing during and after the site is dynamically compacted.

Densification will be verified using CPT soundings with readings taken at 20 cm. intervals from the ground surface, or Standard Penetration Test borings (ASTM D-1586) performed with continuous sampling intervals, within the improvement zone.

Protection of Persons and Adjacent Properties

A detailed safety program will be required to verify that jobsite personnel, off-site personnel and adjacent properties are protected. The safety program will require the Specialty Contractor to perform, using a qualified specialist, a detailed monitoring and documentation program before, during and after the dynamic deep compaction, of all structures within 100 feet of the work. The safety program will address the proposed barriers, fences, etc., to be utilized in protecting off-site personnel. The safety program shall include a complete discussion of the special programs utilized to assure the crane safety. This will include, but not be limited to, the before modification to all parts of the crane effected by dynamic deep compaction, i.e. the crane boom, cables, drums, brakes, clutch, outriggers, etc., and the daily, weekly, and monthly maintenance program.

Products

Provide equipment, materials and personnel required to achieve the results shown on the drawings. Cranes shall be rigged so that at least 95 percent of potential energy is realized at the point of impact.

Execution

The Specialty Contractor shall submit a detailed work plan showing impact layout and schedule.

Prior to production tamping, a test section will be treated to verify or allow modification to the energy levels. After satisfactory completion of the test section, production tamping may begin.

The Specialty Contractor shall keep adequate records of the operations, i.e., location number, number of drops, rate of penetration of weight, pass number, etc., and submit these daily to the geotechnical engineer and General Contractor. A detailed safety program shall be submitted to the General Contractor prior to the commencement of work outlining how the Specialty Contractor intends to protect his personnel, other personnel on the site, the safety of any adjacent structures, and the maintenance program required to assure the safe operation of the crane.

Stone Column Foundation Subgrade Improvement

The installation of stone columns is a subset of a deep soil compaction technique that uses a vibratory probe to densify deep weak soils and allow the improved soil to support the overlying structures on conventional shallow spread foundation elements. Stone columns are constructed through the use of vibrating probe that is typically jetted through the weak soils, to allow advancement of the probe to the required depth. Dry methods of construction have also been employed recently, to avoid inundation of the property with the jetting operation. As the probe is advanced and retrieved, a durable aggregate is placed in the probe hole and subsequently compacted. This sequence of advancement, retrieval and compaction of the aggregate is repeated in lifts up to the ground surface and is used to enhance the relative density and bearing capacity of the weak soils, located within the zone of influence of the overlying spread footings.

On this property, we recommend that the selected specialty contractor design his stone column improvement to satisfy the soil improvement criteria stated herein. Specifically, the selected specialty contractor should specify the number, arrangement, and length of stone columns needed to improve/reinforce soil located a distance of twice the width of isolated (column) footings and four times the width of strip (wall) footings, below the base of the footing area being that is being improved/reinforced. The lateral extent of the improved/reinforced soil mass shall correspond to the width of the footing, plus a margin of 1/4 of the width of the footing, on all sides of the foundation. The improved/reinforced zone shall be capable of supporting a net contact bearing pressure of 6,000 pounds per square foot. The soil between the stone columns is generally improved by this operation to differing degrees, dependant of the composition of the soil. Granular soils are generally densified to a significant degree, while cohesive soils are generally not initially densified, but may consolidate over time, due to the enhanced drainage and compaction provided by the installation of the stone columns. Pre-installation and post-installation Cone Penetration Testing (CPT) soundings should be performed to assess the extent of any inter-column soil improvement, and provide a basis on which to evaluate the success of the installation. In addition, monitoring of the installations should be performed by a representative of our firm, which should include recording of vibration probe power consumption, time to penetrate soil and then consolidate the stone backfill, probe penetration depth, aggregate volume consumed, etc., to verify that the contractor has installed the columns as stated in his proposal.

HSA should be engaged by the owner or his authorized representative to review the specialty contractor's plans and specifications. Authorization to mobilize and install the stone columns should be granted only after the specialty contractor has resolve all comments of the plans and specifications made by HSA, to the complete satisfaction of HSA and the owner.

After completion of the stone column installation operations on the foundation areas, the exposed soils within the construction area plus the margin should be graded to a level condition, and thoroughly moisture conditioned with an ample supply of water. This surface should then be compacted, by applying a minimum of six (6) passes of a steel-wheeled, self-propelled vibratory roller, having a minimum drum centrifugal force of 25,000 pounds, to a depth of 24 inches below exposed grade, to attain or exceed an in-place dry density of 95 percent of the Modified Proctor (ASTM D 1557)

Maximum Dry Density. Each pass of the vibratory roller should overlap a minimum of 12 inches of the adjacent pass track. The specified density level should be measured by a qualified soils technician using procedures described by ASTM D 2937 or approved equal, prior to commencement of subsequent procedures. In the event that initial rolling results in unstable, yielding or pumping conditions, the soils engineer shall be contacted to determine the cause of the problem and make recommendations for remediation. As a minimum, soft, yielding, excessively wet, or otherwise unsuitable material shall be cut out and replaced with compacted clean granular fill. In the event that applied water does not penetrate sufficiently deep into natural soils to act as a lubricant in the compaction process, it will be necessary to disk or otherwise break up the soils before and during application of water.

Foundation Design Recommendations

Following preparation of the subgrade soils, as described previously in this report, the shallow foundations may be proportioned for a maximum net allowable soil bearing pressure of 6,000 pounds per square foot, to support the design dead load plus sustained live load. The above stated allowable soil bearing pressure may be increased by one-third (1/3) when considering cases involving short duration transient load situations, such as that case when wind loads are considered on the structure.

A minimum soil cover of 18 inches, as measured from the bottom of the foundation system to lowest adjacent finished grade (or interior floor surface for interior area foundations) should be provided. Isolated spread footings should be proportioned to be at least 2.5 feet wide. Similarly, a minimum lateral dimension of 18 inches should be specified when proportioning the continuous strip (bearing wall) foundation elements. The foundations should be proportioned using equal dead load distribution methods, in accordance with Florida Building Code requirements.

Predicted Performance of Shallow Spread Footings

Selection of the recommended soil bearing pressure was based primarily on considerations of limiting the expected settlement to tolerable values. Considerations of overstress and general shear of the soils below the footings were found to not have a significant influence on the recommended soil bearing pressure on the compacted soil mass. Based on the expected magnitude of the foundation loads, we estimate that the continuous wall footings' maximum settlement will be on the order of 1/2 inch. Similarly, the estimated maximum settlement of isolated individual pad foundations is 1 inch. We also anticipate that nearly all of the settlement will be distortional in nature. That is, it will be the result of rearrangement of the soil particles which constitute the granular component of the soil and elastic compression of the soil particles which comprise the cohesive component of the soil in response to the applied load. This distortional settlement would occur almost immediately following placement of dead load on the foundations. Furthermore, it is our judgment that the settlement would occur incrementally as the dead weight loads are applied due to the predominantly granular nature of the foundation soils. Further distortional settlement of foundations due to the addition of sustained live loads during the useful life of the structure would also occur shortly following the application of those loads. Foundation settlement, which may be due to the application of transient live loads such as wind, is expected to be negligible. The expected weight of the combined structure and site fills are not expected to transmit significant additional stress to the underlying cohesive soils. Moreover, testing reveals that these soils are likely to be over-consolidated in nature, and that their compressibility is, therefore, judged to be low in the range of the stress to be applied by the proposed improvements. Consequently, we judge that the magnitude of long-term (consolidation-related) settlement of the foundation elements will be negligible.

Excavation and Trench Safety Considerations

The execution of earthwork operations, during the construction period, will require creation of excavations, whose depth may exceed the current threshold limits adopted and enforced by the Occupational Safety and Health Administration (OSHA). These regulations prescribe various geometric excavation cross sections that should be adopted for selected excavation depths to produce a safe operations environment for workers operating in the excavation. We recommend that the current OSHA regulations be incorporated into the project documents. In the event that the proposed excavations will exceed the OSHA published threshold geometric and site condition limitations, we recommend that the contract documents include requirements that the contractor provide plans and calculations, sealed by a geotechnical engineer, which detail the excavation geometry, the dewatering system to be used (if any), any excavation protection systems to be used (i.e., sheeting, shoring, etc.), and excavation execution procedures required to execute and maintain OSHA compliant excavations. Furthermore, we recommend that the contractor be required to submit the above described plans and calculations to the owner's representative at least 10 business days in advance of initiation of the excavations, as a record of his compliance with this requirement. Lastly, the OSHA compliant excavation plans should be incorporated into the contractor's site specific health and safety plan.

PAVEMENT DESIGN AND CONSTRUCTION RECOMMENDATIONS

Proposed Development

Based on information indicated in the project plan sheet, it is our understanding that the proposed development will include the installation of a circulation roadway network and surface parking for the employees, visitors, maintenance staff, and management staff of this complex. Traffic is expected to consist of passenger vehicles and occasional delivery and service vehicles. Traffic consistent with FDOT Type D roadways was assumed to apply to the expected traffic volume on the property pavement. You have indicated that the paved areas will be divided into light-duty and heavy-duty area. Light duty pavement will be expected to withstand 50,000 ESAL applications and heavy-duty pavement will be expected to withstand 125,000 ESAL applications. You have requested that both 10 year and 20 year service life options be evaluated, so that you can make an economic decision on the pavement to be installed.

Soil Evaluation

The soil and groundwater conditions in the pavement area are described previously in this report. Also, as discussed it is expected that seasonal groundwater levels are not likely to rise to within 12 inches of the existing land surface. The compacted surficial soils and anticipated nature of any embankment fill soils will provide an adequate subgrade for the proposed pavement. The expected level of the seasonal high ground water is not expected to affect the durability or performance of a conventional uncemented, moisture-sensitive base. Accordingly, we recommend that an uncemented base be used, in recognition of its generally favorable cost as compared to other materials.

Recommended Pavement Cross Sections

Ten-Year Service Life Pavement Option – Based on using FDOT Type D roadway and local requirements, assuming that the pavement subgrade will be raised to a level sufficient to allow the use of a limerock base, and assuming an in-place limerock bearing ratio of 15 for the pavement subgrade soils, we recommend that the roadway and surface parking pavement cross section consist of the following layers and thicknesses:

Southern Oaks @ Telecom Park Office Building Site				
HSA Project No. 502-5358-00				
Pavement Layer	**Specification**	**Thickness (Inches)**		**Minimum In-Place Density**
		Light Duty Required SN = 1.95	**Heavy Duty Required SN = 2.30**	
Asphalt	FDOT Type S-I	1-1/2	2	-
Base	FDOT Grade Limerock (LBR=100)	5	5	98% Modified Proctor (AASHTO T-180)
Alternate Base	FDOT Grade Crushed Concrete (LBR=150)	6	6	98% Modified Proctor (AASHTO T-180)
Alternate Base	FDOT Grade Shell (LBR=100)	6	6	98% Modified Proctor (AASHTO T-180)
Stabilized Subgrade	FDOT Type B (LBR=40)	6	8	98% Modified Proctor (AASHTO T-180)
Embankment or Subgrade	Cohesionless Sand Similar to Existing Surficial Soils (SP) or (SP-SM)	As Needed	As Needed	98% Modified Proctor (AASHTO T-180)

The civil engineer should confirm that the traffic and other conditions that are assumed in these recommendations are consistent with his anticipated conditions. Modifications to the recommended cross sections should be made by him, on the basis of FDOT design methods and the estimated traffic.

Twenty-Year Service Life Pavement Option – Based on using FDOT Type D roadway and local requirements, assuming that the pavement subgrade will be raised to a level sufficient to allow the use of a limerock base, and assuming an in-place limerock bearing ratio of 15 for the pavement subgrade soils, we recommend that the roadway and surface parking pavement cross section consist of the following layers and thicknesses:

Southern Oaks @ Telecom Park Office Building Site				
HSA Project No. 502-5358-00				
Pavement Layer	Specification	Thickness (Inches)		Minimum In-Place Density
		Light Duty Required SN = 2.16	Heavy Duty Required SN = 2.50	
Asphalt	FDOT Type S-I	1-1/2	2	-
Base	FDOT Grade Limerock (LBR=100)	5	6	98% Modified Proctor (AASHTO T-180)
Alternate Base	FDOT Grade Crushed Concrete (LBR=150)	6	7	98% Modified Proctor (AASHTO T-180)
Alternate Base	FDOT Grade Shell (LBR=100)	6	7	98% Modified Proctor (AASHTO T-180)
Stabilized Subgrade	FDOT Type B (LBR=40)	8	8	98% Modified Proctor (AASHTO T-180)
Embankment or Subgrade	Cohesionless Sand Similar to Existing Surficial Soils (SP) or (SP-SM)	As Needed	As Needed	98% Modified Proctor (AASHTO T-180)

The civil engineer should confirm that the traffic and other conditions that are assumed in these recommendations are consistent with his anticipated conditions. Modifications to the recommended cross sections should be made by him, on the basis of FDOT design methods and the estimated traffic.

Pavement Design

The pavement design and construction should be performed in accordance with the FDOT *Standard Specifications for Road and Bridge Construction* (latest edition). The vehicle pathways, and the parking stalls, may consist of a flexible pavement, which is comprised of a high quality asphaltic concrete surface layer, is supported on a compacted limerock base, and is underlain by a stabilized subgrade layer. Florida D.O.T. Type S asphaltic concrete should be used. Stabilization of the cohesionless sand subgrade, where indicated, may be created by blending a suitable additive with the expected cohesionless subgrade fill soils.

The pavement's base layer should be set sufficiently high above adjacent retention ditches, and be properly graded to prevent rainfall runoff and stored water from adversely affecting the durability or performance of the uncemented base material. As a minimum, the seasonal high water table and stored water levels should not encroach within 2 feet of the bottom of any uncemented base. Special attention should be given to details of any irrigated landscaped areas, to avoid saturation of the adjacent pavement. If necessary, drains should be installed to carry away any excess irrigation water.

Water Table Condition

The high groundwater levels, estimated in this study, indicate that the selection of the pavement base can be made without regard to the effect of excess moisture on the performance of the base material, and that the installation of underdrains will not be necessary.

Site Preparation

All areas to be paved should be stripped and grubbed of all vegetation, roots and organic matter. The stripped surface should then be compacted to a density equivalent to 98 percent of the Modified Maximum Dry Density (AASHTO T-180), to a minimum depth of 12 inches. Any fill placed below the pavement should consist of cohesionless sand, placed in 12-inch thick lifts, which is compacted to 98 percent of the Modified Maximum Dry Density (AASHTO T-180). At completion of grading and fill placement, the pavement subgrade should be proofrolled to confirm that a firm unyielding subgrade has been created. Any areas, which are found to yield under the proofrolling load, should be reworked, as necessary, to achieve an unyielding surface. Such reworking may involve additional compaction of existing soils and/or undercutting, and replacing unstable soil zones.

Wearing Surface

Specific requirements for Type S asphaltic concrete are outlined in Sections 333 and 331 in the FDOT *Standard Specifications for Road and Bridge Construction* (latest edition). Type S-I has a coarse texture and, if considered aesthetically undesirable, then Type S-III asphaltic concrete should be used. Limitations on the thickness of asphalt layers and their sequence of stacking (if necessary) in construction, as stated in the current FDOT flexible pavement design procedures, should be strictly observed in developing the design.

Pavement Base

In general, limerock is selected as the preferred material for the base layer below the asphalt, in this area of Florida, due to its cost and availability, as compared to other FDOT approved base material selections. For these reasons, limerock is recommended for selection as the pavement base layer. The limerock should be obtained from an FDOT approved source. Laboratory testing of the delivered limerock base shall be performed to confirm that the limerock demonstrates a Limerock Bearing Ratio (LBR) value of 100, or better, prior to its placement and compaction upon the underlying components of the pavement. The limerock base should be placed and compacted such that its finished line and grade complies with project plans, such that its thickness meets or exceeds that required by the plans, and such that its measured in-place density meets or exceeds 98 percent of the Modified Proctor (AASHTO T-180) maximum dry density value.

We recommend that the limerock base layer be placed upon a stabilized subgrade layer, where this type of base is used, to promote uniform densification of the base material. The subgrade below the limerock base should be stabilized by blending clayey soil, crushed rock or any other FDOT approved stabilizing additive to the existing cohesionless subgrade soil, or cohesionless embankment soils, as applicable.

Stabilized Subgrade

The stabilized subgrade layer should consist of in-situ or blended materials which comply with the material requirements given for FDOT Type B (LBR 40) stabilized subgrade layers, and should be placed and compacted upon the approved unyielding subgrade layer, created during initial earthwork operations. The stabilized subgrade should be compacted to 98 percent of the Modified Proctor (AASHTO T-180) Maximum Dry Density. This density level and material thickness should be confirmed by field in-place density and thickness tests, prior to proceeding to place and compact the overlying base layer. In addition, laboratory testing of representative samples of this material shall be performed, to confirm that the compacted stabilized subgrade layer demonstrates an LBR value of at least 40.

Concrete Pavement

Experience has indicated that high quality unreinforced concrete placed on compacted free-draining clean natural or fill subgrade can provide satisfactory, long-term performance as a pavement wearing surface. Good performance and low maintenance is highly dependent on satisfactory subgrade drainage and closely spaced joints. A joint pattern of 15 feet by 15 feet is highly recommended by the Florida

Concrete Products Association (FCPA). Pavement thickness and concrete design strength will depend on such variables as anticipated wheel loads and number of load applications, and subgrade LBR value of the native soils. For estimating purposes, a subgrade LBR value of 15 was assigned to the native sand deposits.

Ten-Year Service Life Pavement Option – Based on using FDOT and ACPA recommendations, the following cross section is recommended.

Southern Oaks @ Telecom Park Office Building Site				
HSA Project No. 502-5358-00				
Pavement Layer	Specification	Thickness (Inches)		Minimum In-Place Density
		Light Duty Required SN = 2.16	Heavy Duty Required SN = 2.50	
Plain Concrete	28-Day Compressive Strength = 4,000 psi	4.5	5.5	-
Embankment or Subgrade	Cohesionless Sand Similar to Existing Surficial Soils (SP) or (SP-SM)	As Needed	As Needed	98% Modified Proctor (AASHTO T-180)

Twenty-Year Service Life Pavement Option – Based on using FDOT and ACPA recommendations, the following cross section is recommended.

Southern Oaks @ Telecom Park Office Building Site				
HSA Project No. 502-5358-00				
Pavement Layer	Specification	Thickness (Inches)		Minimum In-Place Density
		Light Duty Required SN = 2.16	Heavy Duty Required SN = 2.50	
Plain Concrete	28-Day Compressive Strength = 4,000 psi	5	6	-
Embankment or Subgrade	Cohesionless Sand Similar to Existing Surficial Soils (SP) or (SP-SM)	As Needed	As Needed	98% Modified Proctor (AASHTO T-180)

STORM WATER MANAGEMENT BASIN EVALUATION AND OPINIONS

Based on the stratigraphy revealed in the soil borings, and considering the proposed storm water management basin, it is our opinion that the bulk of infiltration of storm water flow from the basin into the surficial aquifer soil deposits will occur as a consequence of lateral flow along the surface of the restrictive soil layers, mirroring the expected flow of the water that now passes through the surficial aquifer. We understand that you intend to install and permit a dry retention area. Therefore, we recommend that your construction specifications include provisions that may be necessary or desirable, to avoid restrictions to the passage of water through the sides of the basin. Furthermore, care should be taken to grade the bottom of the pond to prevent the creation of a cut below the surface of the low permeability soil layer, because doing so may create an effective pool (bathtub) in those soil deposits that would tend to hold water and prevent the pond from recovering in the required time period.

FILL SOIL SOURCES

To the extent possible, all fill that is used below the structures and pavement on this site should consist of clean, natural, deposits of granular soil, or of clean, processed, granular soils, which are free of roots and other organic debris. Inorganic debris, such as crushed concrete particles, may be included, insofar as the largest sized fragment will pass through a US Standard sieve, having a 2-inch mesh opening, and the grain size distribution of the material is not gap-graded. Acceptable fill shall be classified SP, SP-SM, SM, SW, or SW-SM. The use of gravel as fill may be permitted, on a case-by-case basis.

On-Site Borrow Fill Production and Soil Suitability

The near surface sands described herein can be categorized as clean fine sand to slightly silty and silty fine sand (Inorganic portions of **Stratum 1 and Stratum 2, Strata 3 through 8 and 10 through 12**) (SP, SP-SM or SM based on the Unified Soil Classification System). These soil deposits are judged to be suitable for use as pavement and structure fill materials. The clean fine sands (SP) will exhibit moderately rapid to rapid internal permeability, as compared to the underlying soils with increased fines content. These fine sands should require minimal conditioning prior to their inclusion in structure and pavement covered areas. Intrinsic moisture content will probably require adjustment in order to assist in efficient densification, depending upon specification requirements. Depending upon the time of year, it is expected that these soil types will be excavated above the water table. Any of these soil deposits that are excavated from below the prevailing ground water surface are expected to be in a relatively saturated state. However, this material should drain within stockpiles or can be dried back through disking and aeration. Select samples should be checked during borrow production for their grain size and plasticity characteristics.

The **Stratum 9** clayey sand deposits are expected to be in a generally moist condition when they are excavated, even below the prevailing ground water surface, as this material has a generally low internal permeability, and will neither readily absorb nor expel water. The cohesive soil deposits of **Strata 13, 14, and 16** appear to exhibit greater degrees of plasticity than the **Stratum 9** clayey sand deposits. These soils are generally classified as clayey sand (SC), which may occasionally grade to a sandy clay (CH) deposit. Careful moisture control and earthwork management of the this material may be required to produce an acceptable product for placement and compaction. Its use below structures and pavement is not recommended. The soils with high percentages of clay fines will likely require careful conditioning to control moisture content to levels suitable for placement and compaction.

CONSTRUCTION PHASE SERVICES

Construction Document Review

When the final design drawings and specifications are completed, HSA Engineers & Scientists should be engaged to review them, in order to determine whether changes in the original design concept may have affected the validity of our recommendations, and to confirm that these recommendations have been implemented in the design drawings and specifications.

Field Observations and Testing

Site earthwork procedures, including preparation of foundation bearing surfaces and compaction of any structural fill, should be observed by a geotechnical engineer or his representative from HSA Engineers & Scientists. Furthermore, testing to evaluate compliance of the earthwork operations with the project specifications should be performed by a geotechnical engineer or his representative from HSA Engineers & Scientists. Observations and testing of the project's earthwork components by our representative are necessary to verify that the subsurface conditions, which are revealed during the earthwork operations, are consistent with those found in this study, to confirm that the foundations are being constructed as indicated in the approved construction documents, and to confirm that the earthwork procedures are completed in accordance with the recommendations contained in this report.

The recovered soil samples are available for examination at our Tampa office. Unless otherwise instructed in writing, the soil samples will be discarded 60 days after the issuance of this report.

LIMITATIONS

Our professional services have been performed, our findings obtained, and our opinions prepared in accordance with generally accepted geotechnical engineering principles and practices. HSA specifically disclaims any responsibility for the conclusions, opinions or recommendations made by others based on these data.

The scope of this study was intended to evaluate generalized soil conditions within the planned development areas, whose extent is described herein. The analyses and recommendations submitted in this report are based, in part, on the data obtained from thirty (30) SPT borings and nineteen (19) AB borings, performed at the locations indicated on the attached Figure 3. This report does not reflect any variation of subsurface conditions between the borings. The nature and extent of such variations may be revealed during the course of construction and/or during the course of any supplemental studies. If variations then appear evident, it will be necessary to re-evaluate the recommendations made in this report on the basis of pertinent on-site observations, which are made by us during the construction period, wherein the characteristics of any variations are noted.

The scope of our services does not include any environmental assessment or investigation to assess the presence or absence of wetlands. Nor does the scope of our services include an environmental assessment or investigation to assess the presence or absence of hazardous or toxic materials in the soil mass, ground water bodies, or surface water bodies that lie either within or beyond the boundaries of the subject site. Any statements in this report regarding odors, staining of soils, or other unusual conditions observed are incidental to this study and are strictly for the information of our client.

CLOSURE

HSA Engineers & Scientists appreciates the opportunity to be of service to you on this project. We look forward to working with you during construction as your materials testing laboratory. Should you have any questions or require additional information, please do not hesitate to contact us at your earliest convenience.

Sincerely,
HSA Engineers & Scientists
Engineering Business No. 00007098

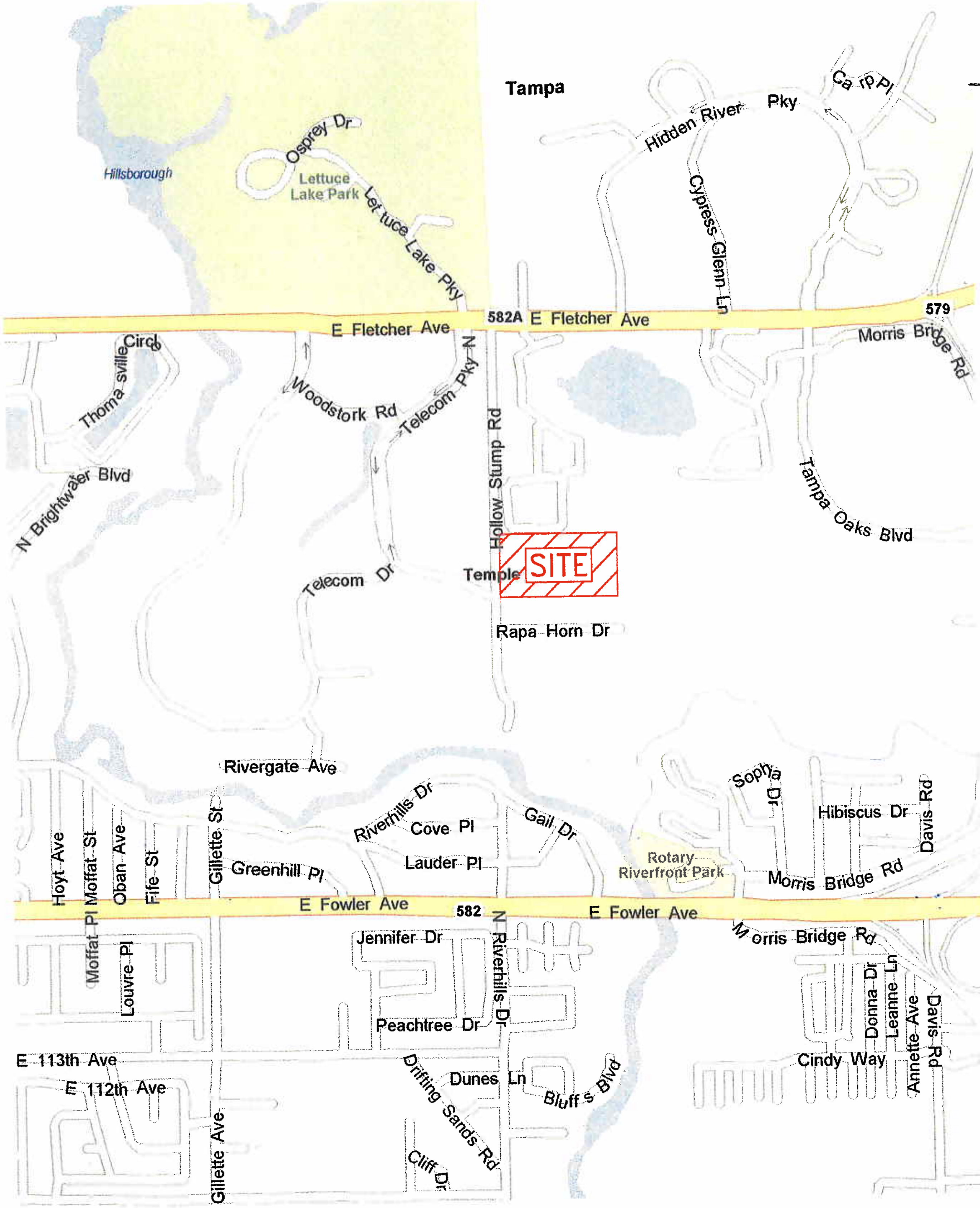
Joseph A. Eduardo
Joseph A. Eduardo, P.E. 11/19/07
Senior Geotechnical Engineer
Florida License No. 33318

John Marquardt
John R. Marquardt, P.E. 11/19/07
Senior Geotechnical Engineer
Florida License No. 24205

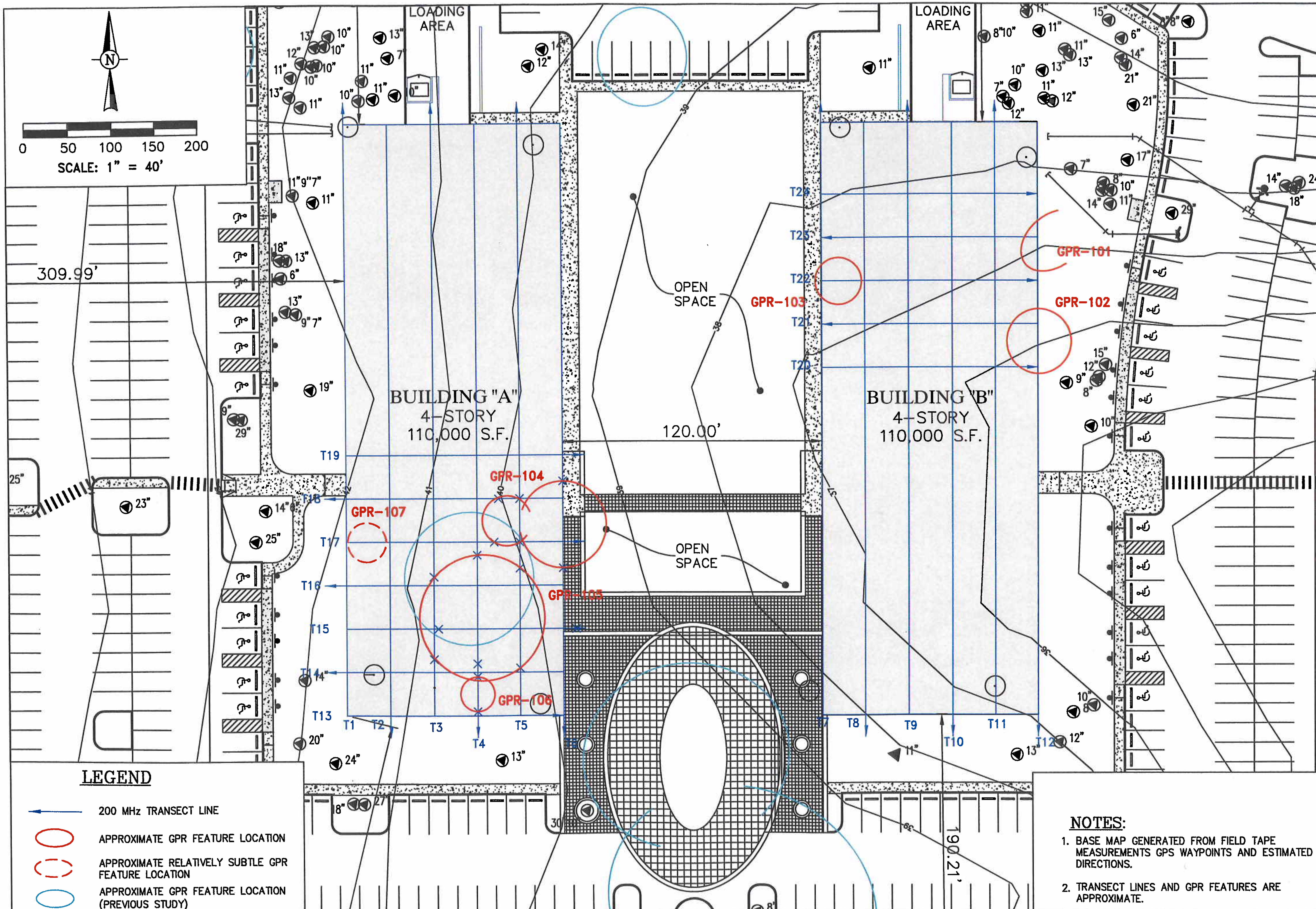
Attachments: Figures 1 through 12
Appendices A and B

Cc: King Engineering Associates, Inc. – Ms. Nicole L. Lynn, E.I.

SECTION 12, TOWNSHIP 28 SOUTH, RANGE 19 EAST
HILLSBOROUGH COUNTY, FLORIDA



JOB NO.: 50253580 CAD NO.: 535800-04 DATE: 5/09/07	**HSA** ENGINEERS & SCIENTISTS 4019 E. Fowler Avenue Tampa, Florida 33617 Tel: (813) 971-3882	PROPOSED OFFICE BUILDING SITE TAMPA TELECOM BUSINESS PARK BUILDING PHASE TELECOM PKWY EASY & STUMP HOLLOW RD TAMPA, FLORIDA	SHEET TITLE SITE LOCATION MAP FIGURE 1



LEGEND

- 200 MHz TRANSECT LINE
- APPROXIMATE GPR FEATURE LOCATION
- APPROXIMATE RELATIVELY SUBTLE GPR FEATURE LOCATION
- APPROXIMATE GPR FEATURE LOCATION (PREVIOUS STUDY)

NOTES:

1. BASE MAP GENERATED FROM FIELD TAPE MEASUREMENTS GPS WAYPOINTS AND ESTIMATED DIRECTIONS.
2. TRANSECT LINES AND GPR FEATURES ARE APPROXIMATE.

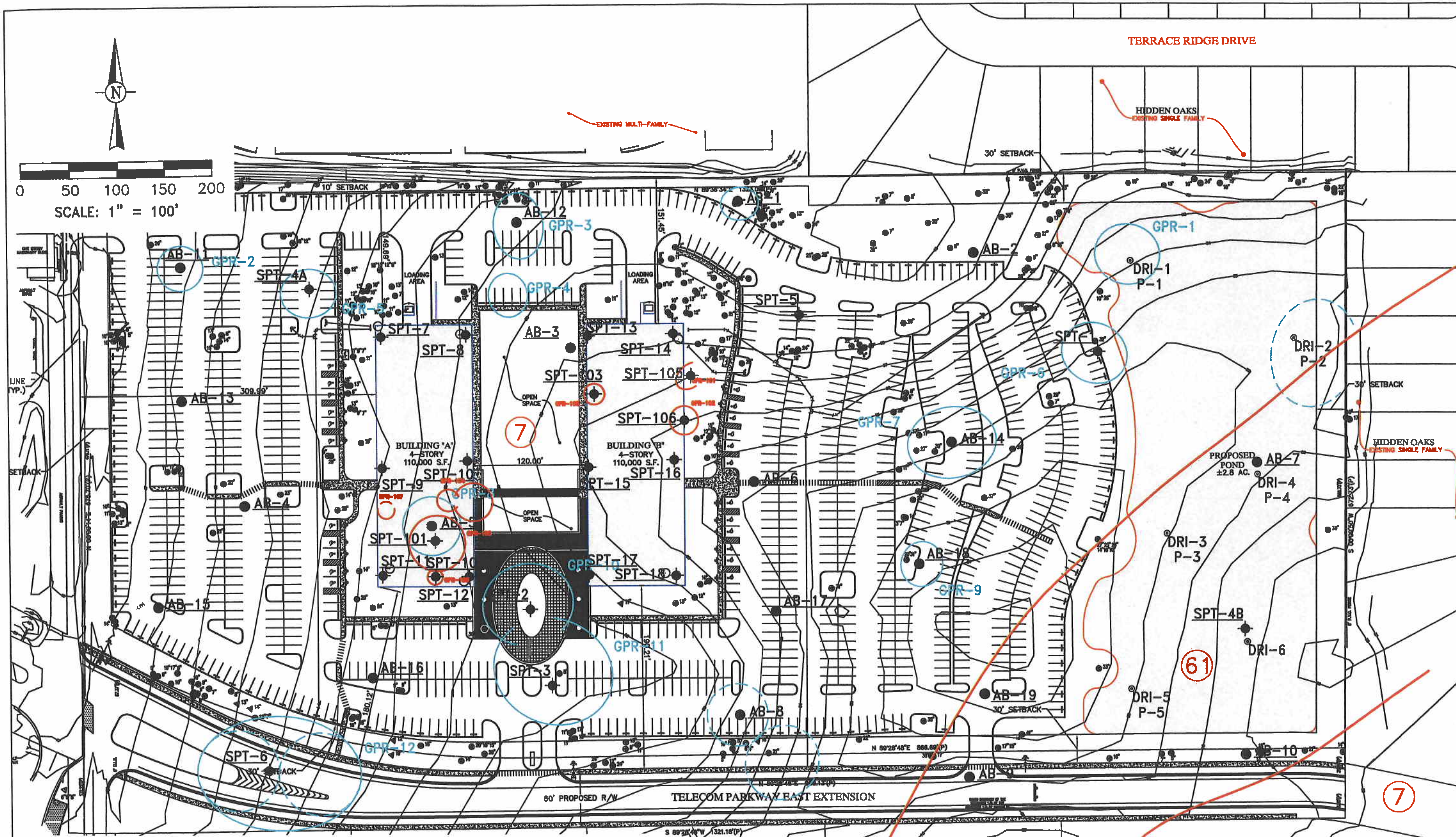
PROPOSED OFFICE BUILDING SITE
TAMPA TELECOM BUSINESS PARK
BUILDING PHASE
TELECOM PKWY EAST & STUMP HOLLOW RD
TAMPA, FLORIDA

HSA ENGINEERS & SCIENTISTS
4019 E. Fowler Avenue Tampa, Florida 33617 Tel: (813) 971-3882

DESIGNED	N/A
DRAWN	RBW
CHECKED	BAD
JOB NO.:	502535800
DATE:	5/09/07
CAD NO.:	535800-04

SHEET TITLE
TRANSECT LINE AND GPR FEATURE LOCATION MAP

FIGURE 2



LEGEND

SPT-1 STANDARD PENETRATION TEST BORING LOCATIONS

AB-1 HAND AUGER BORING LOCATIONS

APPROXIMATE GPR FEATURE LOCATION (CURRENT)

APPROXIMATE RELATIVELY SUBTLE GPR FEATURE LOCATION (CURRENT)

DRI-1 / P-1 COMBINED SPT BORING LOCATION AND DOUBLE RING INFILTRATION TEST LOCATION

APPROXIMATE USDA SOIL BOUNDARY LINE

APPROXIMATE GPR FEATURE LOCATION (PREVIOUS STURDY)

APPROXIMATE RELATIVELY SUBTLE GPR FEATURE LOCATION (PREVIOUS STURDY)

USDA LEGEND

7 CANDLER FINE SAND (0%-5%)

61 ZOLFO FINE SAND

NOTES:

1. BASE MAP PROVIDED BY KING ENGINEERING ASSOCIATES, INC.
2. BORING LOCATIONS ARE APPROXIMATE.

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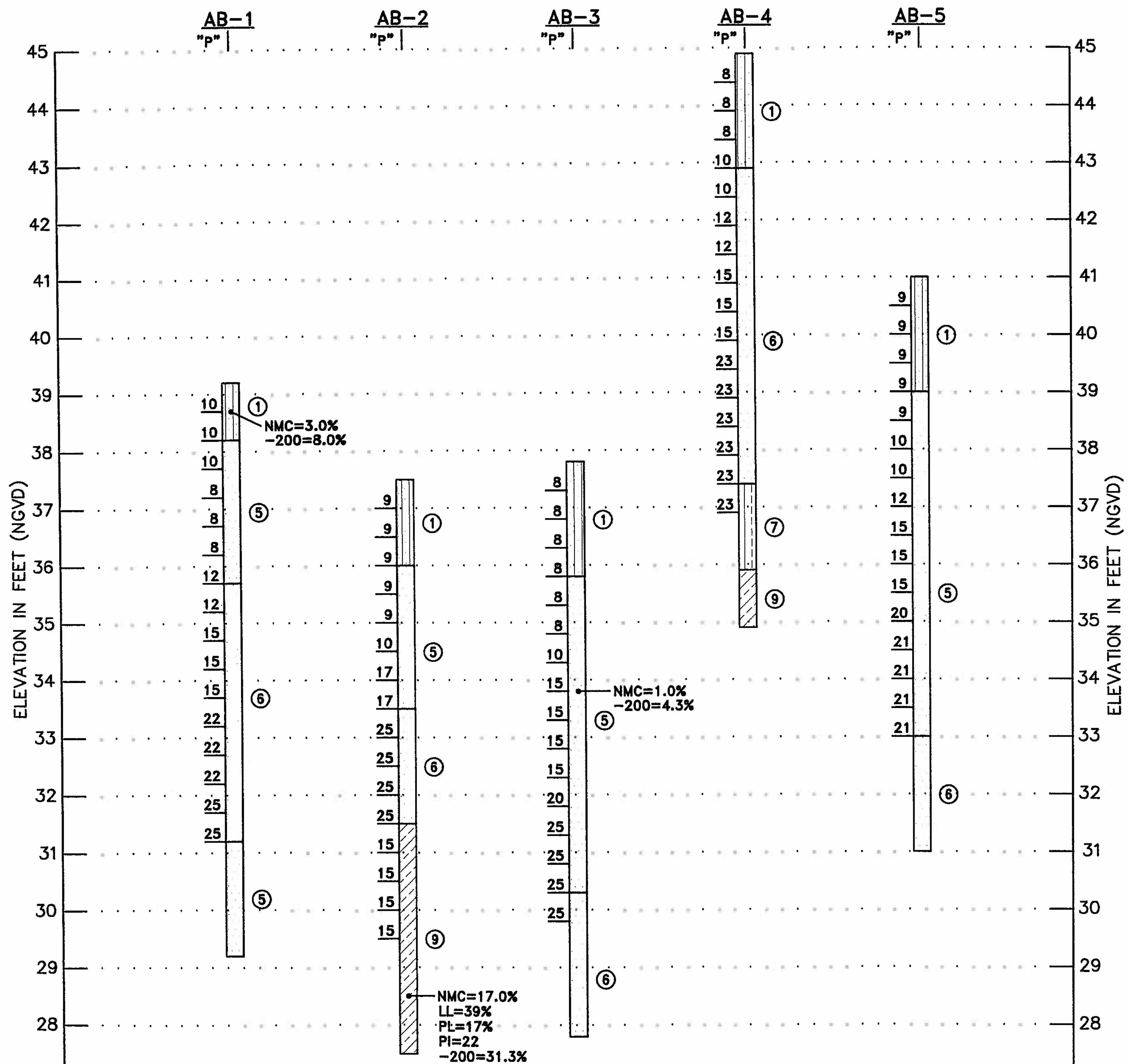
BORING LOCATION MAP

FIGURE 3



NOTE: GROUNDWATER NOT ENCOUNTERED
ELEVATION ESTIMATED FROM PROJECT TOPOGRAPHY

LEGEND:

- ① GRAYISH BROWN SAND WITH SILT (SP-SM)
- ② LIGHT GRAY SAND (SP)
- ③ STRONG REDDISH BROWN SAND WITH SILT (SP-SM)
- ④ BROWN SAND WITH SILT TO SILTY SAND (SP-SM/SM)
- ⑤ LIGHT BROWN TO YELLOWISH BROWN SAND (SP)
- ⑥ VERY LIGHT BROWN TO WHITE SAND (SP)
- ⑦ LIGHT GRAY SILTY SAND WITH TRACE OF CLAY MOTTLED WITH YELLOW IRON STAINS IN THE UPPER REGIONS (SM)
- ⑧ DARK YELLOW SAND WITH SILT (SP-SM)
- ⑨ LIGHT GRAY CLAYEY SAND MOTTLED WITH YELLOW, BROWN, OR DARK ORANGE IRON STAINS OCCASSIONALLY GRADING TO SANDY CLAY (SC)
- ⑩ DARK BROWN SAND WITH SILT (SP-SM)
- ⑪ VERY DARK BROWN SAND WITH TRACE OF ORGANIC SILT (SP-SM)
- ⑫ BROWN SILTY SAND (SM)
- ⑬ GRAY, YELLOW AND LIGHT BROWN MOTTLED CLAYEY SAND (SC)
- ⑭ LIGHT GRAY CALCAREOUS CLAYEY ELASTIC SILT WITH LIMESTONE FRAGMENTS (ML)
- ⑮ WHITE LIMESTONE (LS)
- ⑯ VERY DARK GRAYISH BROWN SANDY CLAY (CH)

(7.5YR 5/1)	MUNSELL SOIL COLOR CHART DESIGNATION
(SP)	UNIFIED SOIL CLASSIFICATION GROUP SYMBOL AS DETERMINED BY VISUAL REVIEW
"N" 5	BLOW COUNTS AT SHOWN DEPTH
"P" 5	HAND PENETROMETER READING AT SHOWN DEPTH (E= ERRATIC)
A	WITH LIMESTONE LENSES OR FRAGMENTS
B	WITH LENSES OF DARK GRAYISH BROWN CLAYEY SILT
100%	LOSS OF DRILLING FLUID CIRCULATION IN PERCENT
WH	WEIGHT OF ROD & HAMMER
WR	WEIGHT OF ROD
(N)	BLOW COUNTS AFTER ADVANCING SPOON MORE THAN 18 INCHES
NMC=37%	NATURAL MOISTURE CONTENT IN PERCENT
LL=37%	LIQUID LIMIT IN PERCENT
PL=37%	PLASTIC LIMIT IN PERCENT
PI=37	PLASTICITY INDEX
-200=37%	PERCENT OF FINES PASSING THE NO. -200 SIEVE

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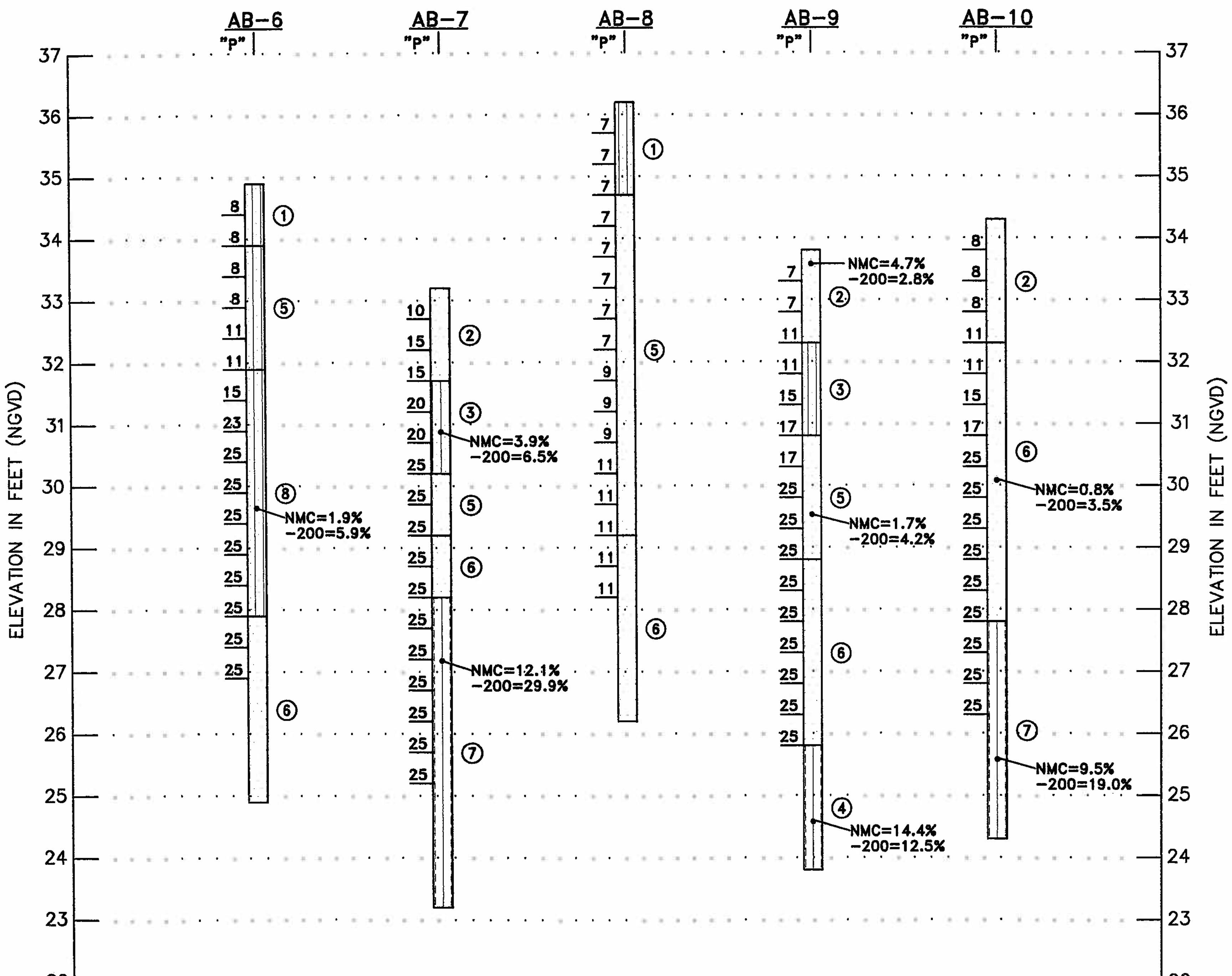
BORING PROFILES

FIGURE 4



NOTE: GROUNDWATER NOT ENCOUNTERED
ELEVATION ESTIMATED FROM PROJECT TOPOGRAPHY

LEGEND:

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②	LIGHT GRAY SAND (SP)
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④	BROWN SAND WITH SILT TO SILTY SAND (SP-SM/SM)
⑤	LIGHT BROWN TO YELLOWISH BROWN SAND (SP)
⑥	VERY LIGHT BROWN TO WHITE SAND (SP)
⑦	LIGHT GRAY SILTY SAND WITH TRACE OF CLAY MOTTLED WITH YELLOW IRON STAINS IN THE UPPER REGIONS (SM)
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⑫	BROWN SILTY SAND (SM)
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⑮	WHITE LIMESTONE (LS)
⑯	VERY DARK GRAYISH BROWN SANDY CLAY (CH)
(7.5YR 5/1)	MUNSELL SOIL COLOR CHART DESIGNATION
(SP)	UNIFIED SOIL CLASSIFICATION GROUP SYMBOL AS DETERMINED BY VISUAL REVIEW
"N" 5	BLOW COUNTS AT SHOWN DEPTH
"P" 5	HAND PENETROMETER READING AT SHOWN DEPTH (E= ERRATIC)
A	WITH LIMESTONE LENSES OR FRAGMENTS
B	WITH LENSES OF DARK GRAYISH BROWN CLAYEY SILT
100%	LOSS OF DRILLING FLUID CIRCULATION IN PERCENT
WH	WEIGHT OF ROD & HAMMER
WR	WEIGHT OF ROD
(N)	BLOW COUNTS AFTER ADVANCING SPOON MORE THAN 18 INCHES
NMC=37%	NATURAL MOISTURE CONTENT IN PERCENT
LL=37%	LIQUID LIMIT IN PERCENT
PL=37%	PLASTIC LIMIT IN PERCENT
PI=37	PLASTICITY INDEX
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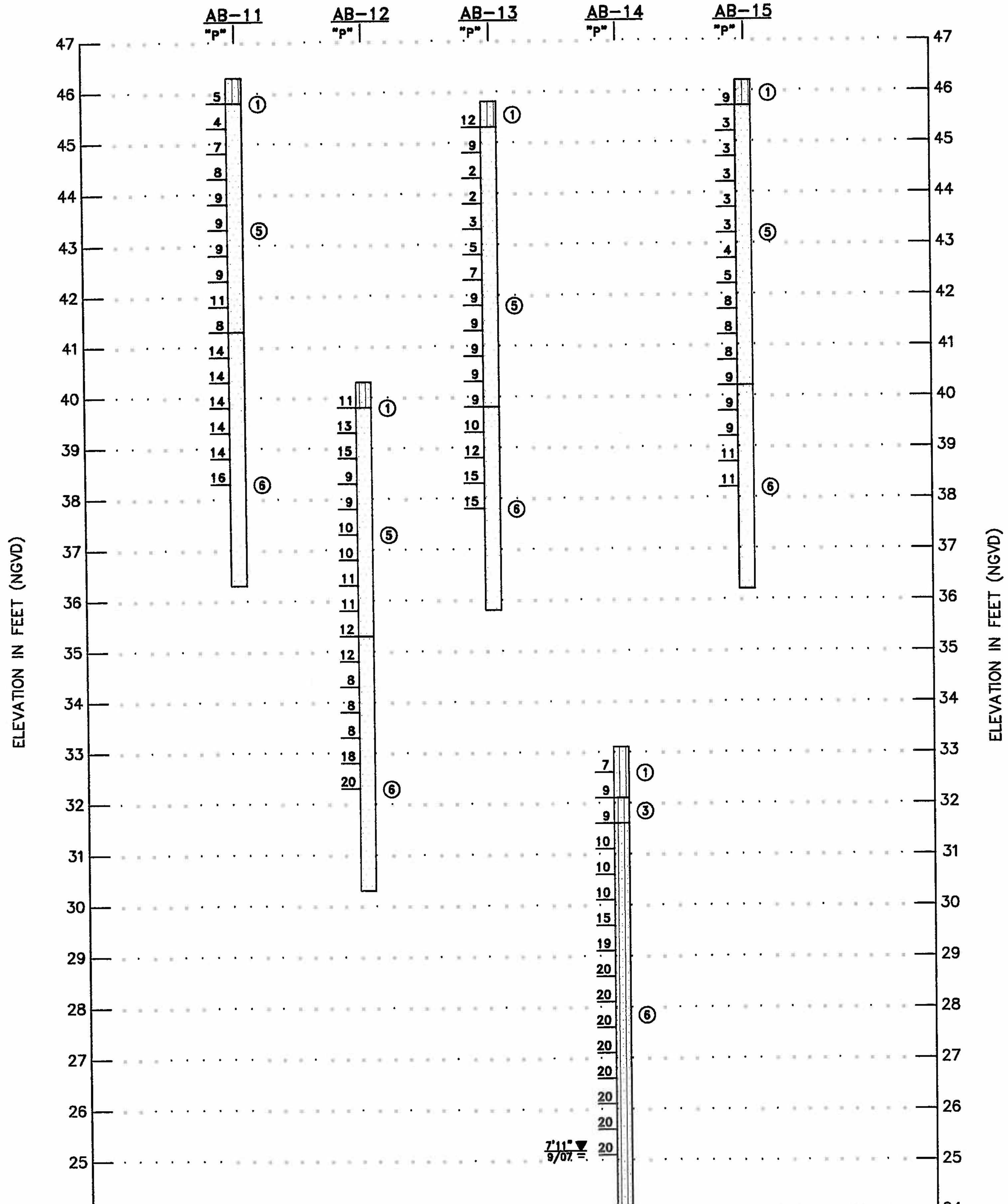
SHEET TITLE
BORING PROFILES

FIGURE 5



NOTE: GROUNDWATER NOT ENCOUNTERED EXCEPT AB-14
ELEVATION ESTIMATED FROM PROJECT TOPOGRAPHY

LEGEND:

① GRAYISH BROWN SAND WITH SILT (SP-SM)
② LIGHT GRAY SAND (SP)
③ STRONG REDDISH BROWN SAND WITH SILT (SP-SM)
④ BROWN SAND WITH SILT TO SILTY SAND (SP-SM/SM)
⑤ LIGHT BROWN TO YELLOWISH BROWN SAND (SP)
⑥ VERY LIGHT BROWN TO WHITE SAND (SP)
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⑫ BROWN SILTY SAND (SM)
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⑮ WHITE LIMESTONE (LS)
⑯ VERY DARK GRAYISH BROWN SANDY CLAY (CH)

(7.5YR 5/1)	MUNSELL SOIL COLOR CHART DESIGNATION
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A	WITH LIMESTONE LENSES OR FRAGMENTS
B	WITH LENSES OF DARK GRAYISH BROWN CLAYEY SILT
100%	LOSS OF DRILLING FLUID CIRCULATION IN PERCENT
WH	WEIGHT OF ROD & HAMMER
WR	WEIGHT OF ROD
(N)	BLOW COUNTS AFTER ADVANCING SPOON MORE THAN 18 INCHES
NMC=37%	NATURAL MOISTURE CONTENT IN PERCENT
LL=37%	LIQUID LIMIT IN PERCENT
PL=37%	PLASTIC LIMIT IN PERCENT
PI=37	PLASTICITY INDEX
-200=37%	PERCENT OF FINES PASSING THE NO. -200 SIEVE

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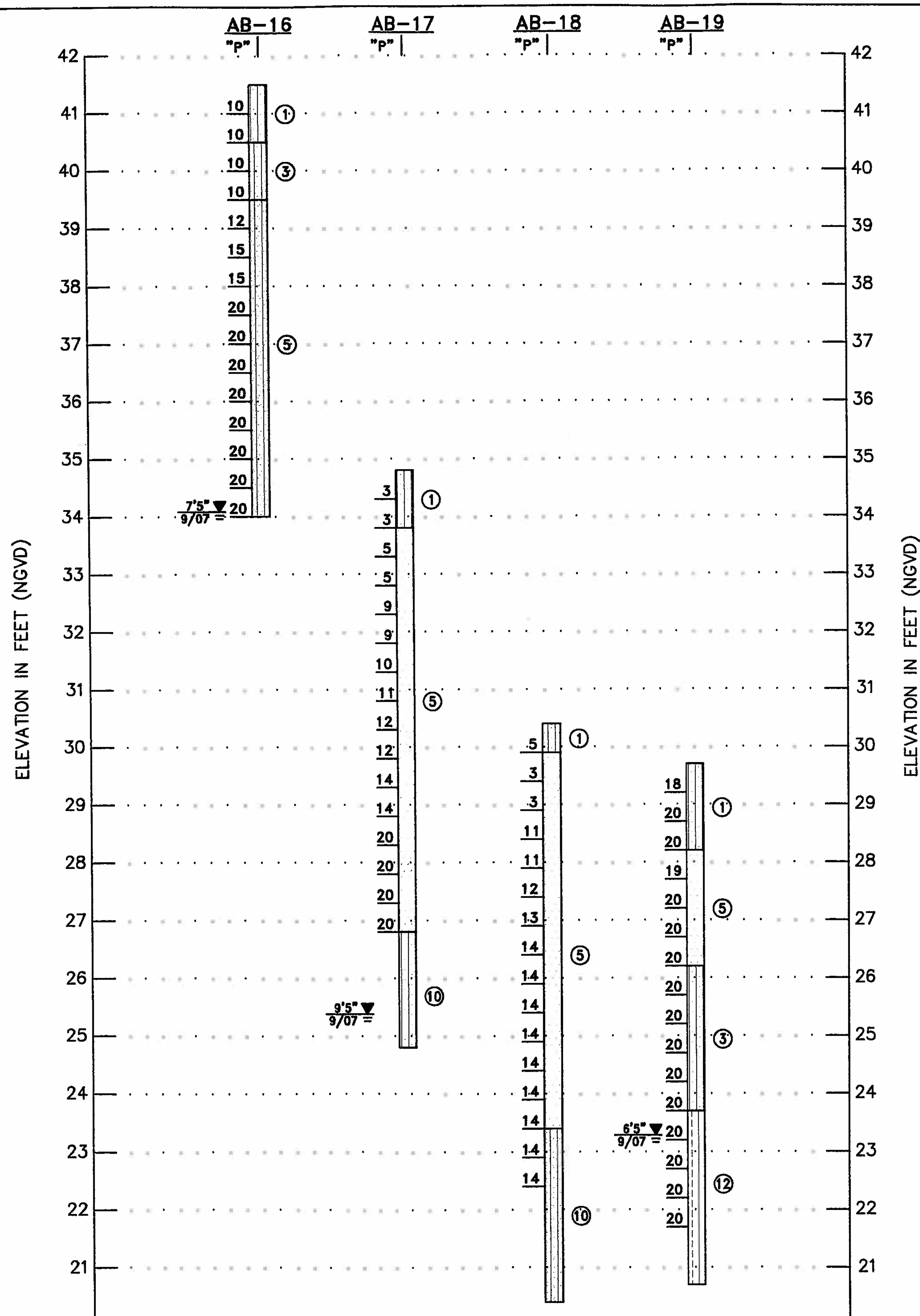
BORING PROFILES

FIGURE 6



NOTE: GROUNDWATER ENCOUNTERED EXCEPT AB-18
ELEVATION ESTIMATED FROM PROJECT TOPOGRAPHY

LEGEND:

1. GRAYISH BROWN SAND WITH SILT (SP-SM)
2. LIGHT GRAY SAND (SP)
3. STRONG REDDISH BROWN SAND WITH SILT (SP-SM)
4. BROWN SAND WITH SILT TO SILTY SAND (SP-SM/SM)
5. LIGHT BROWN TO YELLOWISH BROWN SAND (SP)
6. VERY LIGHT BROWN TO WHITE SAND (SP)
7. LIGHT GRAY SILTY SAND WITH TRACE OF CLAY MOTTLED WITH YELLOW IRON STAINS IN THE UPPER REGIONS (SM)
8. DARK YELLOW SAND WITH SILT (SP-SM)
9. LIGHT GRAY CLAYEY SAND MOTTLED WITH YELLOW, BROWN, OR DARK ORANGE IRON STAINS OCCASSIONALLY GRADING TO SANDY CLAY (SC)
10. DARK BROWN SAND WITH SILT (SP-SM)
11. VERY DARK BROWN SAND WITH TRACE OF ORGANIC SILT (SP-SM)
12. BROWN SILTY SAND (SM)
13. GRAY, YELLOW AND LIGHT BROWN MOTTLED CLAYEY SAND (SC)
14. LIGHT GRAY CALCAREOUS CLAYEY ELASTIC SILT WITH LIMESTONE FRAGMENTS (ML)
15. WHITE LIMESTONE (LS)
16. VERY DARK GRAYISH BROWN SANDY CLAY (CH)

(7.5YR 5/1)	MUNSELL SOIL COLOR CHART DESIGNATION
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WH	WEIGHT OF ROD & HAMMER
WR	WEIGHT OF ROD
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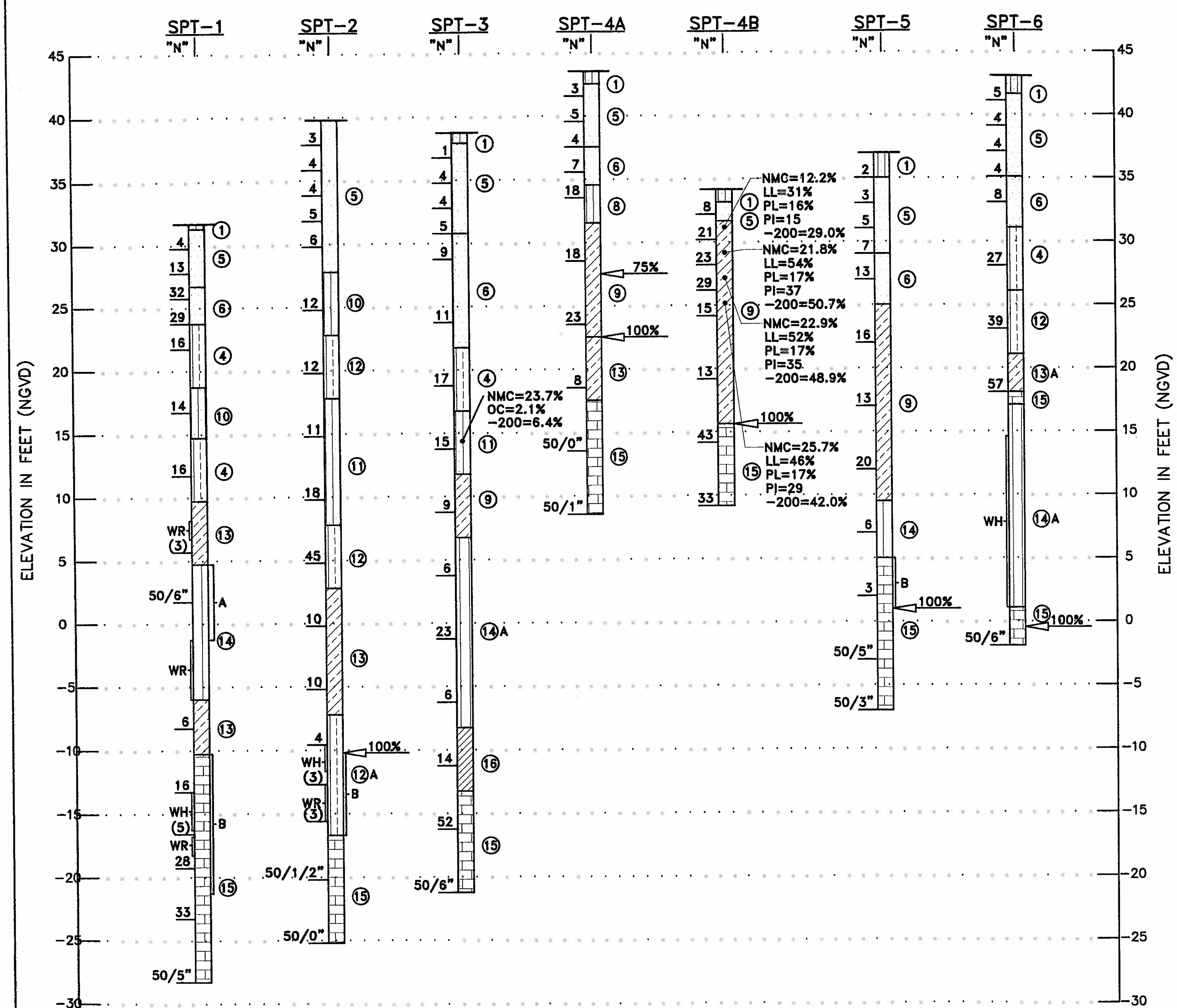
FIGURE 7



NOTE: GROUNDWATER NOT MEASURED

BORING ELEVATIONS ESTIMATED FROM TOPOGRAPHY SHOWN ON PROVIDED SITE PLAN

LEGEND:

1. GRAYISH BROWN SAND WITH SILT (SP-SM)
2. LIGHT GRAY SAND (SP)
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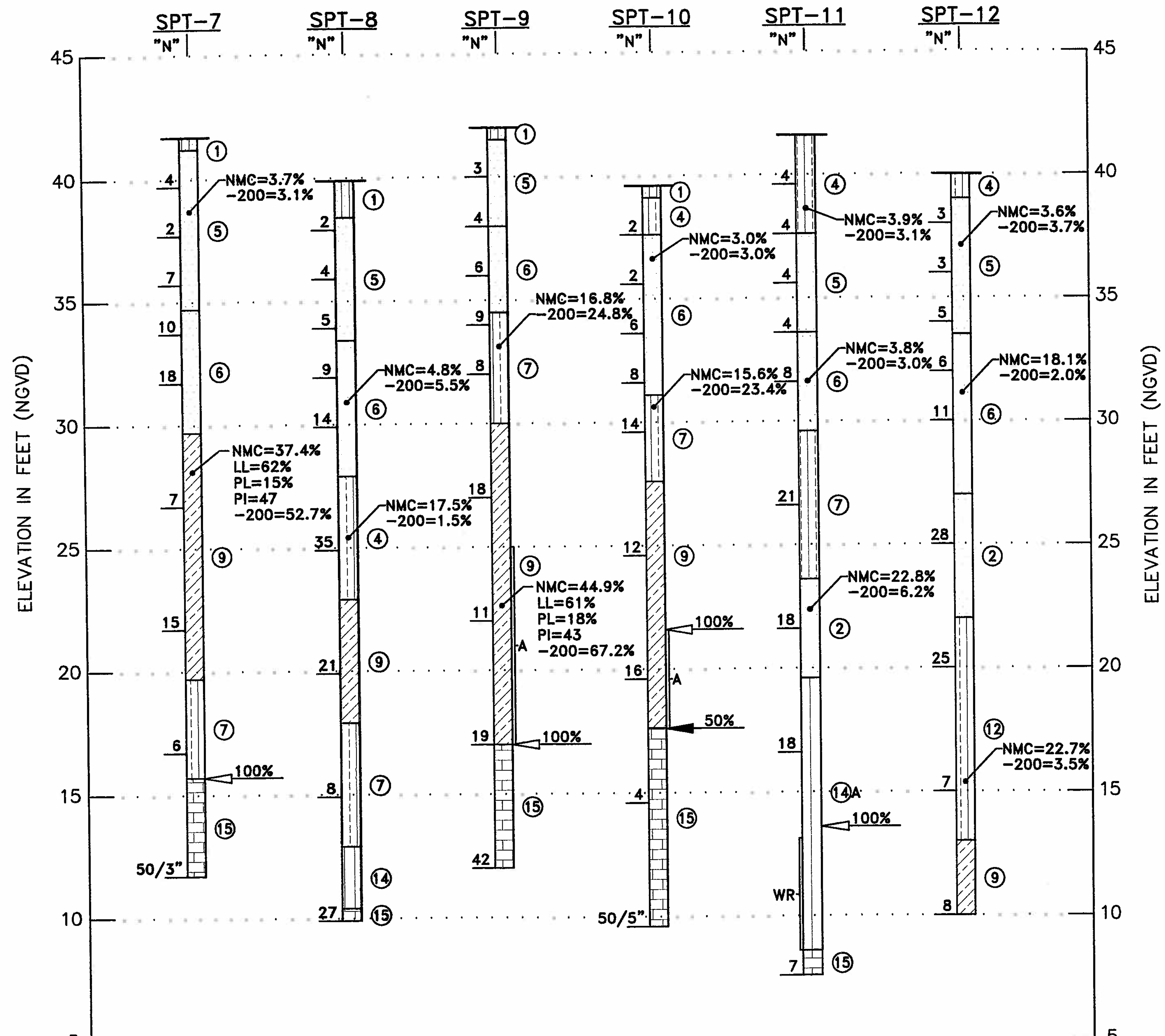
BORING PROFILES

FIGURE 8



NOTE: GROUNDWATER NOT MEASURED
ELEVATION ESTIMATED FROM PROJECT TOPOGRAPHY

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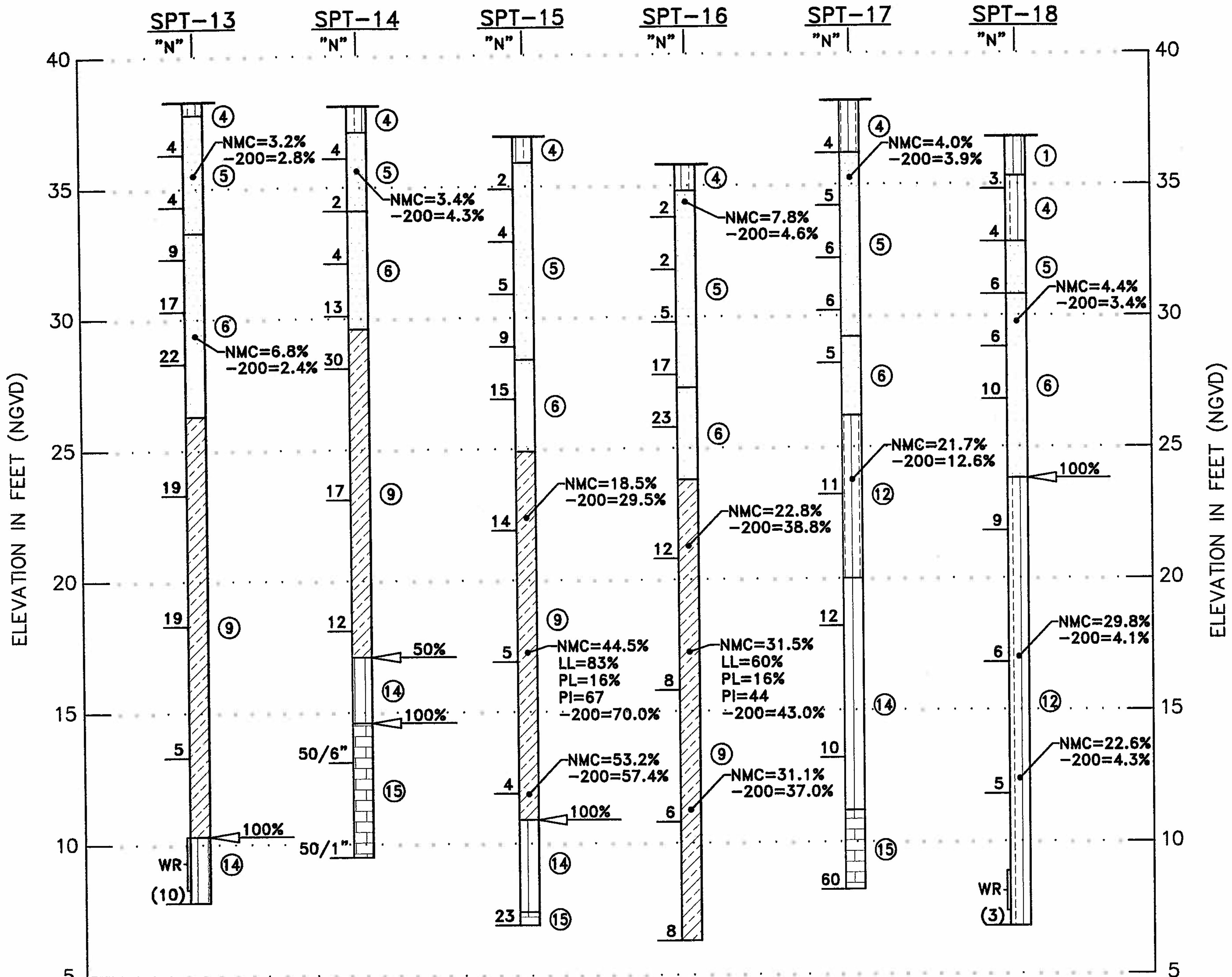
FIGURE 9



NOTE: GROUNDWATER NOT MEASURED
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LEGEND:

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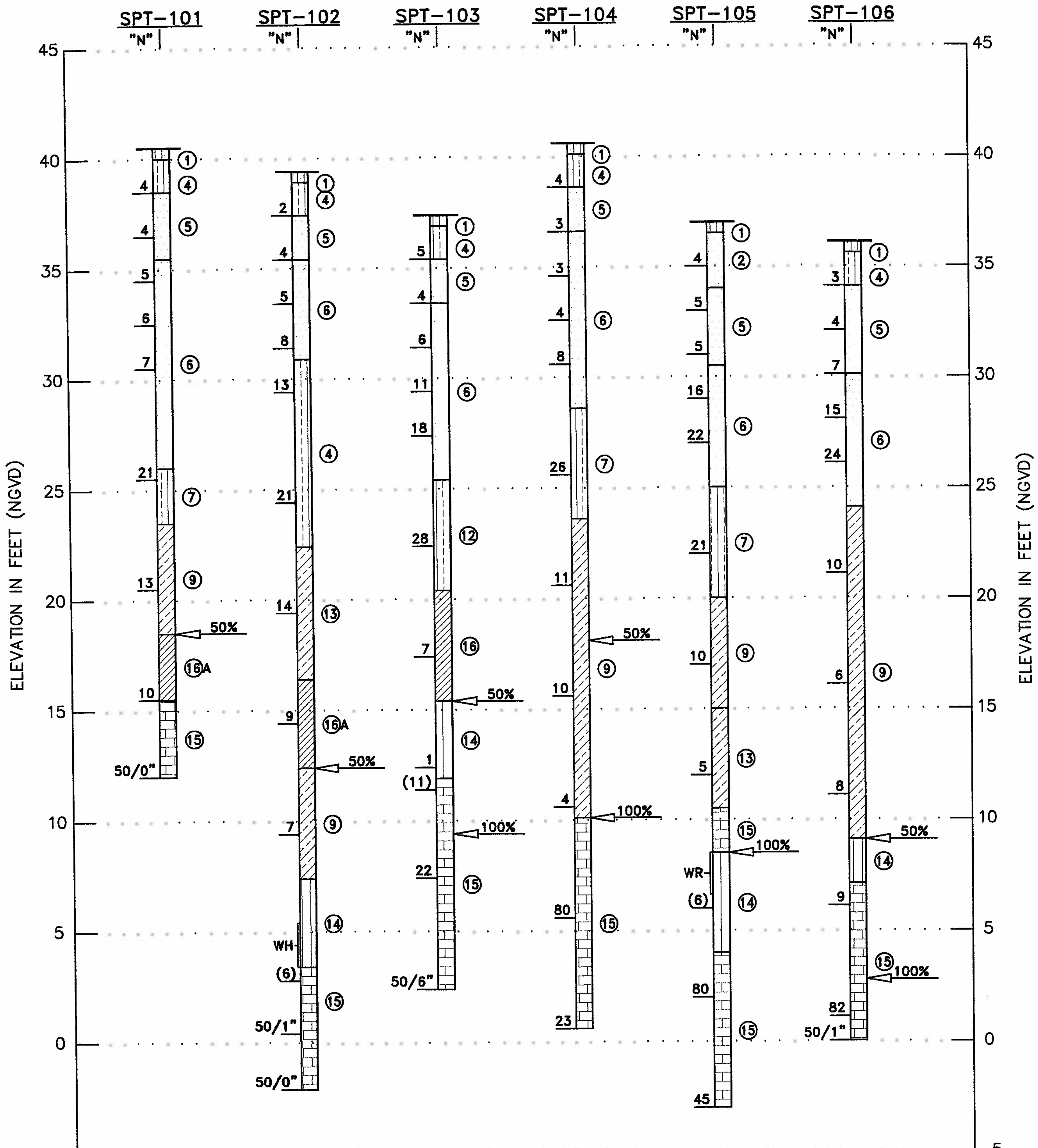
FIGURE 10



NOTE: GROUNDWATER NOT MEASURED
ELEVATION ESTIMATED FROM PROJECT TOPOGRAPHY

LEGEND:

1 GRAYISH BROWN SAND WITH SILT (SP-SM)
2 LIGHT GRAY SAND (SP)
3 STRONG REDDISH BROWN SAND WITH SILT (SP-SM)
4 BROWN SAND WITH SILT TO SILTY SAND (SP-SM/SM)
5 LIGHT BROWN TO YELLOWISH BROWN SAND (SP)
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11 VERY DARK BROWN SAND WITH TRACE OF ORGANIC SILT (SP-SM)
12 BROWN SILTY SAND (SM)
13 GRAY, YELLOW AND LIGHT BROWN MOTTLED CLAYEY SAND (SC)
14 LIGHT GRAY CALCAREOUS CLAYEY ELASTIC SILT WITH LIMESTONE FRAGMENTS (ML)
15 WHITE LIMESTONE (LS)
16 VERY DARK GRAYISH BROWN SANDY CLAY (CH)

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100%	LOSS OF DRILLING FLUID CIRCULATION IN PERCENT
WH	WEIGHT OF ROD & HAMMER
WR	WEIGHT OF ROD
(N)	BLOW COUNTS AFTER ADVANCING SPOON MORE THAN 18 INCHES
NMC=37%	NATURAL MOISTURE CONTENT IN PERCENT
LL=37%	LIQUID LIMIT IN PERCENT
PL=37%	PLASTIC LIMIT IN PERCENT
PI=37	PLASTICITY INDEX
-200=37%	PERCENT OF FINES PASSING THE NO. -200 SIEVE

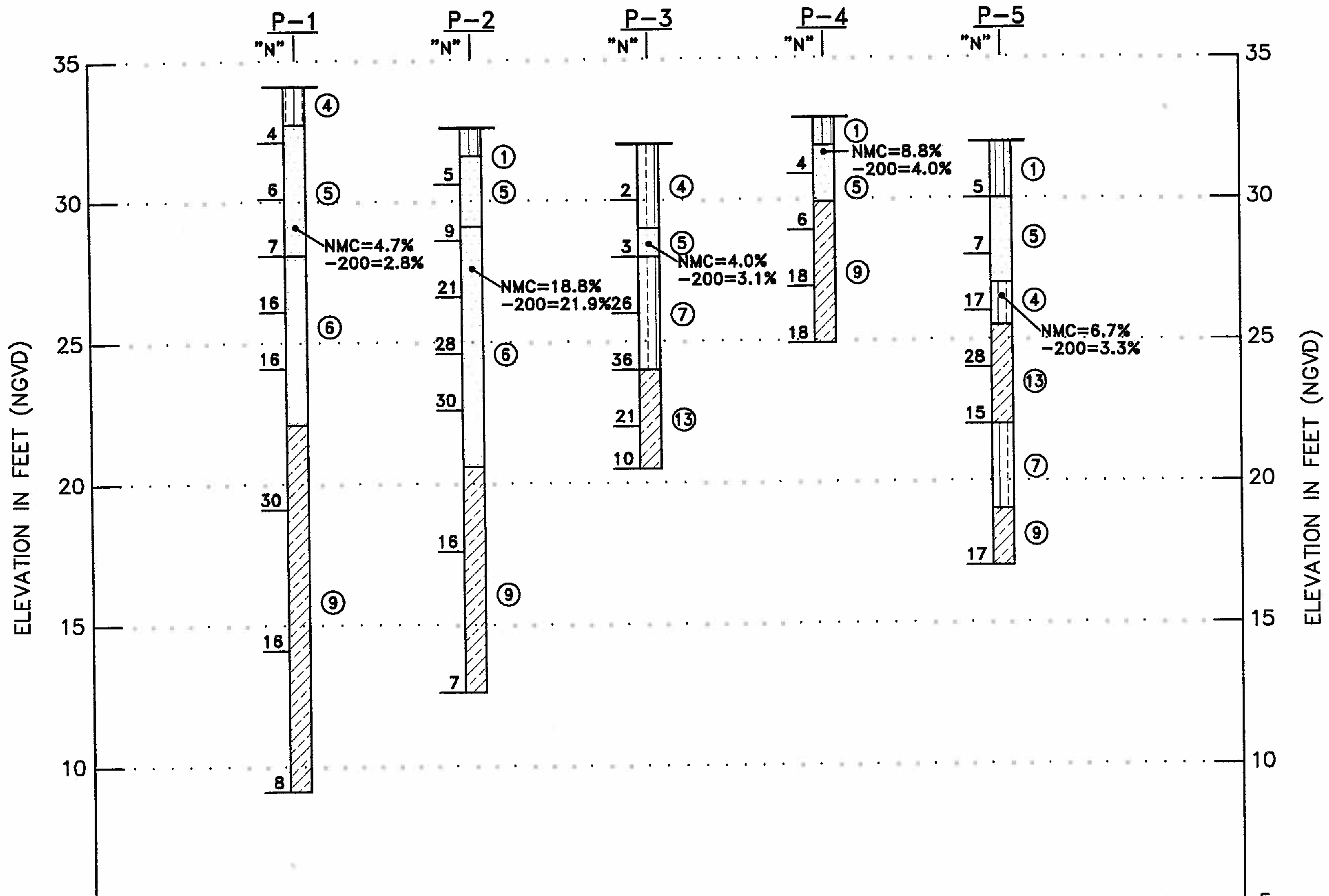
PROPOSED OFFICE BUILDING SITE
TAMPA TELECOM BUSINESS PARK
BUILDING PHASE
TELECOM PKWY EAST & STUMP HOLLOW RD
TAMPA, FLORIDA

HSA ENGINEERS & SCIENTISTS
4019 E. Fowler Avenue Tampa, Florida 33617 Tel: (813) 971-3882

DESIGNED	N/A	JOB NO.:	502535800
DRAWN	DLW	DATE:	5/09/07
CHECKED	RDM	CAD NO.:	535800-04

SHEET TITLE
BORING PROFILES

FIGURE 11



NOTE: GROUNDWATER NOT ENCOUNTERED
ELEVATION ESTIMATED FROM PROJECT TOPOGRAPHY

LEGEND:

1. GRAYISH BROWN SAND WITH SILT (SP-SM)
2. LIGHT GRAY SAND (SP)
3. STRONG REDDISH BROWN SAND WITH SILT (SP-SM)
4. BROWN SAND WITH SILT TO SILTY SAND (SP-SM/SM)
5. LIGHT BROWN TO YELLOWISH BROWN SAND (SP)
6. VERY LIGHT BROWN TO WHITE SAND (SP)
7. LIGHT GRAY SILTY SAND WITH TRACE OF CLAY MOTTLED WITH YELLOW IRON STAINS IN THE UPPER REGIONS (SM)
8. DARK YELLOW SAND WITH SILT (SP-SM)
9. LIGHT GRAY CLAYEY SAND MOTTLED WITH YELLOW, BROWN, OR DARK ORANGE IRON STAINS OCCASSIONALLY GRADING TO SANDY CLAY (SC)
10. DARK BROWN SAND WITH SILT (SP-SM)
11. VERY DARK BROWN SAND WITH TRACE OF ORGANIC SILT (SP-SM)
12. BROWN SILTY SAND (SM)
13. GRAY, YELLOW AND LIGHT BROWN MOTTLED CLAYEY SAND (SC)
14. LIGHT GRAY CALCAREOUS CLAYEY ELASTIC SILT WITH LIMESTONE FRAGMENTS (ML)
15. WHITE LIMESTONE (LS)
16. VERY DARK GRAYISH BROWN SANDY CLAY (CH)

(7.5YR 5/1)	MUNSELL SOIL COLOR CHART DESIGNATION
(SP)	UNIFIED SOIL CLASSIFICATION GROUP SYMBOL AS DETERMINED BY VISUAL REVIEW
"N" 5	BLOW COUNTS AT SHOWN DEPTH
"P" 5	HAND PENETROMETER READING AT SHOWN DEPTH (E= ERRATIC)
A	WITH LIMESTONE LENSES OR FRAGMENTS
B	WITH LENSES OF DARK GRAYISH BROWN CLAYEY SILT
100%	LOSS OF DRILLING FLUID CIRCULATION IN PERCENT
WH	WEIGHT OF ROD & HAMMER
WR	WEIGHT OF ROD
(N)	BLOW COUNTS AFTER ADVANCING SPOON MORE THAN 18 INCHES
NMC=37%	NATURAL MOISTURE CONTENT IN PERCENT
LL=37%	LIQUID LIMIT IN PERCENT
PL=37%	PLASTIC LIMIT IN PERCENT
PI=37	PLASTICITY INDEX
-200=37%	PERCENT OF FINES PASSING THE NO. -200 SIEVE

PROPOSED OFFICE BUILDING SITE
TAMPA TELECOM BUSINESS PARK
BUILDING PHASE
TELECOM PKWY EASY & STUMP HOLLOW RD
TAMPA, FLORIDA

HSA ENGINEERS & SCIENTISTS
4019 E. Fowler Avenue Tampa, Florida 33617 Tel: (813) 971-3882

DESIGNED	N/A	JOB NO.:	502535800
DRAWN	DLW	DATE:	5/09/07
CHECKED	RDM	CAD NO.:	535800-04

SHEET TITLE
BORING PROFILES

FIGURE 12

APPENDIX A

GROUND PENETRATING RADAR SURVEY

On September 12, 2007, HSA Engineers & Scientists (HSA) completed a supplemental geophysical investigation, consisting of a limited ground penetrating radar (GPR) survey at the proposed Tampa Telecom Office Complex site in Hillsborough County, Florida. More specifically, the subject property is a rectangular tract of land that occupies the northeast quadrant of the intersection of Telecom Parkway East, at Hollow Stump Road, at the east end of the Tampa Telecom business park and occupies approximately 18.4 acres. The north and east property boundaries consist of residential developments. The objective of the reconnaissance type geophysical survey was to use GPR to document the lateral and vertical variability of the shallow subsurface soils within the limits of the proposed building footprints with the objective of further identifying and delineating areas of anomalous reflection patterns consistent with potential buried karst or sinkhole topography.

Operational Theory

Ground Penetrating radar (GPR) is a non-intrusive surface geophysical method used for locating subsurface features and/or investigating the lateral extent and continuity of soil and by inference, stratigraphic contacts. GPR operates by transmitting pulses of ultra high frequency radio waves (electromagnetic energy) into the ground to detect interfaces between materials with differing dielectric constants, or relative variance in electrical permittivity of different materials, (*e.g.*, sands and clays)—this means that each time the materials through which the GPR signal travels changes, a reflection is created, which makes GPR a useful tool in detecting variations in the otherwise relatively horizontal signal reflecting layers. The electrical permittivity is commonly proportional to the electrical resistivity of the materials. Therefore, the radar profile depicts changes in the electrical character of the ground rather than density changes. All radar reflections from a given interface are depicted as three bands representing positive and negative voltages associated with polarity interactions of the electromagnetic wave at that interface.

Such reflection patterns, which are either sharply downwarped, are discontinuous, or have an irregular parabolic shape on the digital image profiles produced by the GPR, can be considered anomalous features of interest that may be related to sinkhole activity. These dips and depressions in the reflective soil layers may have formed in the geologic past, and do not necessarily mean that active or incipient sinkhole activity is present. Anomalous GPR readings can also be attributed to variations in the depth at which silty or clayey soils were encountered and variations in the reflective properties of the soils resulting from variations in the clay and silt content that occurred as the materials were deposited.

APPENDIX A

The GPR data is described as an "electrostratigraphic" profile. The profile is not a true cross-section because the vertical scale in all layers may not be equal and elevation changes in the ground surface are not taken into account. Profiles having these limitations are called "virtual" cross-sections, yet they are very helpful for documenting the lateral continuity, thickness and depth of soil, sediment and/or bedrock layers, or locating buried pipes and tanks.

The differences in the GPR signal do not, in themselves, necessarily prove or disprove the existence of a sinkhole indicator. However, the technique serves as an economical and useful means to help target areas for further direct exploration efforts (borings). It should also be noted that subsurface soil conditions or the occurrence of sinkhole activity below the depth of penetration of the GPR signal might prevent detection of a sinkhole indicator by the GPR. Several trial runs were made with the equipment across the site to select GPR signal frequencies and equipment settings to both maximize signal penetration without loss of adequate resolution.

Survey Design

Prior to commencing with the GPR survey, HSA established a general north-south oriented survey pattern within both of the proposed building footprints on the survey site. These test lines are a series of lines on the ground along which the survey is performed and two-dimensional graphical profiles were collected and are referred to as transect lines, or transects. The GPR survey was performed on a quasi-linear, sub-parallel pattern of transects spaced at intervals of approximately 50 feet. Additional east-west oriented lines were also performed in select areas. Use of GPS coordinates were relied upon in part for locating and orienting the GPR transects at the site. Location and layout of the test lines were governed by clearing of pathways conducted prior to our arrival on site.

Figure A-1 presents the approximate location of the GPR transect lines for the entire survey. The GPR survey was performed along a total of 24 transect lines, labeled T-1 through T-24.

Equipment & Application

Instruments- The GPR data was collected and stored with a Subsurface Interface Radar System 2000 (SIR-2000) manufactured by Geophysical Survey Systems, Inc. (GSSI), North Salem, New Hampshire. The SIR-2000 is a lightweight, portable, single channel, general-purpose, ground penetrating radar system that digitally records the GPR profiles on a hard drive. Initial testing was performed with a 200 megahertz (MHz) monostatic antenna system, which utilizes a single antenna housing to combine both the transmitter and receiver electronics. The antenna was towed across

APPENDIX A

the ground by hand at a slow walking pace. The survey was performed utilizing a survey wheel attached to the radar antenna, which allowed for normalization of the data across the entire site, collected at a rate of 4 scans per foot.

Data Acquisition Parameters

Time Window Settings- The time window or "range" setting was adjusted to record reflections returning after a maximum elapsed time, or time window of 238 nanoseconds (ns), allowing for signal penetration to maximum depths of about 50 to 60 feet, assuming mostly dry sandy soils. However, the actual depth of penetration of the GPR signal varies from place to place depending on soil properties such as grain size, mineralogy, and moisture content. Therefore, the actual depth of signal penetration may have been considerably less than the range settings would indicate due to the likely presence of water-saturated soils and/or more conductive soils (silts, organics and/or clays) that tend to slow and attenuate the radar signal, thus effectively limiting the depth of signal penetration.

Gain Settings- The signal received by the transmitting antenna is a very low amplitude wave and therefore must be amplified for interpretation. Gain settings are adjusted on the control unit to enhance these low amplitude signals. Furthermore, the amount of signal attenuation increases with depth, requiring more amplification. Therefore, the SIR-2000 applies a time-varying gain curve to the signal to enable the user to manually increase the amplitude. In order to manually increase the amplitudes along a GPR trace, control points or gain points are assigned at equal intervals along the trace to define the time-varying gain curve. The gain setting parameters are depicted on the header sheets of each of the GPR profiles.

Filter Settings- The filter settings were adjusted to display the weakest-detectable signal without introducing excessive amounts of "noise." Noise manifests itself as smudges or "snow" (high frequency noise) and horizontal bars or banding (low frequency noise) on the profiles. To minimize noise, the vertical low-pass and high pass filters were adjusted to exclude signal return above 400 MHz and below 60 MHz. To maximize the signal to noise power ratio, horizontal stacking was set at 5, meaning that 5 traces or scans of signal pulse were averaged to produce a single high amplitude signal return.

APPENDIX A

Data Post Processing

At the completion of the GPR survey, the digitally recorded GPR profiles were downloaded, processed and visually re-examined on a computer monitor using RADAN (Radar Data Analyzer), supplied by GSSI. The GPR data is typically downloaded and printed for trend analysis on multiple 8.5 by 11 inch sheets, generating printed-paper profiles. When viewed in landscape orientation, the long (horizontal) axis represents the horizontal distance along the traverse or transects line, and the short (vertical) axis represents the elapsed time (two-way-travel time), which can be converted to depth estimates. Any additional filtering and gain adjustments made during processing of the data are also depicted on the header sheet of each GPR profile.

<u>Signal Propagation Rates</u>- The depth estimates for GPR reflections provided herein are based on the typical velocities, or two-way-travel-times, of the radar signal in the subsurface materials that are known, or believed, to exist beneath the survey area. Site-specific determination of the radar velocities was not performed and is not essential to evaluate the data for the objectives of this study. HSA typically assumes the following two-way-travel-time velocities in various materials:

Material	Dielectric Constant (Relative Permittivity) *Er*	Pulse Velocity ft/ns	Two-way Travel Time, *TT* (Propagation Rate ns/ft)
Sand (Dry)	4 to 6*	0.38 to 0.48*	4 to 5
Sand (Saturated)	20 to 30**	0.176*	9 to 11
Silts	5 to 30**	0.224**	5 to 11
Clays	5 to 40**	0.192**	5 to 13
Limestone	7 to 9*	0.32 to 0.36*	5 to 6

* Determined from values provided in American Society of Testing & Materials (ASTM) D 6432 Sept. 1999

** Determined from values provided in Sensors & Software Workshop Notes, July 2000

The vertical (elapsed time) scale on the printed GPR profiles is approximately 70 ns/in with a range setting of 238 ns. Intermediate distance marks (vertical solid white lines) on the GPR profiles represent horizontal distances of 20 feet. This horizontal scale is present along the top of each GPR profile indicating the specified intervals. The marks were placed on the GPR profiles automatically by the GPR control unit connected to a survey wheel, which was calibrated prior to the survey to automatically indicate 20-foot intervals.

GPR Results

Interpretation- As with our previous survey, the GPR profiles initially depict a surficial zone of mostly non-reflective to low reflectivity soils within a time window of up to approximately 50 ns, or from near the ground surface to a depth of about 12-1/2 feet. These mostly non-reflective soils are inferred to represent mostly homogeneous or well-sorted surficial sands.

Lastly, the GPR profiles depict a mostly horizontal, at times gently to steeply undulant and laterally extensive to discontinuous group of relatively high intensity reflections that had first arrivals after elapsed times of approximately 30 ns to later arrivals of up to 150 ns, or from estimated depths ranging from about 8 feet to 24 feet, but may be deeper in areas where these reflections appear discontinuous. These reflections appear consistent with intercalated (small, mostly indiscernible layering) seams or laminations of deeper strata of more silty and/or clayey soils, the more clayey soils typically characterized by an increase in the intensity or amplitude of the reflections. Variance in the amount of silt/clay content across the site affects the rate of signal attenuation, allowing for lateral variability in depth of GPR signal penetration.]

The GPR profiles do not appear to have encountered a surficial water table within the depths and areas explored for this study; however, the preceding depth estimate calculations assume water-saturated soils below a depth of 15 feet as with our previous survey.

Identified Features- Similar to our previous investigation, the GPR profiles depict multiple anomalous subsurface geophysical features of interest, some more prominent than others, characterized by either moderate to steeply dipping, localized downwarping associated with discontinuities in the described clayey soil layer reflections, "bowtie" type, vertically stacked reflection patterns consistent with buried or in-filled depressions, and/or localized increases in depth of penetration of the GPR signal.

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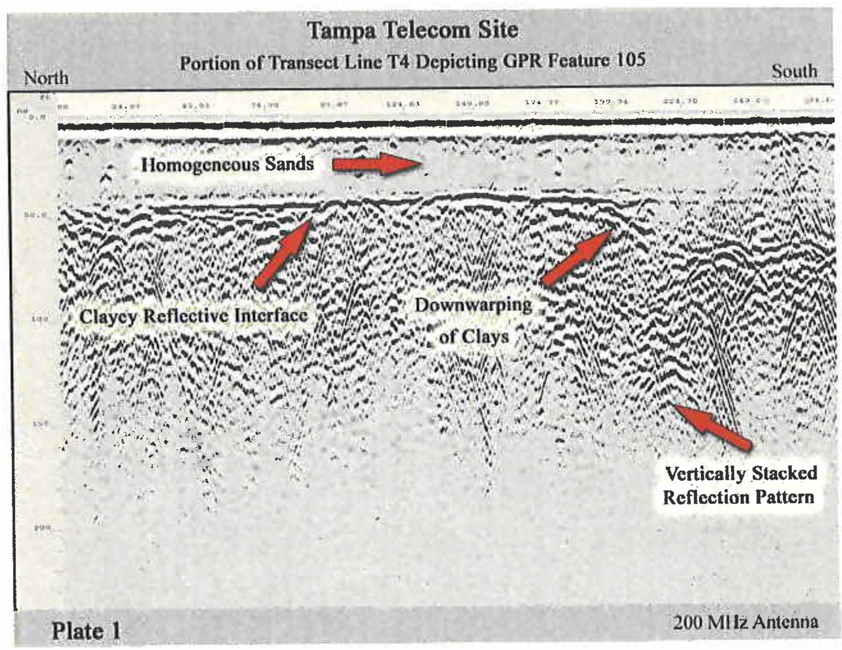
These reflection patterns are, in our experience, characteristic of buried karst topography. These features are inferred to represent sandier, in-filled or buried, possibly paleo-sinkhole or incipient (beginning stages) related depressions in the clay that may mirror similar, deeper seated and possibly sinkhole-related depressions in the underlying limestone.



The described features vary greatly in aerial extent and subsurface vertical relief, measuring between approximately 50 feet to 150 feet wide. ***Plate 1***, shown here, illustrates a typical profile from the GPR survey and details our inference of described soil layers and anomalous subsurface features of interest.

The locations of these features are depicted on **Figure A-1**. In total, 7 features of interest were delineated within the limits of our GPR survey area. If associated with planned development areas, these features represent appropriate target areas for additional testing to evaluate karts or sinkhole erosion of the limestone bedrock and the soils immediately above the limestone.

Sincerely,
HSA Engineers & Scientists

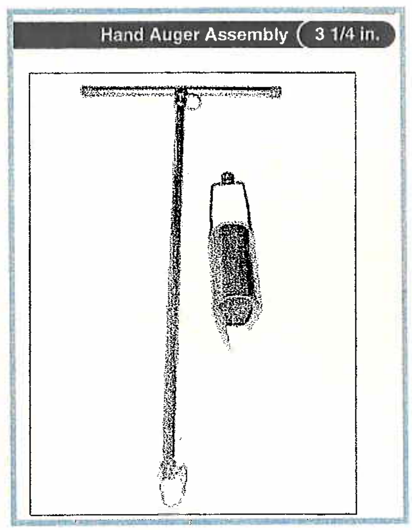
SIGNED ON BEHALF OF
ORDER TO AVOID DELA

Brad A.Dupke, P.G.
Project Geologist
GPR/ERI Certified
Florida License No. 2445

APPENDIX B

HAND CONE PENETROMETER TEST BORINGS

This exploration procedure combines the performance of a hand cone penetrometer sounding with the performance of a hand (bucket) auger boring to produce a quantitative exploration procedure. This procedure is useful in situations where the performance of Standard Penetration Test (SPT) soil borings is not possible because of access restrictions to the desired exploration location, and/or where exploration to shallow depths is sufficient for purposes of analysis. The procedure consists of the performance of a hand cone penetrometer sounding in advance of the excavation of a hand auger boring, along the same alignment. The sounding and boring procedures are individually described below.



Hand Auger Boring – Hand auger borings are performed by simultaneously pushing and rotating a bucket auger tool into the ground to excavate a vertical column of soil at the selected locations. The bucket auger tool consists of a 3-1/4-inch diameter steel cylinder, equipped with soil cutting blades, attached to a "T" handle (*See Photo (R)*).

After the bucket auger bit cuts four to six inches into the ground, the tool is withdrawn. The soil that was directed into the bucket by the cutting blades is then removed, visually classified, and packaged. Although the soil sample obtained in this operation is mixed, its condition is generally sufficient for purposes of identification and classification. The bucket auger tool is advanced in four to six inch increments until the boring is terminated. Extensions to the handle of the tool may be added, as needed, to allow exploration to variable depths. Collapse of the borehole below the surface of the water table of limits the exploration depth using this tool. In addition, the depth of exploration may also be limited when hard materials, such as cemented sand, rock fragments within fill or natural soil deposits, or cemented clay are encountered. It is generally not possible to penetrate such conditions without the powered drilling equipment.

Upon completion of the boring, the borehole is left open, to allow ground water to enter the hole in the saturated soil zone and attain its equilibrium level. The depth to the surface of the ground water is measured following a selected time period, and recorded in the log of the boring. The selected time period will vary depending upon the permeability of the soils that are penetrated, to allow the

APPENDIX B

water in the borehole to approach an equilibrium level. Upon completion of the water level depth measurement, each borehole is filled with borehole cuttings and local soil.

Hand Cone Penetrometer Sounding – The hand cone penetrometer is a widely accepted in-situ testing device used to obtain information concerning the strength and compressibility of shallow foundation soils. In this test, a shaft with a conical point is pushed into the soil and the resistance to penetration of the point is measured. The measured value constitutes the test result. The hand cone penetrometer is a valuable geotechnical engineering tool used in determining the relative compactness of natural and man-placed granular soils.

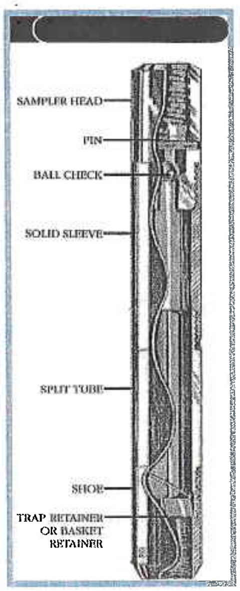
The **HSA** hand cone penetrometer employs a 60-degree hardened steel cone point with a *projected* area of 2.0 square centimeters. The point is located at the end of a smooth cylindrical shaft that is slightly smaller in diameter than the steel cone point. The shaft is attached to a steel ring that is fitted with a handle through which dead weight is manually applied to the assembly. A photograph of a typical assembly is shown here (*See Photo (R)*).

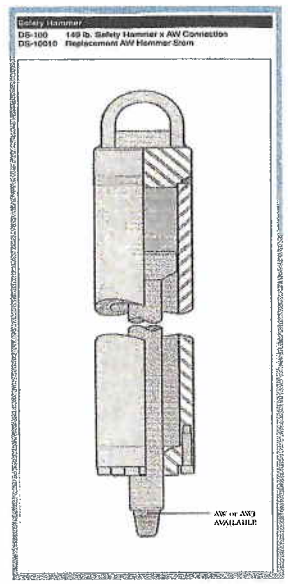
During the test, the rod assembly is pushed through the soil a distance of approximately six inches at a uniform rate. The thrust required to push the cone tip, as indicated by a dial gage that shows deflection in the attached steel ring, is noted and recorded by the operator. The steel ring deflection readings have not been directly correlated either to Standard Penetration Resistance or to Cone Penetration Resistance values. The resistance values provide only a qualitative indication of the degree of relative density of cohesionless soils (sand) and consistency of cohesive soils (silt or clay).

APPENDIX B

STANDARD PENETRATION TEST

The Standard Penetration Test is a widely accepted method of in-situ testing and sampling of foundation soils (ASTM D 1586). For sampling, this test uses a two-foot long by two-inch outside diameter longitudinally split steel cylinder ("split-spoon") sampler, which is held together by a bottom threaded end cap (shoe) and a top threaded ball-check valve assembly. The split-spoon assembly (*See SPT Sampler Detail (L)*) is attached to the end of drilling rods and is driven 18 inches into the ground by successive blows of a 140-pound hammer, "freely" falling a distance of 30 inches (*See Safety Hammer Detail (R)*). The number of blows needed to penetrate each six-inch increment recorded. The penetration resistance (N-value) is computed as the sum of the blows required to penetrate the second and third six-inch increments. After the test, the sampler is extracted from the ground and opened to allow visual examination and classification of the retained soil sample. The N-value has been empirically correlated with various soil properties allowing an estimate of the behavior of soils under load.





SPT Testing Frequency

It is HSA's practice to perform continuous SPT testing of the soils that lie within the upper 10 feet of the land surface, in order to provide detailed stratigraphic and penetration resistance information within a zone of soil that is often critical to the support of structures. Hand Cone Penetrometer Test (HCPT) borings may be substituted for the SPT procedure in the upper zones of the borehole, when care is required to avoid striking buried utility lines. Within the upper 10 feet, the SPT sampler is driven a distance of 24 inches, rather than 18 inches, per test sample. The penetration resistance value of these 24-inch long samples remains the sum of the blows required to penetrate the second and third six-inch increments. Below a depth of 10 feet, one SPT sample is retrieved at intervals of five feet. However, more frequent or continuous SPT testing is done by HSA through depths where a more detailed definition of the soil stratification is required.

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SPT Testing In Resistant Soil and Rock Formations

It is common practice in Florida to perform SPT testing and sampling within both highly resistant soil and lithified sedimentary (bedrock) formations. Penetration refusal, defined herein as the application of 50 blows of the 140-pound hammer described above through a distance of six inches or less, is often encountered in these formations. When penetration refusal is encountered during the SPT test, hammering of the sampler into the formation is halted and the shortened SPT sample is retrieved. The penetration resistance value is recorded as 50 blows through what ever distance was penetrated by the hammer blows (e.g. 50/3", 50/5", etc.). It should be understood that the limestone bedrock formations in Florida are often lenticular in nature, being composed of a mass of generally hard calcareous elastic silt to clayey elastic silt, with lenses of more highly lithified sediment. Therefore, while the SPT resistance values that are recorded in the bedrock zone may suggest that the rock consists of highly lithified sediments, the sampled material is less than 6 inches thick, and it does not necessarily imply that the surrounding bedrock condition is similar to the sampled material. HSA combines the results of these high resistance SPT test results with qualitative notes from the driller to assess whether the high resistance values represent a highly lithified rock mass, or whether they represent thin rock lenses, fragments, or anomalies (e.g. concrete fragments in fill soils or large rock fragments in clay deposits).

Borehole Advancement

HSA uses mud-rotary drilling methods to create the borehole. The boreholes are advanced to the SPT test elevations by rotary drilling with a cutting bit, using circulating fluid to remove the cuttings and hold the fine soil grains in suspension. Usually, the circulating fluid, which is a bentonite drilling mud, also serves to keep the hole open below the water table by maintaining an excess hydrostatic pressure inside the hole. In some soil deposits, particularly in highly pervious ones, flush-coupled casing must be driven to just above the testing depth to keep the hole open and/or to prevent the loss of circulating fluid.

SPT Sample Retention

Representative soil samples are selected from each SPT split-spoon sample and from every different stratum, packaged in airtight containers, and transported to our laboratory for further evaluation and testing, if necessary. Samples that are not used in testing are stored for at least sixty (60) days, prior to being discarded.

APPENDIX B

Ground Water Measurements

When directed, the borehole is kept open until a steady state ground water level is recorded. The hole is then sealed and/or backfilled in accordance with regulatory rules and local practice.

SPT Test Correlations

The relationship between the recorded penetration resistance value and the relative density or consistency of soil is summarized in the following table, for both coarse-grained and fine-grained soil deposits.

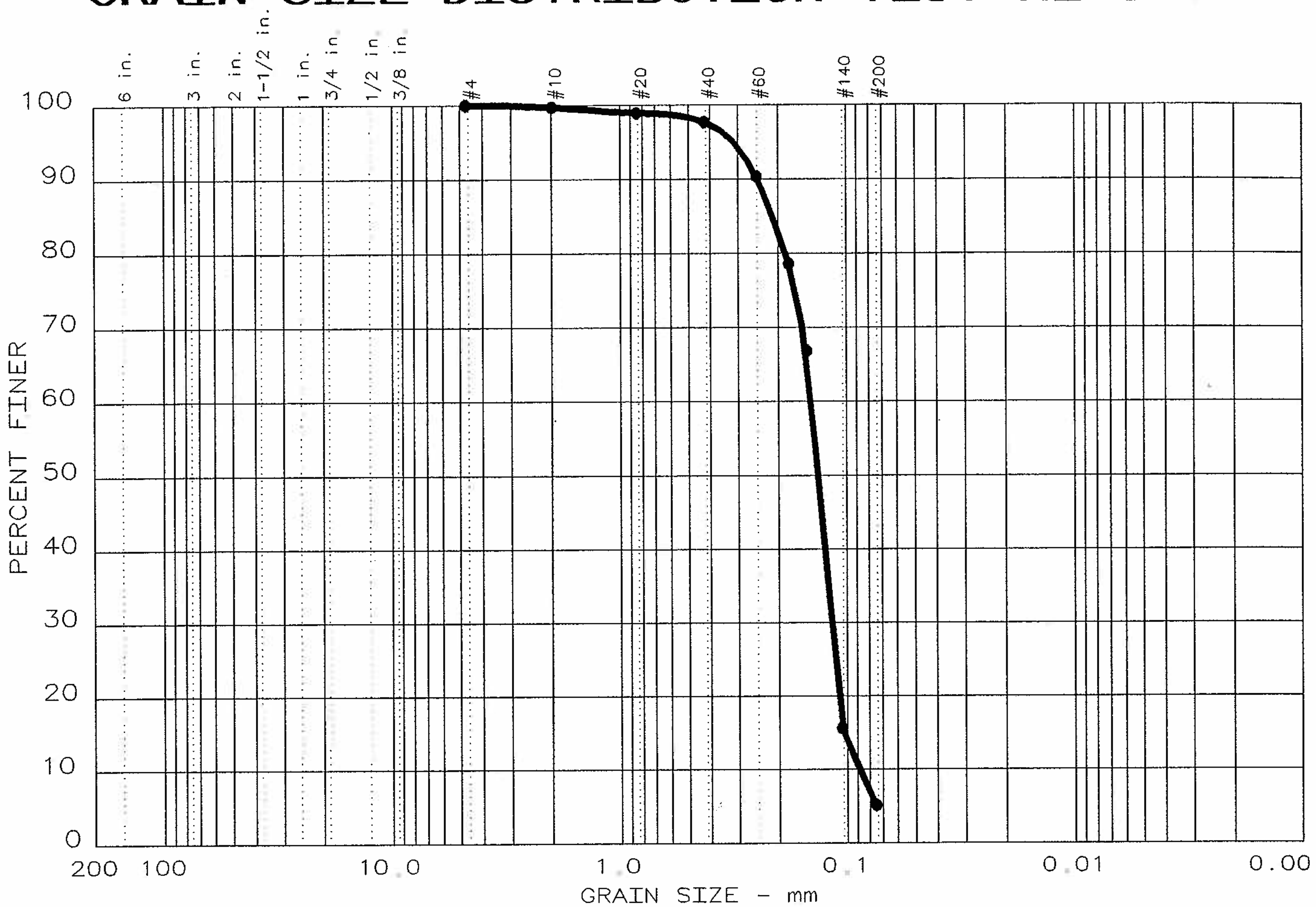
Coarse Grained Soil (Sand) (Less Than 50% Passing US No. 200 Sieve)		Fine Grained Soil (Clay and Silt) (More Than 50% Passing US No. 200 Sieve)	
SPT Resistance (N) Value (Blows Per Foot)	**Relative Density**	**SPT Resistance (N) Value (Blows Per Foot)**	**Soil Consistency**
0 – 4	Very Loose	2 <	Very Soft
4 – 10	Loose	2 – 4	Soft
10 – 30	Medium-Dense	4 – 8	Medium Stiff
30 – 50	Dense	8 – 15	Stiff
50+	Very Dense	15 – 30	Very Stiff
		> 30	Hard
After Gibbs & Holtz			

APPENDIX B

BULK SAMPLING OF SOILS

Bulk sampling of soils is performed, when it is necessary to secure a relatively large sample of soil, for subsequent use in the soils laboratory. Commonly bulk samples are used to provide test specimens for laboratory moisture-density relations (Standard or Modified Proctor) tests, soil-cement mix designs, California Bearing Ratio (CBR) tests, Limerock Bearing Ratio (LBR) tests, and other less common tests, where a large specimen size is needed. Samples my be obtained by any convenient means. The most common are excavation from stockpiles, excavation from test pits, and collection from the soil contained in the flights of a power auger boring. Sample quartering and blending procedures may vary depending upon the requirements of the test for which the sample is being obtained. These samples are classified as being highly disturbed.

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	94.9	5.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.211	0.143	0.134	0.117	0.104	0.0880	1.08	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: P-1, 4'-6"

Date: 10-15-07

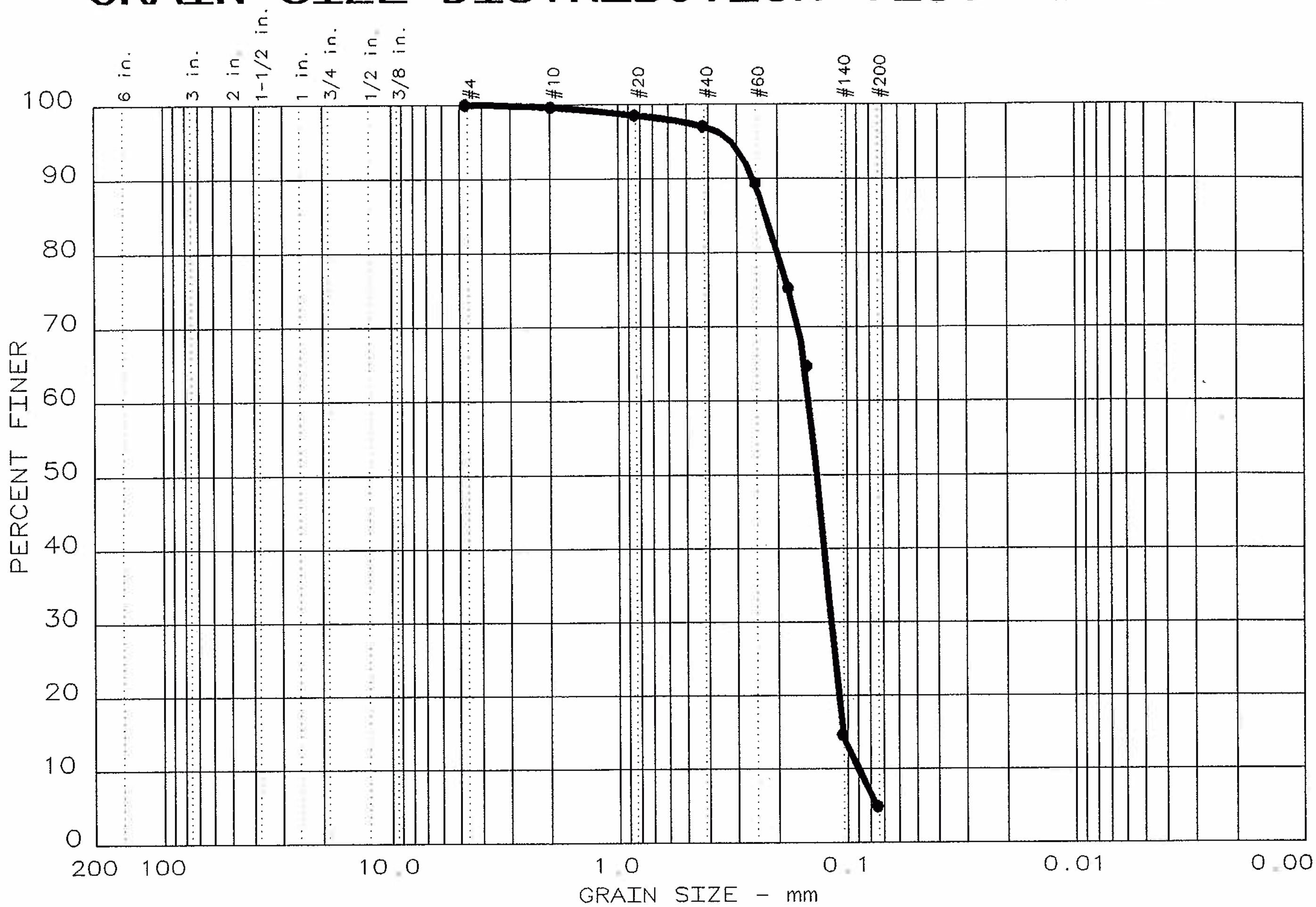
Remarks:

DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4785

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	95.1	4.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.224	0.145	0.135	0.118	0.106	0.0898	1.07	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: P-2, 4'-6'

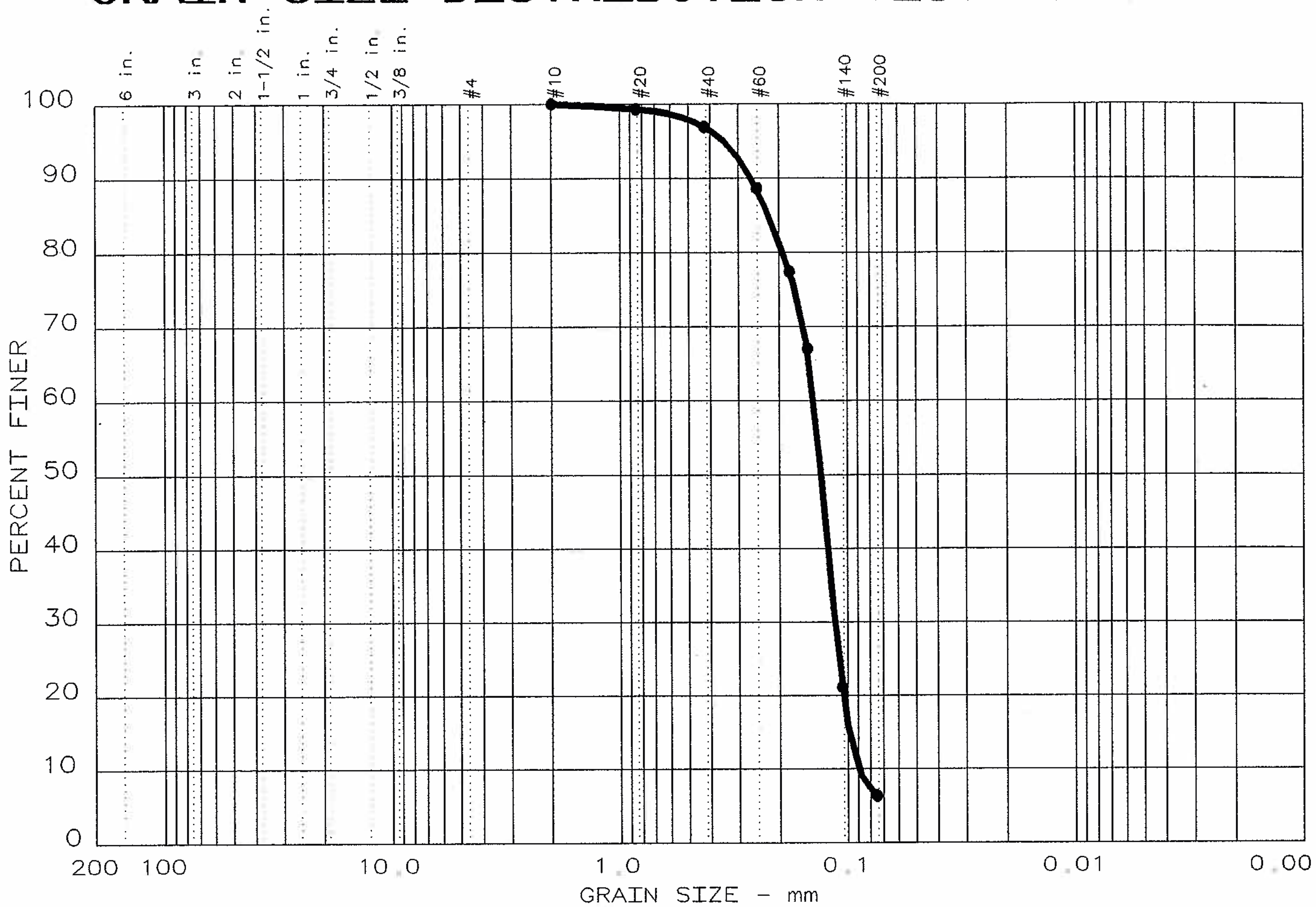
Date: 10-15-07

Remarks:
DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4777

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	93.6	6.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.221	0.141	0.131	0.114	0.0986	0.0897	1.03	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: P-3, 2'-4'

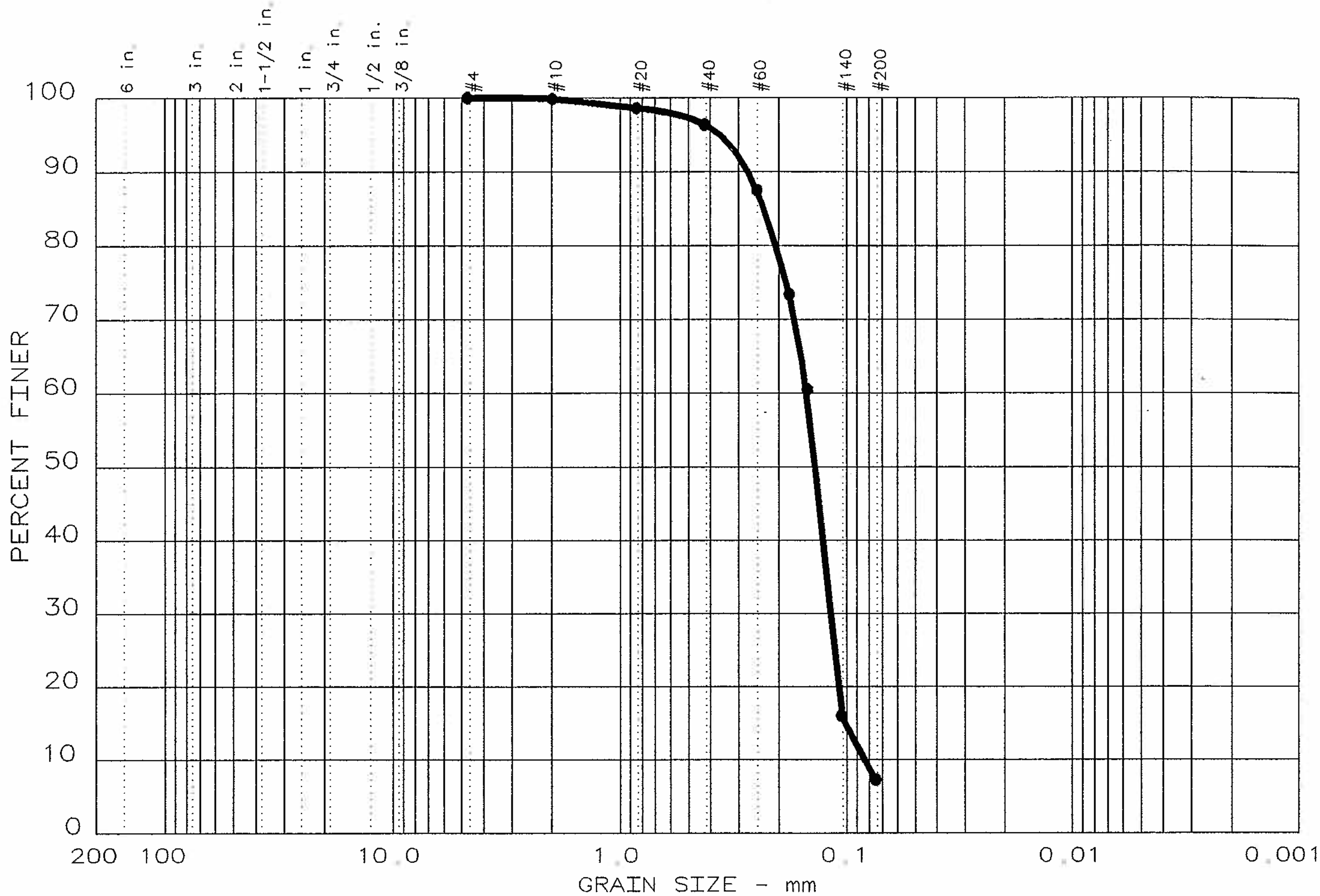
Date: 10-15-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Remarks:
DATE RECEIVED: 10-07-07

Fig. No.: 4778

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	92.7	7.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.232	0.149	0.138	0.118	0.102	0.0835	1.12	1.8

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: P-4, 0'-2'

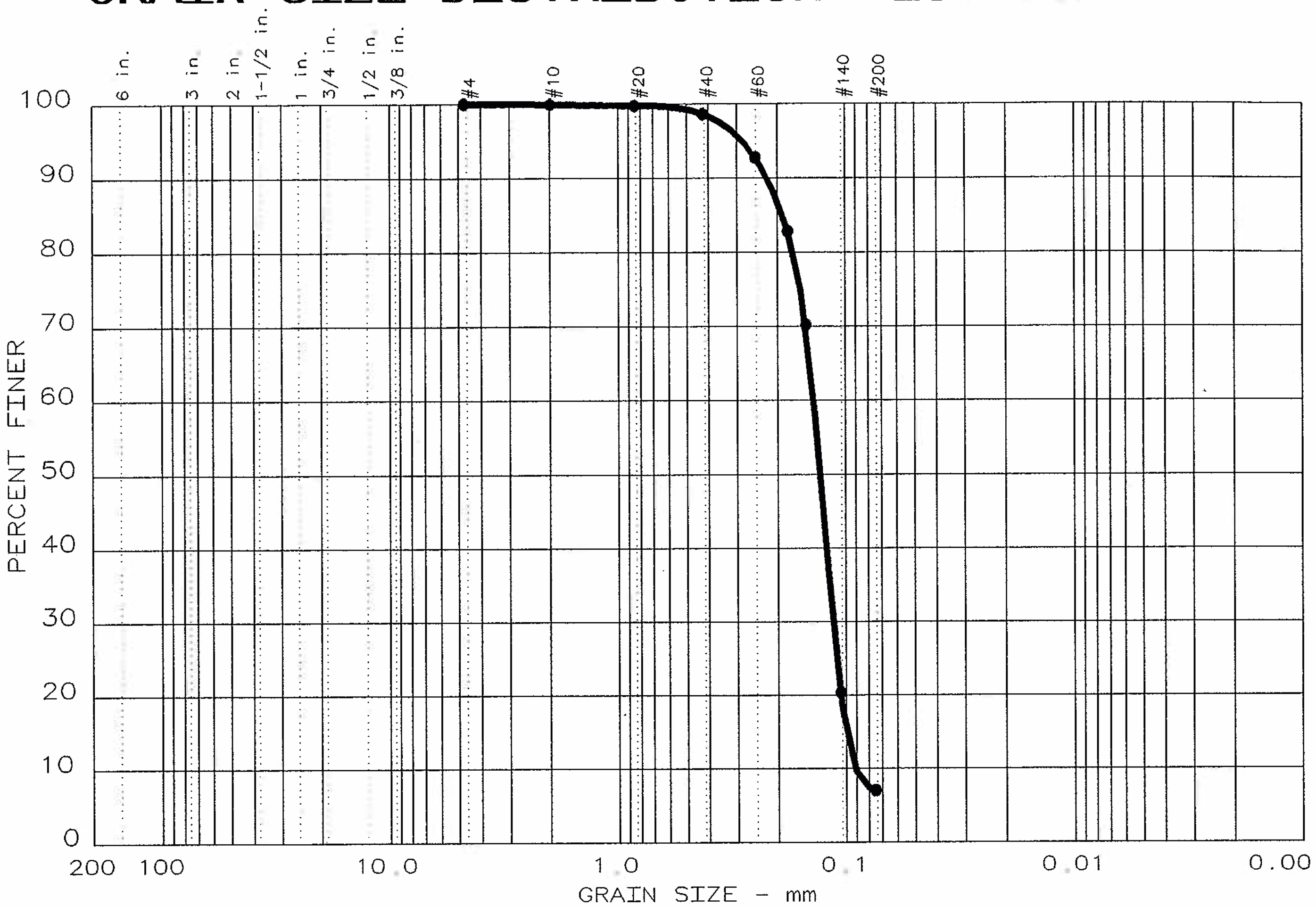
Date: 10-15-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Remarks:
DATE RECEIVED: 10-07-07

Fig. No.: 4773

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	93.0	7.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.189	0.139	0.130	0.114	0.0999	0.0911	1.03	1.5

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: P-5, 4'-6'

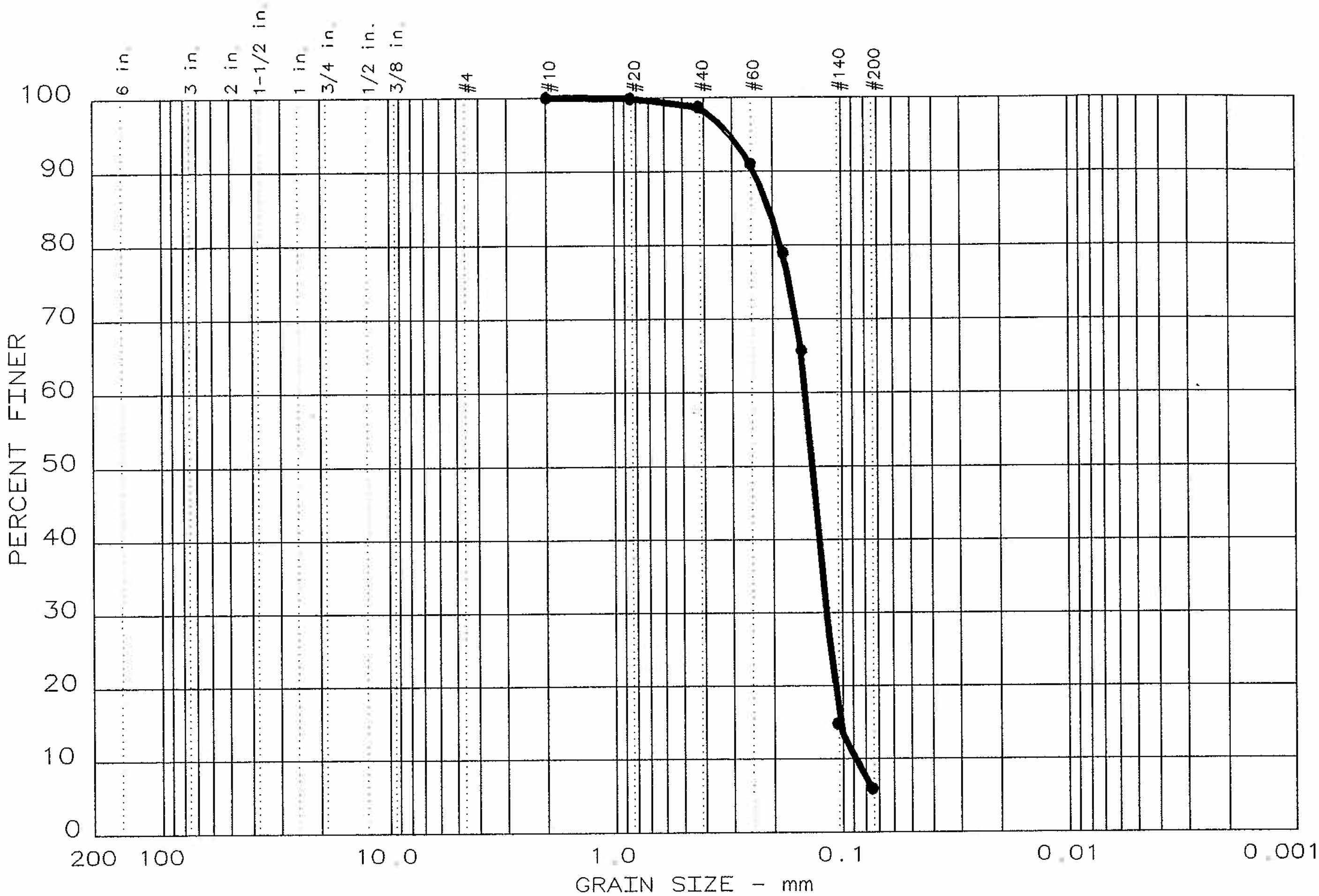
Date: 10-15-07

Remarks:
DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4775

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	94.1	5.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.207	0.144	0.135	0.117	0.106	0.0878	1.09	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-7, 2'-4'

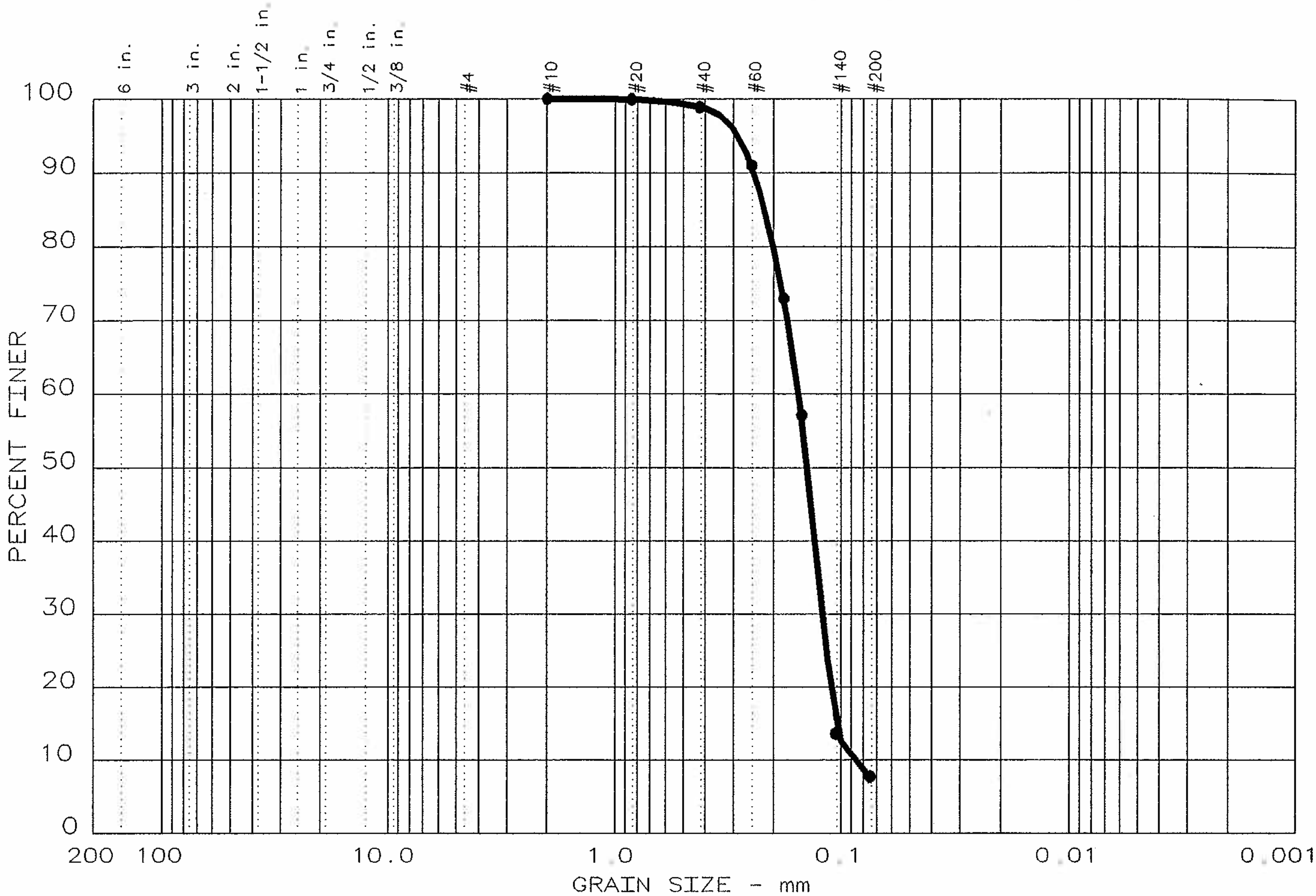
Date: 10-15-07

Remarks:
DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4776

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	92.3	7.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.219	0.154	0.142	0.121	0.107	0.0851	1.11	1.8

MATERIAL DESCRIPTION	USCS	AASHTO
●	SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-8, 8'-10'

Date: 10-15-07

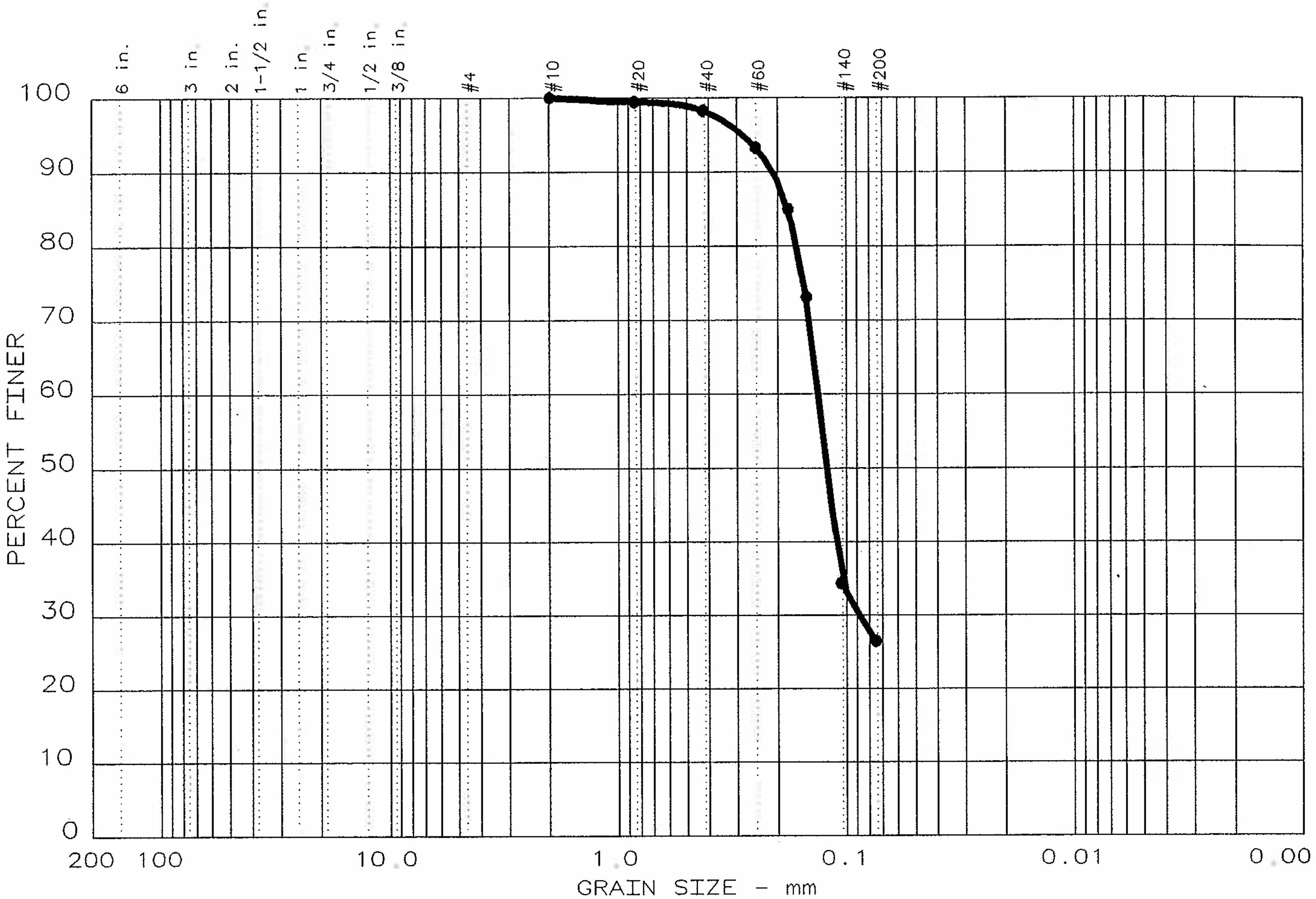
Remarks:

DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4774

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	73.5	26.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.180	0.133	0.122	0.0873				

MATERIAL DESCRIPTION	USCS	AASHTO
●	SM	A-2-4(0.0)

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-9, 8'-10'

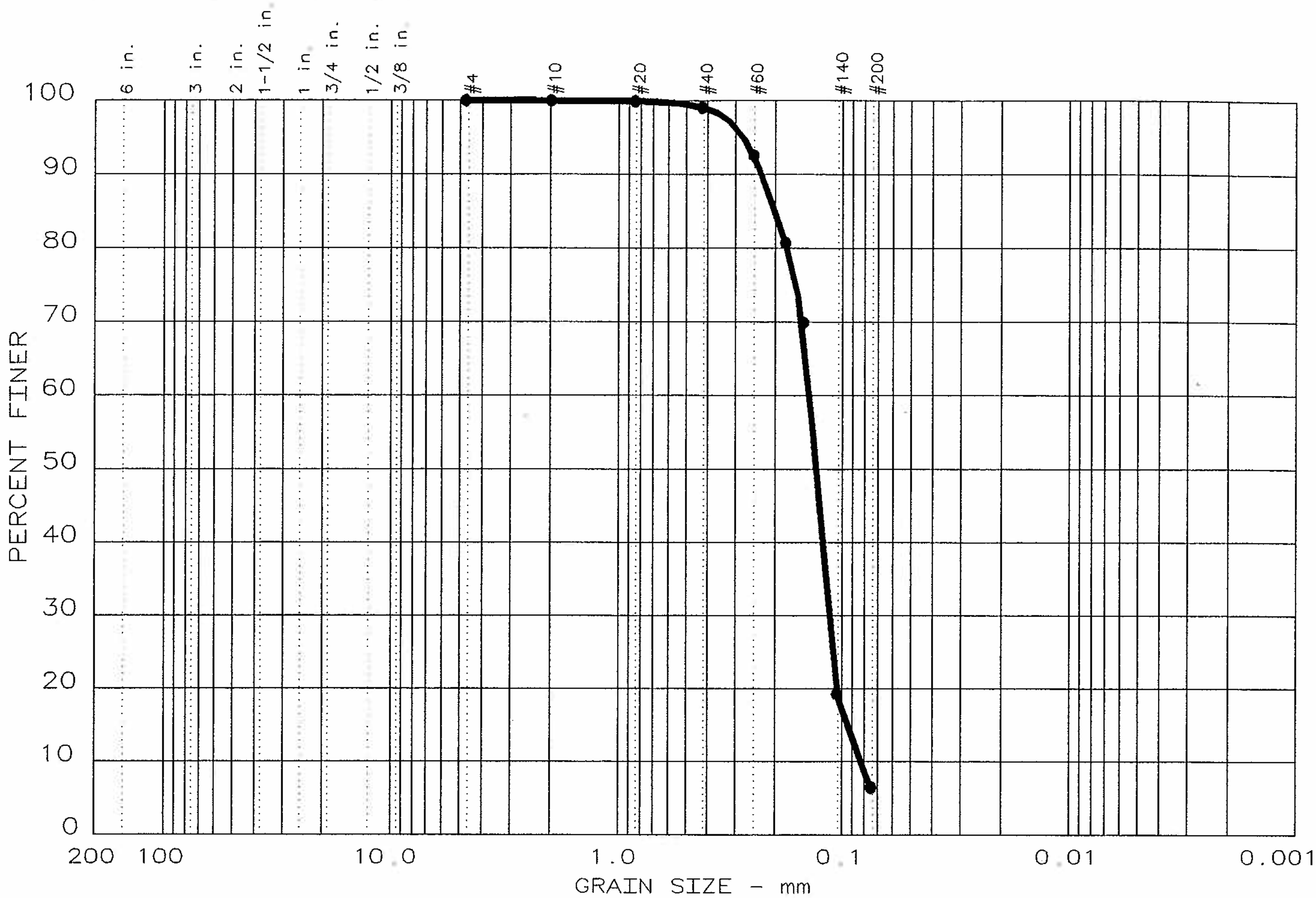
Date: 10-15-07

Remarks:
DATE RECEIVED:10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4784

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	93.5	6.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.201	0.140	0.131	0.114	0.0944	0.0822	1.13	1.7

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800

Project: SOUTHERN OAKS @ TELECOM PARK

● Location: SPT-10, 2'-4'

Date: 10-15-07

Remarks:

DATE RECEIVED: 10-07-07

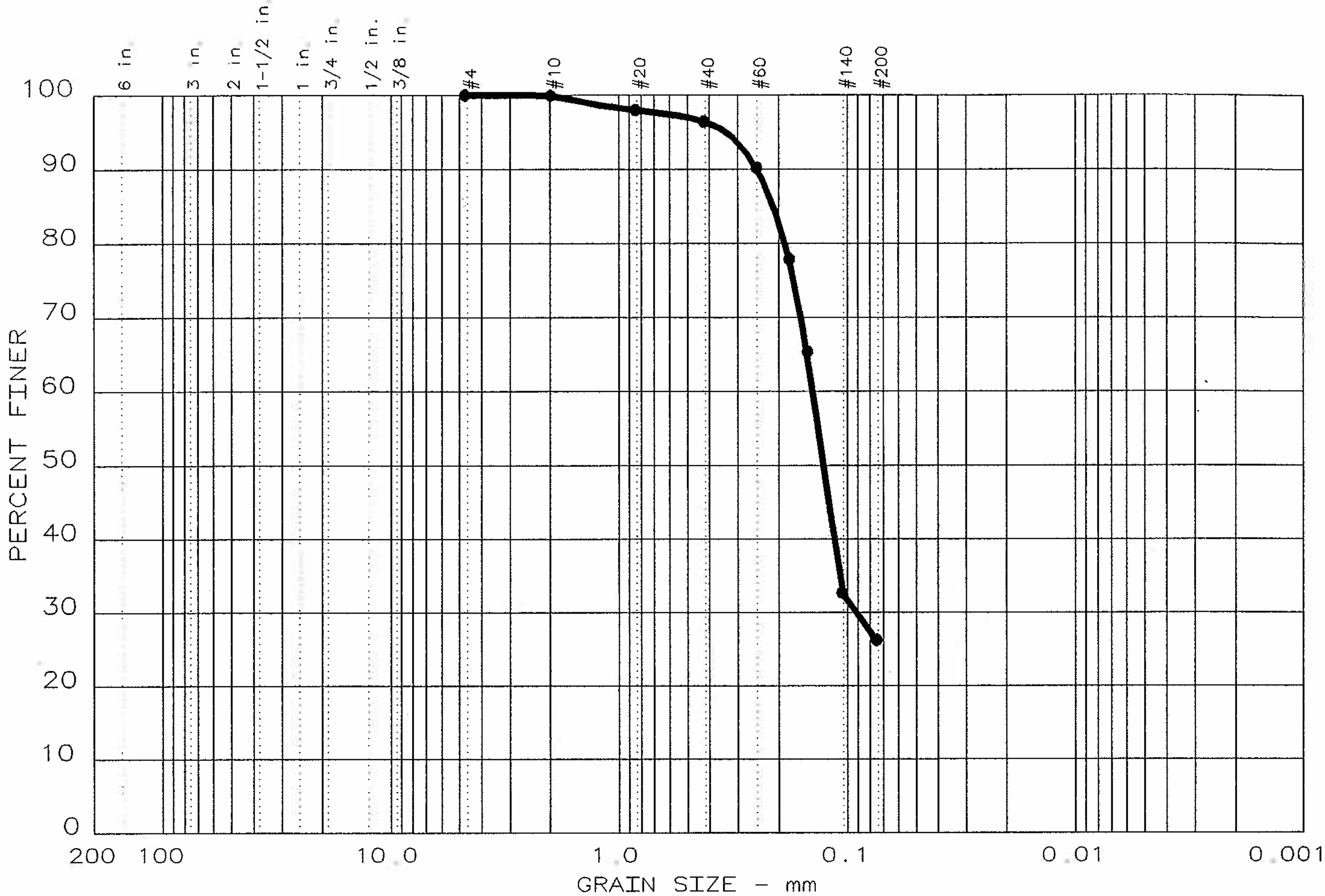
GRAIN SIZE DISTRIBUTION TEST REPORT

HSA ENVIRONMENTAL

ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4779

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	73.8	26.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.210	0.142	0.127	0.0912				

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SM	A-2-4(0.0)

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-10, 8'-10'

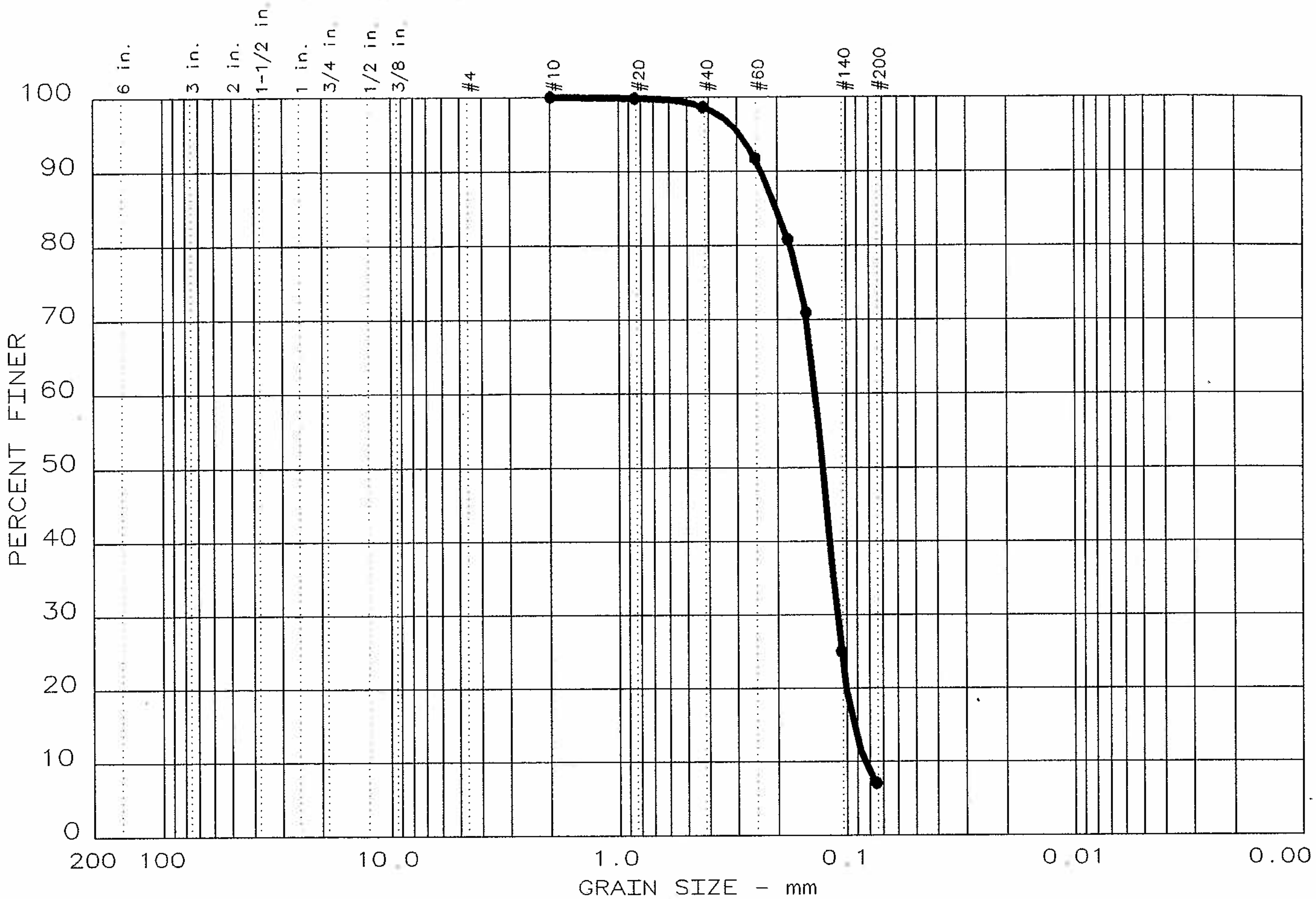
Date: 10-15-07

Remarks:
DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4772

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	92.9	7.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.202	0.136	0.127	0.110	0.0939	0.0837	1.07	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-11, 2'-4'

Date: 10-15-07

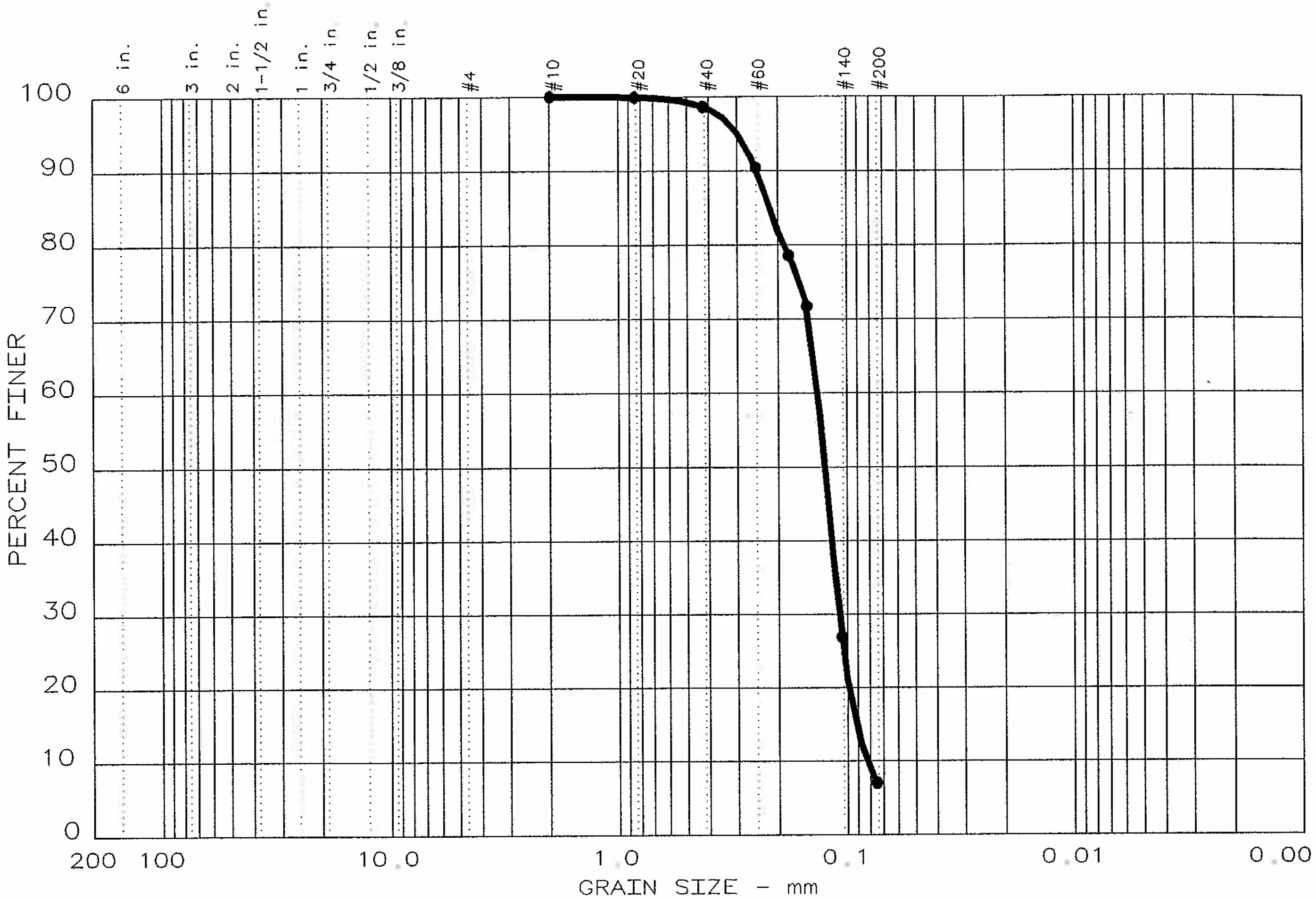
Remarks:

DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4780

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	92.9	7.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.216	0.134	0.125	0.109	0.0918	0.0823	1.07	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-11, 8'-10'

Date: 10-15-07

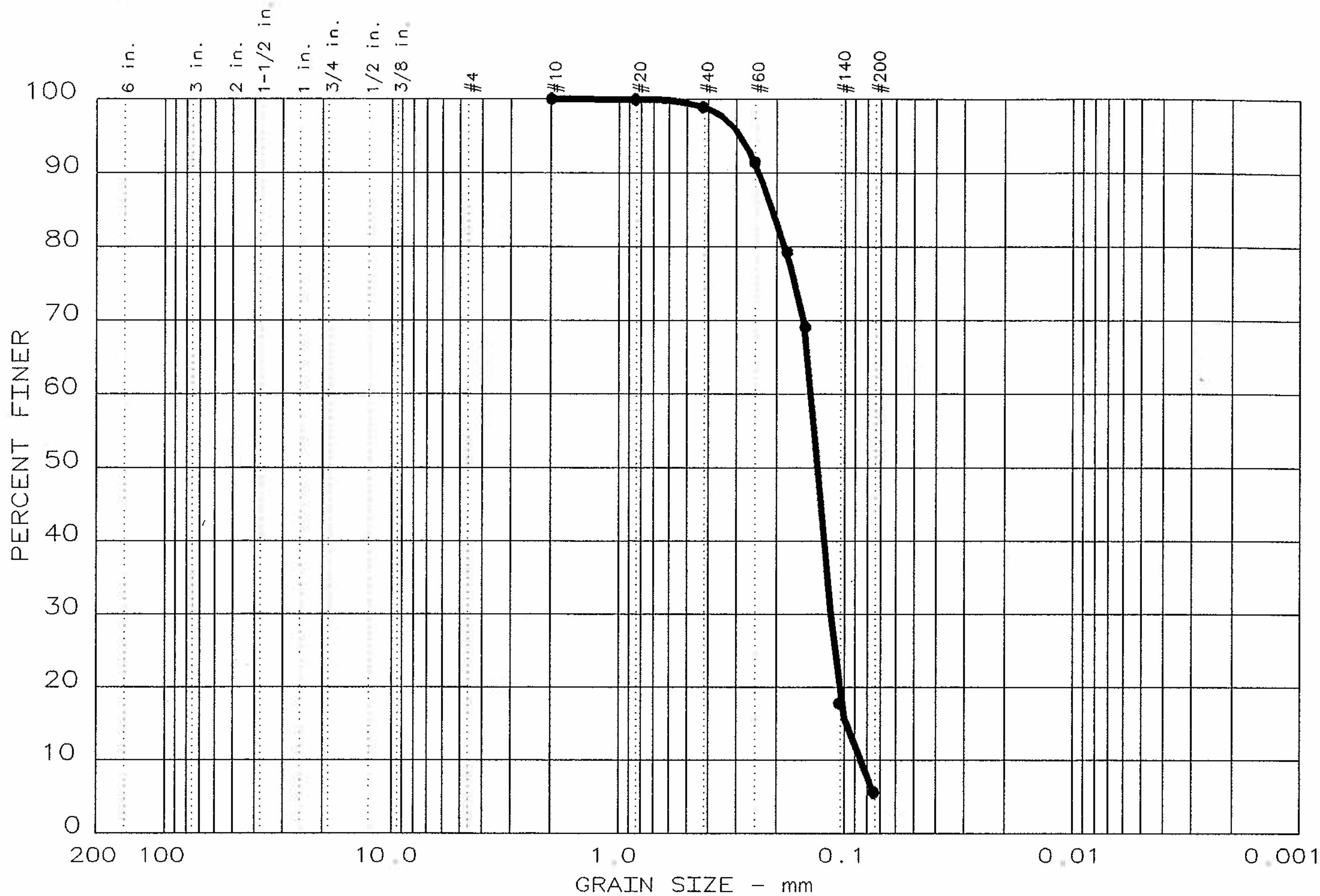
GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Remarks:

DATE RECEIVED: 10-07-07

Fig. No.: 4781

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	94.4	5.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.209	0.141	0.132	0.115	0.0978	0.0849	1.11	1.7

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: B-12, 8'-10'

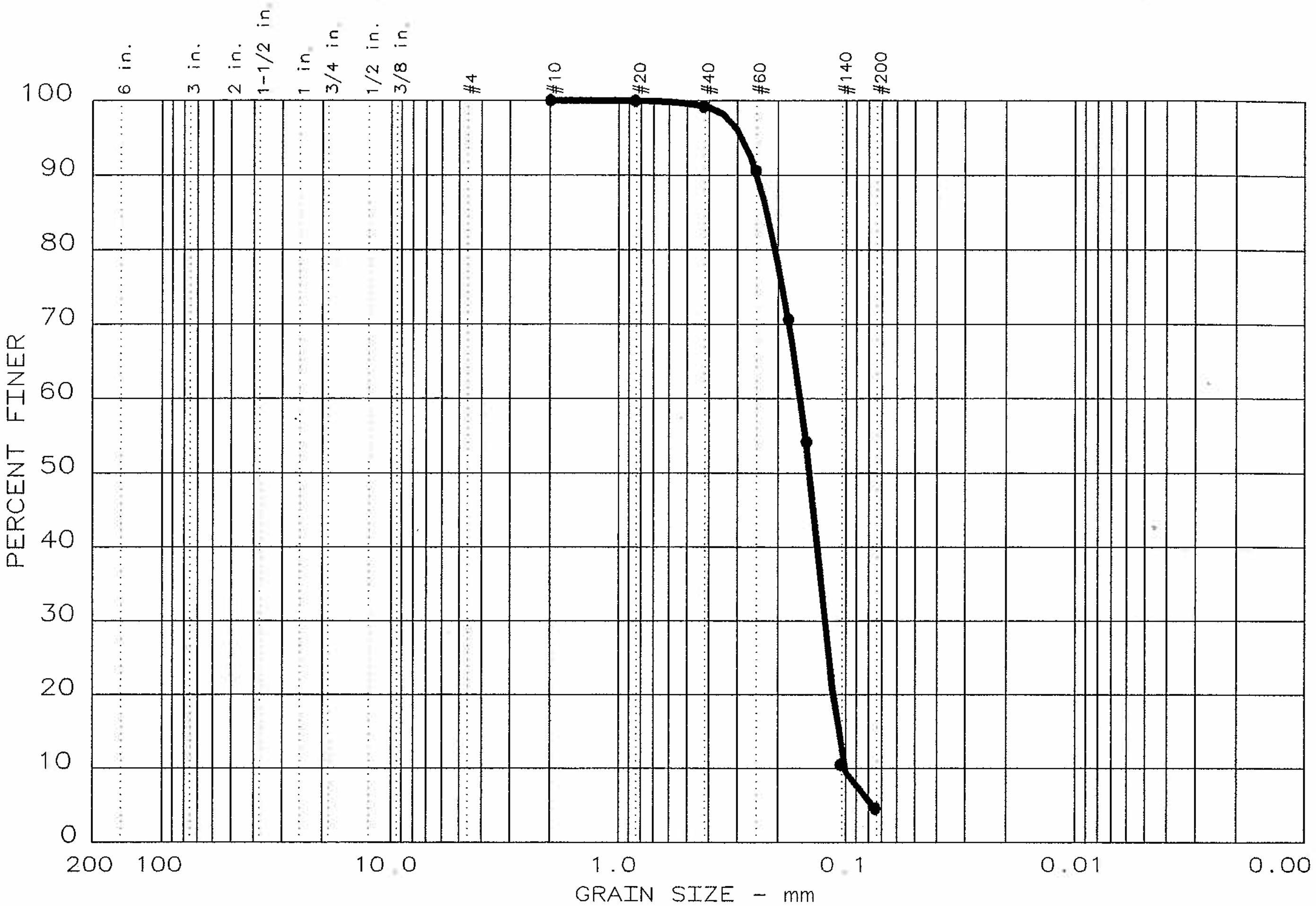
Date: 10-15-07

Remarks:
DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4782

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	95.4	4.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.223	0.159	0.145	0.124	0.110	0.102	0.94	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-13, 2'-4'

Date: 10-15-07

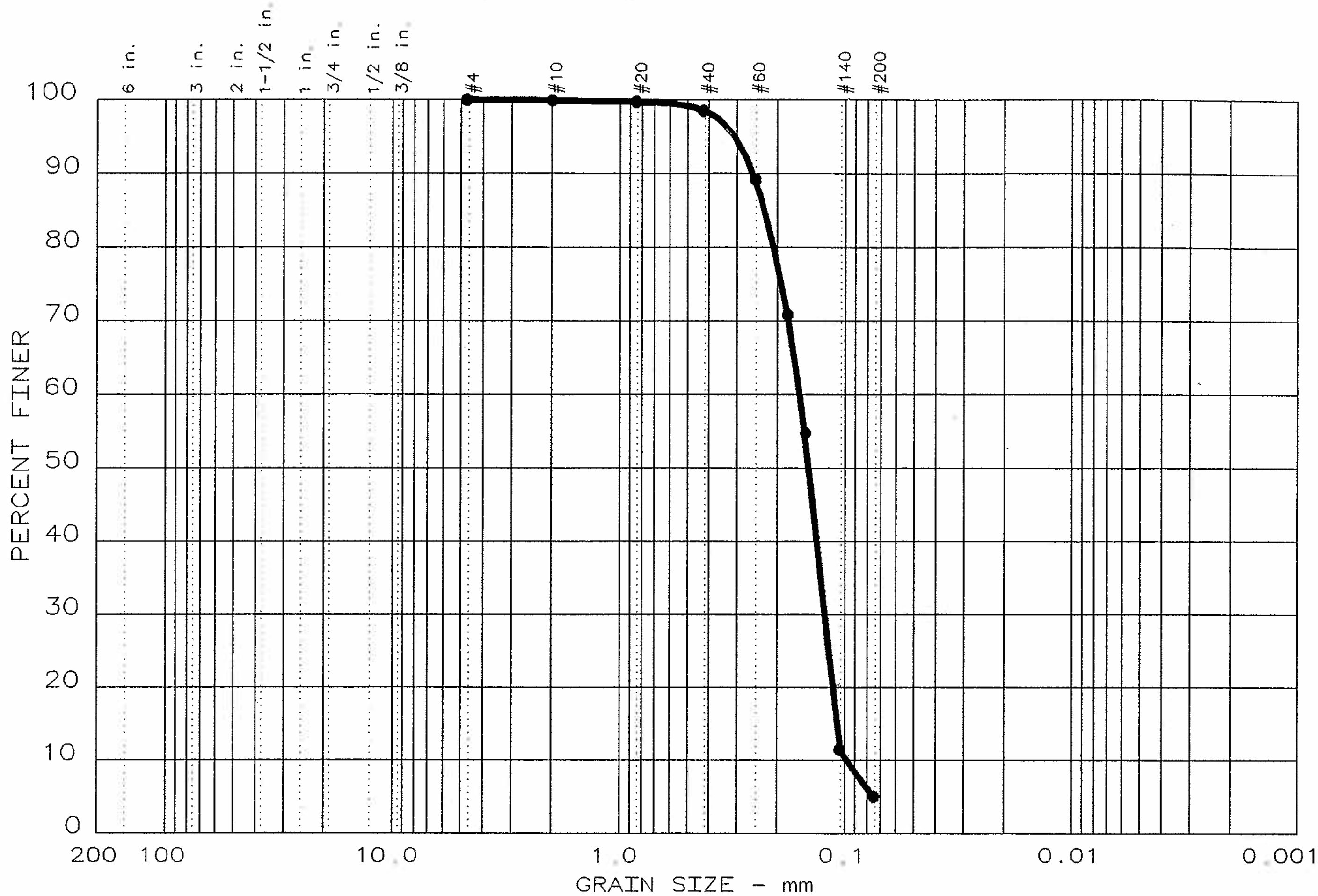
Remarks:

DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4783

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.1	94.9	5.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.227	0.158	0.144	0.123	0.109	0.0971	0.98	1.6

MATERIAL DESCRIPTION	USCS	AASHTO
●	SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-16, 0'-2'

Date: 10-17-07

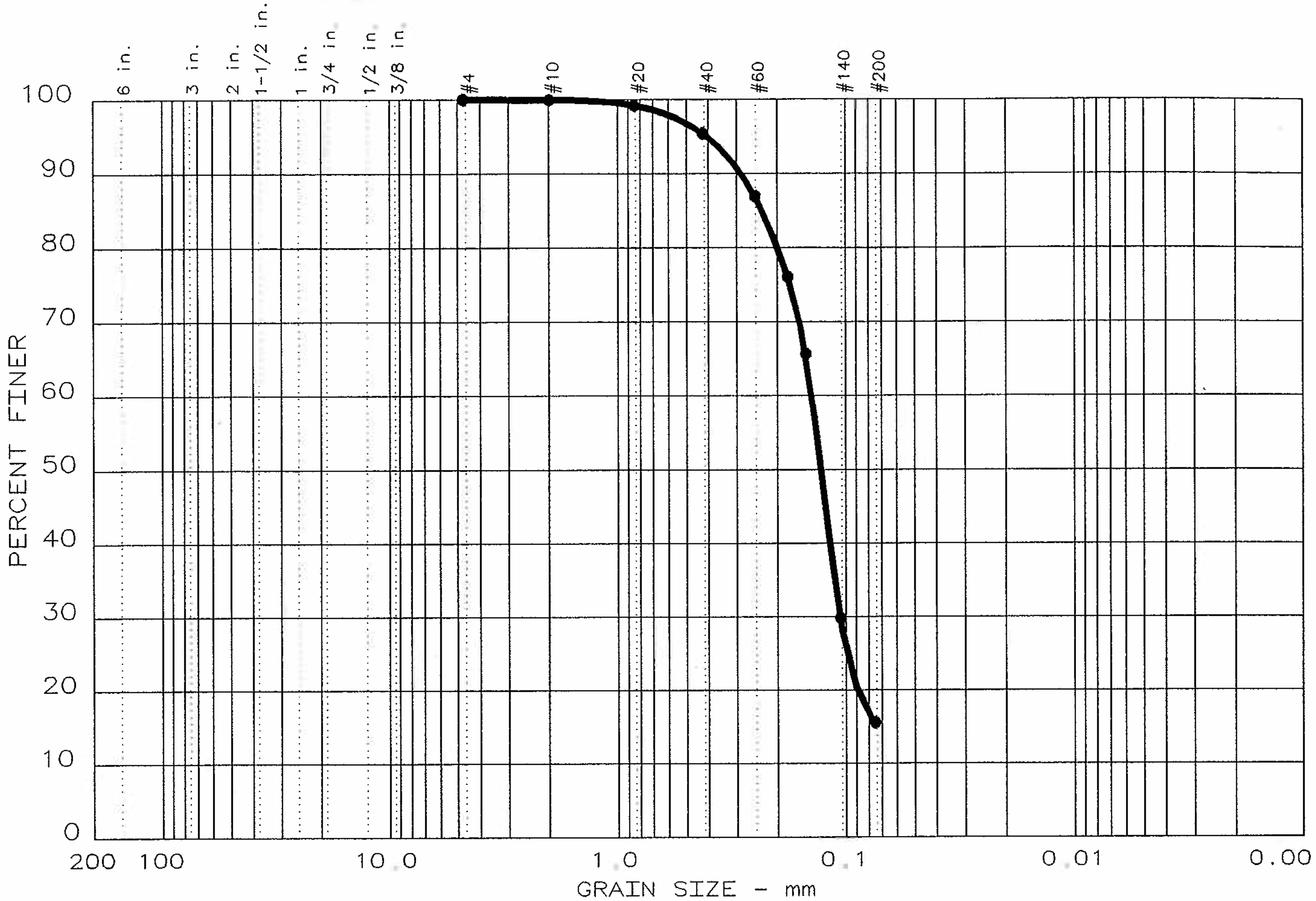
Remarks:

DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4791

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	84.4	15.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.232	0.141	0.128	0.106				

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SM	A-2-4(0.0)

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-17, 13.5'-15'

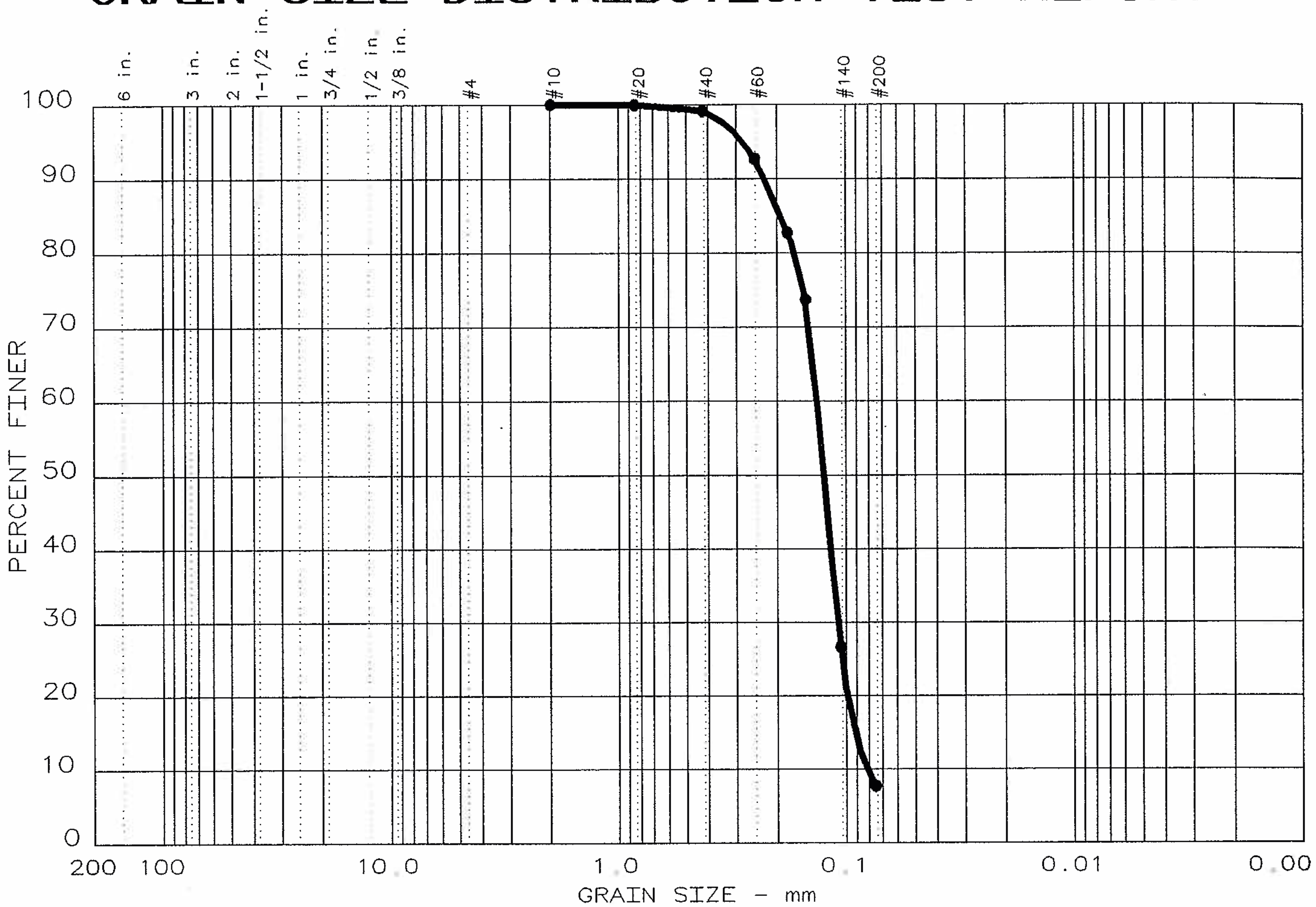
Date: 10-17-07

Remarks:
DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4792

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	92.3	7.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.193	0.134	0.125	0.109	0.0920	0.0811	1.09	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-18, 18.5'-20'

Date: 10-17-07

Remarks:
DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4793

GRAIN SIZE DISTRIBUTION TEST REPORT

	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	93.6	6.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.216	0.149	0.137	0.119	0.103	0.0952	1.00	1.6

	MATERIAL DESCRIPTION	USCS	AASHTO
●		SP-SM	A-3

Project No.: 502535800
Project: SOUTHERN OAKS @ TELECOM PARK
● Location: SPT-18, 23.5'-25'

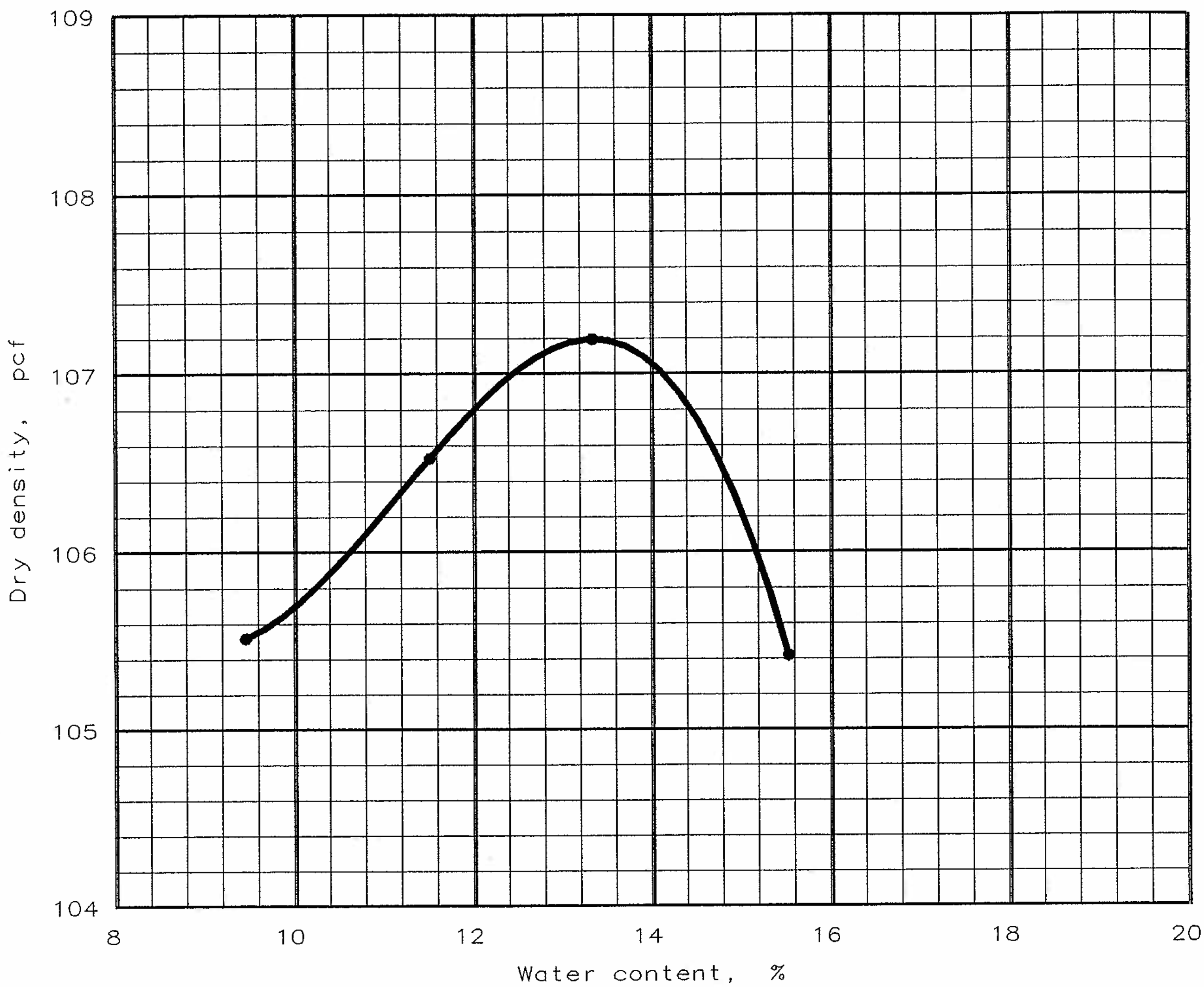
Date: 10-17-07

Remarks:
DATE RECEIVED: 10-07-07

GRAIN SIZE DISTRIBUTION TEST REPORT
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No.: 4794

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: AASHTO T 180 Method A, Modified

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 107.2 pcf Optimum moisture = 13.3 %	GRAY BROWN FINE SAND

Project No.: 502-5358-00

Project: SOUTHERN OAKS @ TELECOM PARK

Location: P-1, 1'-2'

Date: 10-19-2007

Remarks:

RECEIVED: 10-16-07

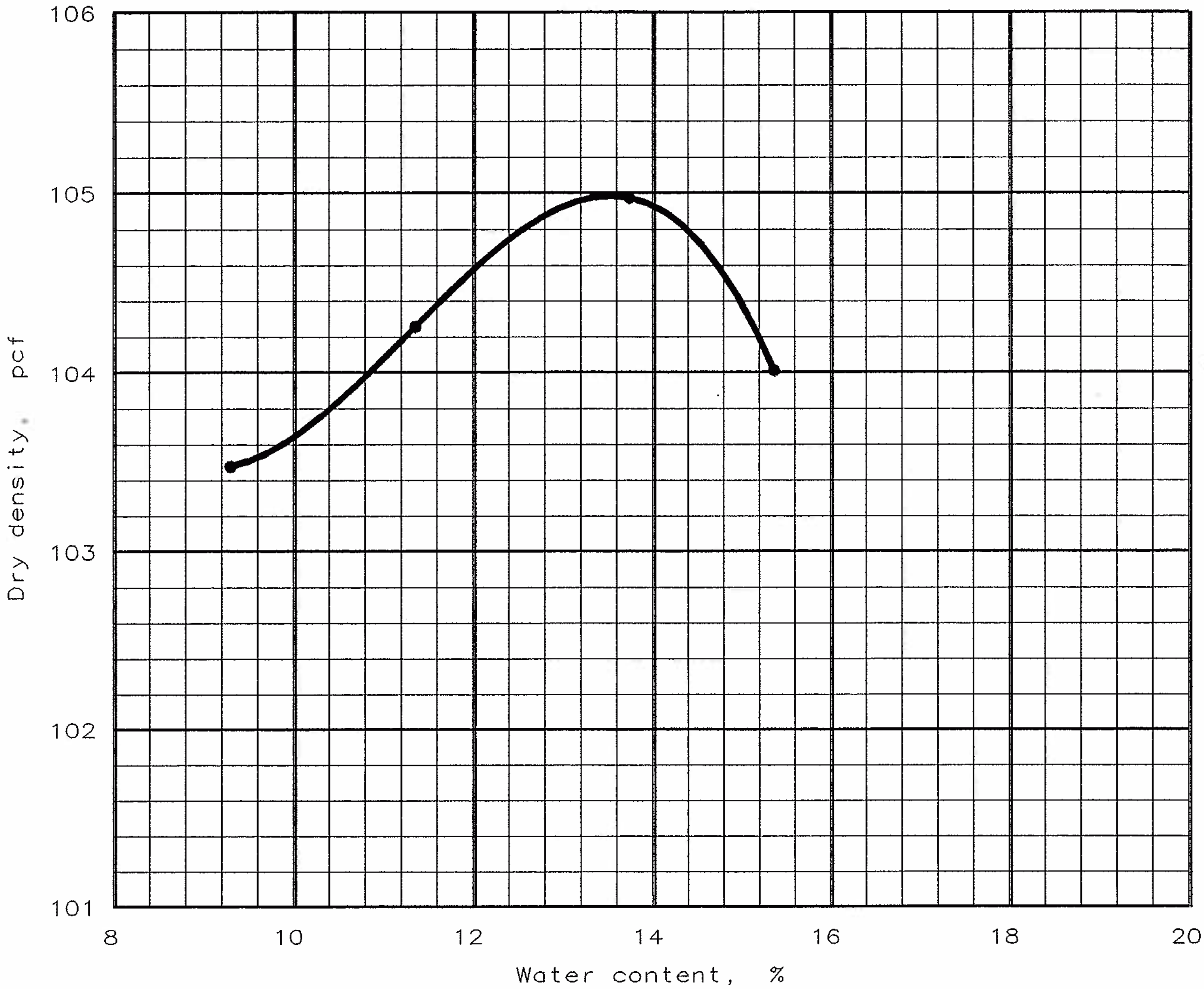
MOISTURE-DENSITY RELATIONSHIP TEST

HSA ENVIRONMENTAL

ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No. 4800

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: AASHTO T 180 Method A, Modified

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 105.0 pcf Optimum moisture = 13.5 %	LIGHT BROWN FINE SAND

Project No.: 502-5358-00

Project: SOUTHERN OAKS @ TELECOM PARK

Location: P-2, 4.5'-5'

Date: 10-19-2007

Remarks:

RECEIVED: 10-16-07

MOISTURE-DENSITY RELATIONSHIP TEST

HSA ENVIRONMENTAL

ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No. 4802

MOISTURE-DENSITY RELATIONSHIP TEST

Test specification: AASHTO T 180 Method A, Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 106.0 pcf Optimum moisture = 13.5 %	GRAY BROWN FINE SAND

Project No.: 502-5358-00

Project: SOUTHERN OAKS @ TELECOM PARK

Location: P-3, 2'-2.5'

Date: 10-19-2007

Remarks:

RECEVIED: 10-16-07

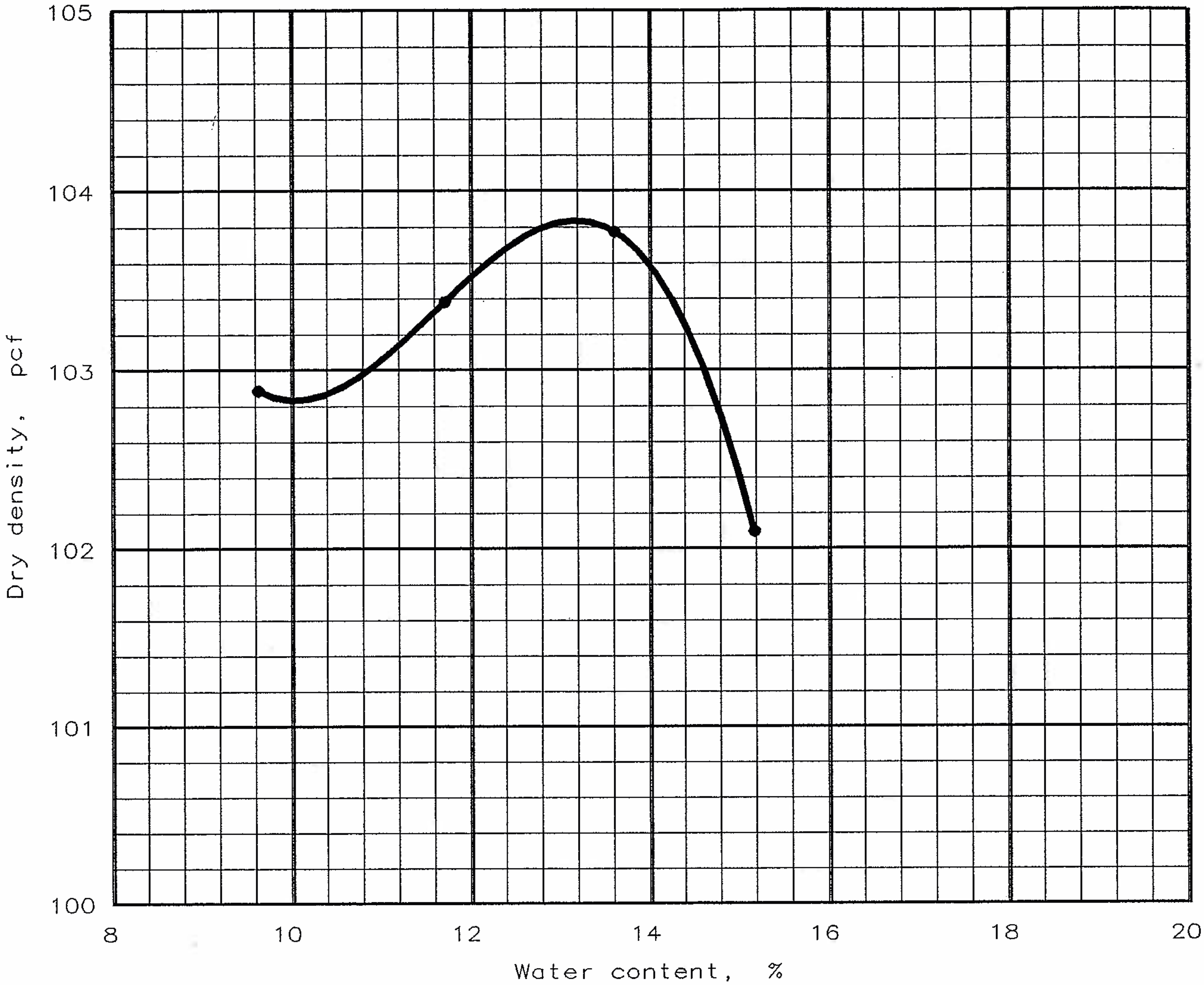
MOISTURE-DENSITY RELATIONSHIP TEST

HSA ENVIRONMENTAL

ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No. 4798

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: AASHTO T 180 Method A, Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 103.8 pcf Optimum moisture = 13.2 %	GRAY BROWN FINE SAND

Project No.: 502-5358-00

Project: SOUTHERN OAKS @ TELECOM PARK

Location: P-5, 2'-2.5'

Date: 10-19-2007

Remarks:

RECEIVED: 10-16-07

MOISTURE-DENSITY RELATIONSHIP TEST
HSA ENVIRONMENTAL
ENVIRONMENTAL & GEOTECHNICAL ENGINEERING

Fig. No. 4799

Contract Number **1799-000-50-501165 K**

Exhibit 1

PROJECT AGREEMENT

The undersigned Engineer agrees to perform the Services described below within the time and for the compensation described below, and Ryan agrees to pay such compensation, all in accordance with the terms and conditions contained herein and in that certain Master Agreement entered into between Engineer and Ryan. Such Master Agreement is hereby incorporated herein by reference and made a part hereof.

Project: Two (2) four story office buildings each consisting of 110,000 square foot facility to be constructed on a site located at Telcom Office Park, folio – 37476.5302 (approximately 18.4 acres).

Services: Geotechnical Study services which are described in Attachment A hereto.

Completion Date: Approximately 10/31/07

Ryan Project Manager: Brian Smith

Compensation (check appropriate box):

☐ Lump Sum $ N/A

■ Hourly Rates specified in Attachment B hereto, ☐ estimated/ ■ guaranteed to not exceed $ 34,525.00

Plus reimbursable expenses described in Attachment C hereto.

Other Matters:

Date: October 11 , 2007.

ENGINEER:

RARE EARTH SCIENCES, INC.
dba HSA ENGINEERS & SCIENTIST

By: Joseph A. Edwards
Its:

RYAN:

RYAN COMPANIES US, INC.

By: Brian R Smith
Its:

Attachment A

Description of Service

Design Basis Subsurface
Exploration and Geotechnical Evaluation Services
Proposed Telecom Park Office Building Site
Telecom Parkway East and Hollow Stump Road
Tampa Telecom Business Park
Tampa, Florida
HSA Proposal Number: 502-5358-98

EXISTING SITE CONDITIONS

The subject property is a rectangular parcel of land that covers approximately 18.4 acres (M.O.L), and is located east of the intersection of Telecom Parkway East and Hollow Stump Road, in Tampa, Florida. Most of the land is vegetated in a mature oak woodland, with a sparse to moderately dense under-story of vines, sapling and scrub ground cover vegetation. We have determined that most areas can be accessed by a conventional truck-mounted exploration equipment, after paths have been cleared between the trees to permit access to selected soil boring locations. The proposed study fee is based on that assumption.

PROJECT DESCRIPTION

Proposed Development – We understand that the proposed development includes the construction of two four-story office buildings, each providing 110,000 square feet of office space and occupying a rectangular footprint of about 27,500 square feet. The development will also include the installation of a ±2.8-acre storm water management basin at the east end of the property. Most of the balance of the property will be paved to accommodate employees and visitors. Lastly, the development will include the installation of standard power, water, wastewater, communications, and other miscellaneous standard drainage and utility infrastructure. In performing this study, we understand that foundation loads may range from light (2,000 pounds per lineal foot, 50,000 pounds, and 100 pounds per square foot for linear, concentrated and area wide loads, respectively) to moderately heavy (10,000 pounds per lineal foot, 300,000 pounds, and 200 pounds per square foot for linear, concentrated and area wide loads, respectively).

Expected Subsurface Conditions - Based on the results of our due-diligence study of this property, we have learned that between 3 and 37 feet of cohesionless sand deposits overlie variably-thick layers of cohesive soils, all lying upon the surface of the carbonate bedrock, which occurs at depths ranging between 18 and 47 feet, where the rock has not been corroded away. Where the rock has been corroded, we have discovered that the less-corroded generally hard rock surface lies at depths that occasionally exceed 60 feet below the land surface, and that the sediments that overlie the rock surface are abnormally weak within extended depth zones. These conditions are consistent with those expected to be created as the result of past sinkhole subsidence activity. These latter conditions were discovered in areas that were indicated to be suspect, based on the result of our geophysical (Ground Penetrating Radar (GPR)) survey of the property. One of the identified GPR anomaly areas lies within the western most office building footprint, and others lie at each end of the open space between the two proposed buildings.

FIELD EXPLORATION SERVICES

Geotechnical engineering geophysical survey services were requested to provide design-basis information pertaining to the development of foundation and pavement design and earthwork construction recommendations to properly support the elements of the proposed office building site. The herein described scope of work will explore both shallow and deep subsurface conditions below the proposed improvements, within the expected zone of stress of shallow foundations, within the expected stress zone of pavement and vehicular traffic, within the zone where storm water will percolate and flow through the storm water management basin, and within zones where sinkhole conditions may exist. Exploration related to design of storm water management basins will include soil borings drilled to depths sufficient to identify the depth to the surface of the first hydraulically-restrictive soil layer, and performance of infiltration tests to comply with requirements of the City of Temple Terrace, Florida.

Layout of Exploration Elements – Layout of the positions of the soil borings located outside of the proposed building footprint will be made using hand-held GPS devices, referenced to the property plan layout showing existing land surface topography, using the Florida State Plane coordinate system. Survey lathe stakes will be installed at all the exploration points for future survey reference. Land surface altitudes reported in our study will be determined based on interpolation between the ground surface elevation contour lines, at the position of the soil borings, plotted on the property plan layout. In the event that you require the positions and altitudes of the borings to be determined to a greater degree of precision than that indicated by our proposed methods, we request that you engage a land survey firm. Layout of the soil borings and GPR transect pathways that lie within the proposed building envelope will be made with reference to building perimeter boundary and corner stakes, which we understand will be placed on the land by survey personnel of King Engineering, Inc.

Transect Pathway Clearing for Ground Penetrating Radar Survey – Prior to the initiation of the gathering of data in the GPR survey, a bush hog mower will be used to clear the under-story ground cover vegetation, along the selected GPR alignment paths, to allow the survey equipment to traverse the transect alignments, as necessary. We have budgeted one (1) day of this activity in our proposed fee.

Ground Penetrating Radar Survey – A GPR survey of the entire building area envelope on the property will be performed in two phases, to assess the lateral continuity of subsurface conditions on the property, and to detect the position and lateral extent of anomalous conditions, within the depth of signal penetration, which may be related to the presence of conditions conducive to the development of sinkhole subsidence on this property. The first phase of the GPR field work will consist of scans of the entire building area envelope along approximately parallel transect paths, nominally spaced 20 feet apart, consistent with the position of major trees, using a 200 MHz antenna, to maximize the depth of penetration. It may be necessary to alter the paths used in some instances, in order to avoid excessive clearing of pathways.

The second phase of the GPR field work will consist of scans of the entire building area envelope along transect paths perpendicular to the first phase paths, in order to provide additional detail of the lateral extent of anomalous areas that were identified in the first phase of the field work, and thereby allow us to produce a more informed estimate of the expected extent and cost that may be necessary to mitigate sinkhole conditions that lie in areas that may threaten the buildings, in the event that the anomalous areas are activated by changes in the hydrogeologic conditions on the property or in the geologic stress in the soils that lie in those areas.

Soil Borings, Sampling and Field Testing – In preparing this proposal, we have assumed that the boring locations will be accessible to our truck-mounted drilling equipment. The number and depth of the soil borings that are included in the scope of our exploration was set forth by our firm, in light of the expected conditions, and to address the potential geologic hazards, that may underlie this property, which will be delineated in the proposed GPR survey. The scope of services to be provided is listed below:

- Notify Sunshine State One Call of Florida to mark underground utilities.
- Locate and layout all test locations using taped measurements within the proposed building envelopes, and a Global Positioning System (GPS) device, elsewhere.
- Mobilize a truck-mounted drill rig to the project site.
- Perform twelve (12) Standard Penetration Test (SPT) borings, each to a depth of 30 feet, within the proposed building areas, to reveal the foundation subgrade conditions below the buildings.
- Perform up to four (4) Standard Penetration Test (SPT) borings to penetrate between 5 and 10 feet of the rigid surface of the carbonate bedrock, at selected areas within the to-be-delineated GPR anomaly areas, and thereby provide information that will be used to assess those areas and develop pre-construction mitigation plans, designed to prevent the development of sinkhole subsidence below the proposed buildings. Our budget includes performance of these borings to an average depth of 65 feet. In the event that additional or deeper borings appear to be necessary

to adequately delineate sinkhole conditions, we will contact you to gain your approval, before proceeding with such additional work.

- Perform five (5) Standard Penetration Test (SPT) borings, each to a depth necessary to penetrate between 3 and 5 feet of the first hydraulically restrictive soil layer in the proposed storm water management basin, but not more than 25 feet below the land surface. Preliminary boring SPT-4B was drilled at a location that will be incorporated into the necessary exploration of the storm water basin. Our proposed budget includes performance of borings to 25 feet, each.
- Perform six, (6) Double-Ring Infiltration (DRI) tests within soils in the storm water management basin through which water will pass. The proposed basin bottom has been preliminarily set at 25 feet NGVD. In order to perform these tests at the proposed bottom of the basin, it will be necessary to excavate DRI test pits that range in depth from 4 to 9 feet below the existing land surface. Our budget includes performance of these tests, plus the services of a backhoe for a period of two days, to excavate and backfill the potentially deep test pits;
- Perform nine (9) hand auger soil borings with associated hand cone penetrometer soundings, each to a depth of 10 feet below the land surface, to provide information on the stratigraphy that underlies proposed pavement areas, and provide a general coverage of the property to allow us to address utility infrastructure earthwork issues.
- Excavate soil within the DRI test pits to collect representative samples of soil that may be used as fill in the earthwork operations on the project site. The collected samples will be used as specimens in proposed laboratory testing to determine their moisture-density relationships.

The following table summarizes the scope of the proposed exploration of subsurface conditions:

Exploration Item Summary				
Site Feature(s)	Number of Explorations	Exploration Type (1)	Proposed Exploration Depth (Feet)	Extended Depth Quantity (Lineal Feet)
Proposed 4-Story Office Buildings	12	SPT	30	360
GPR Anomaly Areas	4 (Estimated)	SPT	65 (Estimated)	260
Storm Water Management Basin	5	SPT	25 (Estimated)	125
Storm Water Management Basin	6	DRI	Varies	Not Applicable
Pavement Areas	9	HA	10	90
Pavement Areas	9	HCPT	8	72
(1) SPT – Standard Penetration Test Boring MA – Mechanical (Power) Auger Boring DRI – Double-Ring Infiltration Test		HA – Hand (Manual-Bucket) Auger Boring HCPT – Hand Cone Penetrometer Sounding ECPT – Electric Cone Penetrometer Sounding		

LABORATORY TESTING SERVICES

Sample and Field Log Review – After completion of the soil borings, field testing, and bulk soil sampling, laboratory services will consist of visual and tactile examination of soil samples that are returned from the field exploration, and preliminary classification of the soils, using the Unified Soils Classification System (USCS). Based on that examination, samples will be selected and laboratory soil index tests will be assigned, to confirm or modify the preliminary USCS classification, and to provide pertinent data, to estimate soil engineering parameters.

Sample Index Testing – At this time we expect that index testing may consist of the following tests:

Proposed Soil Index Test Schedule			
Item No.	Test Name	Test Specification Reference	No. of Tests
	Moisture Content of Soils Clay and Silt Fines Content of Soils Grain-Size Distribution of Soils (Coarse) Atterberg Limit Indices of Soils	ASTM D 2216 ASTM D 1140 ASTM D 421, D 422 ASTM D 4318	20 Tests 15 Tests 5 Tests 4 Tests

Bulk Soil Sample Testing

The bulk soil samples that will be collected during the drilling operations will be transported to our Tampa, Florida laboratory, where each sample will be subjected to the following series of tests.

Proposed Bulk Sample Test Schedule			
Item No.	Test Name	Test Specification Reference	No. of Tests
	Visual Classification Of Bulk Sample Moisture Content of Soils Modified Proctor Moisture-Density Determinations	None ASTM D 2974 ASTM D 1557	4 Tests 4 Tests 4 Tests

The actual testing may differ from the proposed schedule, consistent with the subsurface conditions that are discovered in our exploration, and the objectives of the study.

ENGINEERING AND REPORT SERVICES

The data gathered in the course of performance of the field and laboratory testing phases of this study will be evaluated by our engineering staff. The evaluations will include calculations and analyses that are directed at the following:

1. Evaluation of the property, with the goal of identifying areas that may be at risk to develop sinkhole subsidence. Assessment of the qualitative risk for future sinkhole subsidence, based on the hydrogeologic conditions on the property and the conditions revealed in the anomaly soil borings. Preparation of a strategy and recommendations to mitigate sinkhole conditions to prevent the development of sinkhole subsidence below the office buildings.
2. Determination of the feasibility of the use of shallow foundation systems to support the proposed office buildings. Development of foundation design and earthwork preparation recommendations to support the expected foundation loads. Estimation of the in-service performance of foundations that are designed and constructed as recommended.
3. If shallow foundations do not appear to be feasible, determination of feasible alternatives to improve load supporting capability of the soil and/or determination of feasible alternate foundation support systems.
4. Development of pavement design and earthwork preparation recommendations to support the expected vehicular traffic.

5. Development of utility trench design and earthwork recommendations.
6. Assessment of the potential utility and recommended limitations of soil that may be excavated during earthwork operations on the property, for incorporation into the project.
7. Estimation of the position of the seasonal high ground water level, in each soil boring that is performed in the proposed storm water management basin, if evident.

Based on the above assessments, we will prepare a final report which summarizes the collected information and which presents geotechnical related design and construction recommendations for the proposed project elements that are described herein. The report will also summarize our assessment of the risk presented by sinkhole conditions, and will present a strategy to mitigate these conditions, where they are found.

Project Staffing – Qualified drillers, engineering technicians and/or staff geologists will perform the field exploration. The geotechnical evaluation will be performed and the report prepared under the supervision of a licensed professional engineer, specializing in geotechnical engineering. We reserve the right to substitute testing and personnel, as necessary, in accordance with project needs, and consistent with staffing requirements. The proposed fee will not be exceeded without your prior written authorization.

STUDY LIMITATIONS

Conditions between exploration locations on this property may differ from those revealed at the selected locations. Please be aware that the opinions, conclusions, and recommendations made in this study will be based on our interpretation of the conditions revealed in the soil borings and other testing performed. We reserve the right to amend our opinions, conclusions, and recommendations, based on our evaluation of any post-exploration information that becomes available.

FEE FOR SERVICES

Based on the above described scope of work, we propose to perform this study for an estimated fee of **$34,525.00**. Our invoice will reflect the geophysical survey effort, the drilling, the laboratory testing, and personnel time expended, in accordance with HSA's current (2007) unit rates. Details of the estimated quantities and unit rates are summarized in Attachment A. We reserve the right to substitute or delete tests consistent with the objectives of the study and the proposed budget, as necessary, to complete the project. If site conditions appear to warrant additional exploration or engineering evaluation, we will notify you for approval before proceeding with services beyond the above-mentioned scope of services. Collected soil samples will be retained for 90 days and subsequently discarded.

SCHEDULE

The fieldwork can be initiated within one (1) to two (2) weeks after our receipt of your written approval of this proposal. It is expected that the GPR survey will require approximately two (2) weeks, to schedule and perform clearing, perform the field scans, process the data and identify the lateral extent of features of interest. Following completion of the GPR services, field exploration (drilling) will require approximately seven to eight (7 to 8) days to complete. Selected preliminary, verbal results would be available within one (1) week after completion of the fieldwork. We anticipate that our final report would be completed approximately three (3) weeks after the completion of the fieldwork. However, we will work with you by providing preliminary design information, to assist your time schedules, as needed.

ATTACHMENT B

Estimate of Services and Fees

Subsurface Exploration and Geotechnical Evaluation Services

Proposed Telecom Park Office Building Site

HSA Proposal No. 502-5358-98

I. SUBSURFACE EXPLORATION

	Quantity	Unit	Rate	Cost
1. Geophysical - Ground Penetrating Radar (GPR) Survey				
a. Transect Path Clearing - Contractor Charges	$800.00	Estim.	Cost + 5%	$840.00
b. Transect Path Clearing - HSA Direction	6	Hour	$65.00	$390.00
c. Phase I GPR Survey	1	LS	$3,400.00	$3,400.00
d. Phase II GPR Survey	1	LS	$2,000.00	$2,000.00
2. Drill Rig Mobilization (Men & Equipment)				
Limited Access/Trailer/Truck Mounted Drill Rig	1	Trip	$150.00	$150.00
3. Geotechnical Exploration Technician				
a. Layout Exploration Stations	4	Hour	$45.00	$180.00
b. Log Soil Borings	64	Hour	$45.00	$2,880.00
c. Perform HA/HCPT Borings/Soundings	10	Hour	$45.00	$450.00
d. Excavate DRI Test Pits - Contractor Charges	$800.00	Estim.	Cost + 5%	$840.00
e. Perform Double-Ring Infiltration Tests	6	Test	$500.00	$3,000.00
4. Standard Penetration Test (SPT) borings				
Truck Mounted Drill Rig (21 borings)				
Depth Interval (Feet)				
a. (0 to 50)	735	L.F.	$11.00	$8,085.00
b. (50 to 100)	60	L.F.	$13.00	$780.00
c. (100 to 150)	0	L.F.	$16.00	
Installation and Retrieval of Steel Bore Hole Casing				
Depth Interval (Feet)				
a. (0 to 50)		L.F.	$5.75	
b. (50 to 100)		L.F.	$6.75	
5. Excavate Bulk Soil Samples	4	Each	$50.00	$200.00
6. Seal Bore Holes (Per SWFWMD)				
a. Materials (Cement/Bentonite)	32	Sacks	$15.00	$480.00
b.Crew Time (Grout/Clean-Up)	11	Hours	$150.00	$1,650.00
SUBTOTAL SUBSURFACE EXPLORATION				**$25,325.00**

II. LABORATORY TESTING

	Quantity	Unit	Rate	Cost
1. Project Geologist/ Engineer (Visual Classification)	6	Hours	$80.00	$480.00
2. Laboratory Soil Tests				
a. Moisture Content (ASTM D 2216)	20	Each	$10.00	$200.00
b. Organic Content (ASTM D 2216)		Each	$30.00	
c. Soil Fines Content (ASTM D 1140)	15	Each	$30.00	$450.00
d. Grain Size-Coarse (ASTM D 421, D 422)	5	Each	$50.00	$250.00
e. Atterberg Limits (ASTM D 4318)	4	Each	$180.00	$720.00
f. Mod. Proctor Moisture-Density (ASTM D 1557)	4	Each	$85.00	$340.00

SUBTOTAL LABORATORY TESTING **$2,440.00**

III. PROFESSIONAL SERVICES

	Quantity	Unit	Rate	Cost
1. Senior Geotechnical Engineer (P.E.)	40	Hours	$150.00	$6,000.00
2. Technical Drafting	12	Hours	$50.00	$600.00
3. Secretarial/ Word Processing	4	Hours	$40.00	$160.00

SUBTOTAL PROFESSIONAL SERVICES **$6,760.00**

TOTAL FEE ESTIMATE **$34,525.00**

Please note that this total is for estimate purposes only. Please do not send a payment now. The invoice for the services will be sent with the report.

ACORD **Certificate of liability Insurance**

Issue Date: October 11, 2007

PRODUCER:

SAMPLE

Sample Professional Insurance Agency
1200 Main Street
Any Town, State 85016
FAX: (602) 555-1000 Phone: (602) 555-1100
Contact: Bill Johnson

THIS CERTIFICATE IS ISSUED AS A MATTER OF INFORMATION ONLY AND CONFERS NO RIGHTS UPON THE CERTIFICATE HOLDER. THIS CERTIFICATE DOES NOT AMEND, EXTEND OR ALTER THE COVERAGE AFFORDED BY THE POLICIES BELOW

INSURED:

Sample Architects & Engineers Group.
1500 Industrial Boulevard
Any Town, State 85016

COMPANIES AFFORDING COVERAGE	
COMPANY A	Dependable Casualty Insurance Company
COMPANY B	Dependable Indemnity Company
COMPANY C	Global Indemnity Company
COMPANY D	Professional Liability Underwriters Co
COMPANY E	

Coverages

THIS IS TO CERTIFY THAT THE POLICIES OF INSURANCE LISTED BELOW HAVE BEEN ISSUED TO THE INSURED NAMED ABOVE FOR THE POLICY PERIOD INDICATED, NOTWITHSTANDING ANY REQUIREMENT, TERM OR CONDITION OF ANY CONTRACT OR OTHER DOCUMENT WITH RESPECT TO WHICH THIS CERTIFICATE MAY BE ISSUED OR MAY PERTAIN, THE INSURANCE AFFORDED BY THE POLICIES DESCRIBED HEREIN IS SUBJECT TO ALL TERMS, EXCLUSIONS AND CONDITIONS OF SUCH POLICIES, LIMITS SHOWN MAY HAVE BEEN REDUCED BY PAID CLAIMS.

CO LTR	TYPE OF INSURANCE	POLICY NUMBER	Policy Effective Date	Policy Expiration Date	LIMITS	
A	**GENERAL LIABILITY** ☒Commercial General Liability ☐ Claims Made ☒ Occur. ☐ Owner's & Contractor's Prot ☒ General Agg - Per Project ☐	GL1234567/	4/01/00	4/01/01	GENERAL AGGREGATE	$ 1,000,000
					PRODUCTS-COMP OPS AGG.	$ 1,000,000
					PERSONAL & ADV. INJURY	$ 1,000,000
					EACH OCCURRENCE	$ 1,000,000
					FIRE DAMAGE (ANY ONE FIRE)	$ 50,000
					MED. EXPENSE (ANY ONE PERSON)	$ 5,000
B	**AUTOMOBILE LIABILITY** ☒ Any Auto ☒ All Owned Autos ☒ Scheduled Autos ☒ Hired Autos ☒ Non-Owned Autos ☐ Garage Liability	AL1234567	4/01/00	4/01/01	COMBINED SINGLE LIMIT	$ 1,000,000
					BODILY INJURY (PER PERSON)	$
					BODILY INJURY (PER ACCIDENT) PROPERTY DAMAGE	$
					Comprehensive Deductible	$
B	**EXCESS LIABILITY** Umbrella Form ☐ Other than Umbrella Form	UMX1234567	4/01/00	4/01/01	EACH OCCURRENCE	$ If required
					AGGREGATE	$ If required
C	**WORKER'S COMPENSATION AND EMPLOYERS' LIABILITY**	WC1234567	4/01/00	4/01/01	X WC Statutory Limits / Other	
					EL EACH ACCIDENT	$ 100,000
					EL DISEASE - POLICY LIMIT	$ 500,000
					EL DISEASE - EACH EMPLOYEE	$ 100,000
D	**OTHER** Professional Liability Retroactive Date:______________	PL 1234567	4/01/00	4/01/01	$2,000,000 Each Claim/$2,000,000 Aggregate (If architectural or structural services) ☐ $1,000,000 Each Claim/$1,000,000 Aggregate (If other design/professional services) ☐	

DESCRIPTION OF OPERATIONS/LOCATIONS/VEHICLES/SPECIAL ITEMS

Certificate applicable to all projects for which Insured is performing services as a Design Consultant, Architect or Engineer for Ryan Companies US, Inc. or its subsidiaries. Ryan Companies US, Inc. ("Ryan") and the Owner of any Project for whom Ryan is working shall be named as additional insureds on the above Commercial General Liability and Umbrella/Excess Liability policies. Professional Liability policy shall have a "retroactive date" not later than the date on which services are first performed.

CERTIFICATE HOLDER

Ryan Companies US, Inc.
50 South Tenth Street, Suite 300
Minneapolis, MN 55403
Attn: Sue Hayes

Fax #: (612) 492-3000

CANCELLATION

SHOULD ANY OF THE ABOVE DESCRIBED POLICIES BE CANCELLED BEFORE THE EXPIRATION DATE THEREOF, THE ISSUING COMPANY WILL ~~ENDEAVOR TO~~ MAIL 30 DAYS WRITTEN NOTICE TO THE CERTIFICATE HOLDER NAMED TO THE LEFT. ~~BUT FAILURE TO MAIL SUCH NOTICE SHALL IMPOSE NO OBLIGATION OR LIABILITY OF ANY KIND UPON THE COMPANY, ITS AGENTS OR REPRESENTATIVES.~~

AUTHORIZED REPRESENTATIVE

Signature