REPORT OF SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING SERVICES

PROPOSED MONOPOLE TOWER
BERKLEY GROUP HIDE-AWAY HILLS SITE
2119 MAYA LANE
SUGAR GROVE, OHIO

ECS PROJECT NO. 16:8597

FOR

AC&S ENGINEERING AND SURVEYING, INC.

AUGUST 8, 2011

August 8, 2011

Mrs. Terry Aldrich, P.E.
AC&S Engineering and Surveying, Inc.
3 Marcus Drive
Greenville, South Carolina 29605

ECS Project No. 16:8597

Reference: Report of Subsurface Exploration and Geotechnical Engineering Services, Proposed Monopole Tower, Berkley Group Hide-Away Hills Site, Sugar Grove, Ohio

Dear Mrs. Aldrich:

As authorized by your acceptance of our Proposal No. 16:9250-GP, dated July 11, 2011, ECS Midwest, LLC has completed the subsurface exploration for the proposed Berkley Group monopole tower site to be located at 2119 Maya Lane, in Sugar Grove, Hocking County, Ohio.

A report, including the results of the subsurface exploration, boring data, laboratory testing and engineering recommendations are enclosed herein. The recommendations presented are intended for use by your office and for use by other professionals involved in the design and construction of the project described herein.

We appreciate this opportunity to be of service to AC&S Engineering and Surveying, Inc. during the design phase of this project. If you have any questions with regard to the information and recommendations presented in this report, or if we can be of further assistance to you in any way during the planning or construction of this project, please do not hesitate to contact us.

Respectfully,

ECS MIDWEST, LLC

Danilo A. Guevarra
Senior Project Engineer

Stephen J. Geiger, P.E.
Principal Engineer

STATE OF OHIO
STEPHEN JOHN GEIGER
E-64822
REGISTERED
PROFESSIONAL ENGINEER

8/8/2011

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REPORT

PROJECT

Subsurface Exploration and Geotechnical Engineering Services
Proposed Monopole Tower
Berkley Group Base Hide-Away Hills Site
2119 Maya Lane
Sugar Grove, Hocking County, Ohio

CLIENT

AC&S Engineering and Surveying, Inc.
3 Marcus Drive
Greenville, South Carolina 29605

SUBMITTED BY

ECS Midwest, LLC
1575 Barclay Boulevard
Buffalo Grove, Illinois 60089

PROJECT	#16:8597
DATE	August 8, 2011

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EXECUTIVE SUMMARY

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The proposed tower site is to be located near the intersection of Maya Lane and Blacktail Court at 2119 Maya Lane, Sugar Grove, Hocking County, Ohio The proposed 90 feet by 90 feet Berkley Group lease area is to be located on the east side of the subject property. The tower lease area is currently vacant covered with gravel and grass at the ground surface. The ground surface of the tower site slopes down to the east with exist site grades at approximately EL. +962 feet to EL. +957 feet.

Surficial material consisting of approximately 4 inches of topsoil was encountered at the boring location. Below the topsoil, natural, cohesive, hard, brown Silty CLAY (CL) with sand and gravel was encountered to a depth of approximately 3 feet below existing site grade. Below 3 feet, very stiff, brown Sandy CLAY (CL) with weathered sandstone was encountered to a depth of approximately 14 feet below existing site grade at which auger refusal due to apparent bedrock was encountered. To verify the bedrock condition, the boring was cored 10 feet into rock. The cores indicated the presence of brown and gray, weathered SANDSTONE to the termination depth of 24 feet below existing site grade. The percent recoveries obtained from rock coring ranged from 60 to 68 percent. The rock core runs were determined to have RQDs ranging from 32 to 45 percent. Unconfined compressive strength tests performed on two representative rock core samples at depths of approximately 18 feet and 22 feet below existing grade, indicated unconfined compressive strengths of approximately 1,745 pounds per square inch (psi) and 3,450 psi, respectively. During and upon completion of drilling, groundwater was encountered at a depth of about 22 feet below the existing ground surface or about EL. +938 feet.

Our findings indicate that a drilled pier (caisson) foundation may be used to support the proposed tower. Drilled piers bearing at least 14 feet below the existing ground surface or top of the weathered sandstone rock can be designed for a maximum net allowable soil bearing pressure of 20,000 psf. Drilled piers embedded at least 2 feet into competent rock can be designed for a maximum net allowable soil bearing pressure of 40,000 psf. The net allowable soil bearing pressure refers to that pressure which may be transmitted to the foundation bearing soils in excess of the final minimum surrounding overburden pressure. For straight drilled piers, uplift forces can be resisted by the tower dead load including the factored weight of the pier, and the side shear along the circumference of the pier (skin friction). In determining the weight of the pier, we recommend a minimum factor of safety of 1.25 be applied to the weight of the concrete in the pier. The compression forces can be also resisted by the side shear along the circumference of the pier and the end bearing capacity. We anticipate drilling operations will penetrate into the weathered sandstone encountered below 14 feet from the existing grade. We recommend that the pier installer be prepared to case the pier excavations if the sidewalls are unstable, or if groundwater seepage is encountered in deep pier excavations. Appropriate rock augers and clean-out tools should also be on site during excavation to expedite construction and help reduce the potential for delays.

Our findings indicate that a mat foundation bearing directly in competent natural, very stiff, sandy clay with weathered sandstone encountered in our boring may be used to support the tower. Based on considerations of the general subsurface conditions indicated by the borings and the project characteristics, the actual uplift forces, the horizontal force and overturning moment will govern the design (depth and size) of the foundation. Based on our current findings, we assume that a mat foundation would bear at a minimum depth of 4 feet below the adjacent ground surface elevation. A mat foundation bearing directly in competent natural, very stiff, sandy clay with weathered sandstone can be designed for a maximum net allowable soil bearing pressure of 6,000 psf. The mat foundation may be designed using a modulus of subgrade reaction of 150 pounds per cubic inch (pci).

The ancillary structures may be supported by conventional shallow foundations and/or monolithic slabs-on-grade with foundation elements bearing in competent natural soils or on properly placed and compacted engineered fill. Shallow foundations bearing in competent natural very stiff to hard silty clay and sandy clay soils encountered in our boring or on properly compacted engineered fill can be designed for a maximum net allowable soil bearing pressure of 3,000 psf. Shallow foundations should be designed to bear at least 3 feet below the final exterior grades to provide adequate frost cover protection. Slabs-on-grades may be designed using a modulus of subgrade reaction of 150 pounds per cubic inch (pci).

Specific information regarding the subsurface exploration procedures used, the site and subsurface conditions at the time of our exploration, and our conclusions and recommendations concerning the geotechnical design and construction aspects of the project are discussed in detail in the subsequent sections of this report. Please note this Executive Summary is an important part of this report but should be considered a ***"summary"*** only and should not be relied upon exclusive of the entire report. The subsequent sections of this report constitute our findings, conclusions, and recommendations in their entirety.

Report Prepared By:

Danilo A. Guevarra
Senior Project Engineer

Report Reviewed By:

Stephen J. Geiger, P.E.
Principal Engineer

PROJECT OVERVIEW

Introduction

This report presents the results of our subsurface exploration and geotechnical engineering recommendations for the proposed monopole tower to be located at 2119 Maya Lane in St. Anne, Hocking County, Ohio. A General Location Map included in the Appendix of this report shows the approximate location of the project site. This study was conducted in general accordance with ECS Proposal No. 16:9250-GP, dated July 11, 2011, and authorized by your office.

Existing Site Conditions

The proposed tower site is to be located near the intersection of Maya Lane and Blacktail Court at Hide-Away Hills Club in Sugar Grove, Hocking County, Illinois. The common address of the site is 2119 Maya Lane, Sugar Grove, OH. Overall site relief is moderate, sloping downward to the east. The proposed 90 feet by 90 feet Berkley Group lease area is to be located on the east side of the subject property. The tower lease area is currently vacant covered with gravel and grass at the ground surface. The ground surface of the tower site slopes down to the east with exist site grades at approximately EL. +962 feet to EL. +957 feet.

Proposed Construction

Information regarding the proposed telecommunications tower was provided to us by AC&S Engineering & Surveying, Inc. According to the information provided, the structure will be a 190-foot monopole telecommunication tower. The provided preliminary loads for the proposed 190-foot high monopole consist of an overturning moment of approximately 1,806,934 foot-pounds and a shear load of about 16,038 pounds at the monopole base plate. The associated preliminary maximum axial load is about 42,657 pounds. We estimate the minimum diameter of the drilled piers required to permit anchor bolt installation will be approximately 8 feet and that the tower base plate will be set approximately 6 inches above the existing ground surface.

In addition, ancillary structures will likely be constructed adjacent to the tower. Details regarding the ancillary structures were also not provided; however, the tower equipment is expected to be relatively light. Based on our experience with similar construction, we anticipate that the ancillary structures will consist of pre-fabricated panel buildings with concrete slab-on-grade floor systems.

Purposes of Exploration and Scope of Services

The purposes of this exploration were to explore the soil and groundwater conditions at the site and to develop engineering recommendations to guide in the design and construction of the project. We accomplished these purposes by performing the following scope of services:

1. Drilling one boring to a depth of approximately 24 feet below the existing ground surface or approximately 10 feet into rock at the staked center of the planned tower to explore the subsurface soil and groundwater conditions;

2. Classifying samples of the recovered subsurface materials and reviewing the soil test boring and rock coring data to evaluate pertinent engineering properties; and

3. Analyzing the field and laboratory data to develop appropriate engineering recommendations.

The conclusions and recommendations contained in this report are based on the one (1) soil boring (Boring B-1) conducted at the project site under ECS' direction. The boring was performed in the immediate vicinity of the staked tower location. The boring was drilled to a depth of 24 feet below existing site grade. The subsurface exploration included split-spoon soil sampling, standard penetration tests (SPT), rock coring and groundwater level observations in the borehole. The results of the completed soil boring along with a Boring Location Plan are included in the Appendix of this report.

The Boring Location Plan was developed from the site plan provided by AC&S Engineering and Surveying, Inc. The boring was staked in the field by AC&S Engineering and Surveying, Inc. and the approximate location is shown on the Boring Location Plan. Ground surface elevation at the individual boring location was interpreted based on the provided site plan. Based on our review of the provided site plan, we anticipate the ground surface elevation at the center of the tower to be approximately EL. +960 feet.
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EXPLORATION PROCEDURES

Subsurface Exploration Procedures

The driller contacted the State of Ohio One-Call System, OUPS, to clear and mark underground utilities in the vicinity of the tower location prior to drilling operations. ECS contacted the property general manager, Mr. Randy Swetnam, to inform him of the tentative schedule of the drilling operations. The boring location was staked in the field by AC&S Engineering and Surveying, Inc. prior to drilling operations.

The soil boring was performed with a CME-75 truck-mounted rotary-type auger drill rig which utilized continuous hollow stem augers to advance the borehole. Representative soil samples were obtained by means of conventional split-barrel sampling procedures. In this procedure, a 2-inch O.D., split-barrel sampler is driven into the soil a distance of 18 inches by a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler through a 12-inch interval, after initial setting of 6 inches, is termed the Standard Penetration Test (SPT) or N-value and is indicated for each sample on the boring logs. The SPT value can be used as a qualitative indication of the in-place relative density of cohesionless soils. In a less reliable way, it also indicates the consistency of cohesive soils. This indication is qualitative, since many factors can significantly affect the standard penetration resistance value and prevent a direct correlation between drill crews, drill rigs, drilling procedures, and hammer-rod-sampler assemblies. The drill rig utilized an automatic trip hammer to drive the sampler. Consideration of the effect of the automatic hammer's efficiency was included in the interpretation of subsurface information for the analyses prepared for this report.

An NX core barrel with a diamond impregnated bit was used to extend the boring into the bedrock. Two 5-foot core run were obtained to verify rock conditions from a depth of approximately 14 to 24 feet below existing site grade.

A field log of the soils and rock encountered in the boring was maintained by the drill crew. After recovery, each geotechnical soil and rock samples was removed from the sampler and visually classified. Representative portions of each soil sample were then sealed in jars. The rock samples were placed in a core box. The soil and rock samples were then shipped to our laboratory in Buffalo Grove, Illinois for further visual examination and laboratory testing. After completion of the drilling operations, the borehole was backfilled with auger cuttings to the existing ground surface.

Laboratory Testing Program

Representative soil samples were selected and tested in our laboratory to check field classifications and to determine pertinent engineering properties. The laboratory testing program included visual classifications, calibrated hand penetrometer unconfined compressive strength testing and moisture content determinations.

Each soil sample was classified on the basis of texture and plasticity in accordance with the Unified Soil Classification System. The group symbols for each soil type are indicated in parentheses following the soil descriptions on the boring log. A brief explanation of the Unified System is included with this report. The various soil types were grouped into the major zones noted on the boring log. The stratification lines designating the interfaces between earth materials on the boring log are approximate; in situ, the transitions may be gradual.

The unconfined compressive strength of relatively cohesive clay soil samples was estimated with the use of a calibrated hand penetrometer. In the hand penetrometer test, the unconfined compressive strength of a soil sample is estimated, to a maximum of 4½ tons per square foot (tsf) by measuring the resistance of a soil sample to penetration of a small, calibrated spring-loaded cylinder.

The rock cores were logged for percent recovery and Rock Quality Designation (RQD). The RQD is the standard measurement of rock competence. RQD is determined by the ratio of the total length of cores, 4 inches or greater, to the total length of core run expressed in percentage. Core lengths smaller than 4 inches may be considered for RQD if they are judged to have been broken or fractured during coring and handling. Unconfined compressive strength tests were also performed on two representative intact rock core samples.

The soil and rock samples will be retained in our laboratory for a period of 60 days, after which, they will be discarded unless other instructions are received as to their disposition.

EXPLORATION RESULTS

Subsurface Conditions

The subsurface conditions encountered at the soil boring performed in the vicinity of the proposed tower location is summarized below. The specific soil and rock types observed at the boring is noted on the boring log, enclosed in the Appendix.

Surficial material consisting of approximately 4 inches of topsoil was encountered at the boring location. Below the topsoil, natural, cohesive, brown Silty CLAY (CL) with sand and gravel was encountered to a depth of approximately 3 feet below existing site grade. Below 3 feet, brown Sandy CLAY (CL) with weathered sandstone was encountered to a depth of approximately 14 feet below existing site grade at which auger refusal due to apparent bedrock was encountered. To verify the bedrock condition, the boring was cored 10 feet into rock. The cores indicated the presence of brown and gray, weathered SANDSTONE to the termination depth of 24 feet below existing site grade.

In general, the brown silty clay and sandy clay were observed to be very stiff to hard in consistency based on calibrated hand penetrometer unconfined compressive strength measurements ranging from 2½ tsf to 4½ tsf. The moisture contents of the clayey soils were determined to range from about 11 to 20 percent.

The percent recoveries obtained from rock coring ranged from 60 to 68 percent. The rock core runs were determined to have RQDs ranging from 32 to 45 percent. Unconfined compressive strength tests performed on two representative rock core samples at depths of approximately 18 feet and 22 feet below existing grade, indicated unconfined compressive strengths of approximately 1,745 pounds per square inch (psi) and 3,450 psi, respectively.

Groundwater Observations

Observations for groundwater were made during sampling and upon completion of the drilling operations at the boring location. During and upon completion of drilling, groundwater was encountered at a depth of about 22 feet below the existing ground surface or about EL. +938 feet. The highest groundwater observations are normally encountered in late winter and early spring and our current groundwater observations are not expected to be at the seasonal maximum water table. It should be noted that the groundwater level can vary based on precipitation, evaporation, surface run-off and other factors not immediately apparent at the time of this exploration. Surface water runoff will be a factor during general construction, and steps should be taken during construction to control surface water runoff and to remove any water that may accumulate in the proposed construction areas.

ANALYSIS AND RECOMMENDATIONS

The following recommendations have been developed on the basis of the previously described project characteristics and subsurface conditions encountered at the project site. If there are any changes to the project characteristics or if different subsurface conditions are encountered during construction, ECS Midwest, LLC should be consulted so that the recommendations of this report can be reviewed.

Tower Drilled Pier Foundations

Our findings indicate that a drilled pier (caisson) foundation may be used to support the proposed tower. Drilled piers bearing at least 14 feet below the existing ground surface or top of the weathered sandstone rock can be designed for a maximum net allowable soil bearing pressure of 20,000 psf. Drilled piers embedded at least 2 feet into competent rock can be designed for a maximum net allowable soil bearing pressure of 40,000 psf. The net allowable soil bearing pressure refers to that pressure which may be transmitted to the foundation bearing soils in excess of the final minimum surrounding overburden pressure.

According to the information provided, the preliminary loads for the proposed 190-foot high monopole consist of an overturning moment of approximately 1,806,934 foot-pounds and a shear load of about 16,038 pounds at the monopole base plate. The associated preliminary maximum axial load is about 42,657 pounds. We estimate the minimum diameter of the drilled pier required to permit anchor bolt installation will be approximately 8 feet.

To provide adequate resistance to the previously noted preliminary downward and lateral design loading conditions, our analyses indicate a drilled pier having a minimum diameter of 8 feet and extending at least 5 feet into the weathered sandstone to a depth of approximately 19 feet below the existing ground surface can be used to support the monopole tower. A drilled shaft bearing into weathered sandstone rock can be designed for a maximum net allowable soil bearing pressure of 40,000 psf. This design approach does not consider an evaluation of horizontal deflections; however, our evaluation indicates that for the shear and moment loads anticipated, horizontal deflection at the top of the drilled pier should be within tolerable limits. The estimated axial compression load of approximately 43 kips will exert a contact stress at the bottom of the drilled pier that is less than the net allowable bearing pressure. Therefore, the monopole drilled pier foundation settlement is anticipated to be negligible.

For straight drilled piers, uplift forces can be resisted by the tower dead load including the factored weight of the pier, and the side shear along the circumference of the pier (skin friction). In determining the weight of the pier, we recommend a minimum factor of safety of 1.25 be applied to the weight of the concrete in the pier. The compression forces can be also resisted by the side shear along the circumference of the pier and the end bearing capacity.

The following allowable side shear values, estimated undrained shear strength and estimated angle of internal friction may be used for design of the drilled pier foundation. The values were determined based upon the soil conditions encountered in Boring B-1 and were calculated for the mid-point between the referenced depths. The recommended allowable values for shear resistance were developed considering factors of safety of 2 for cohesive clayey soils and 4 for weathered sandstone rock. Submerged unit weights should be used for materials below the groundwater level.

Depth Below Ground (feet)	Average Side Shear Value (psf)		Estimated Undrained Strength (psf)	LPile Soil Modulus (k) (pci)	Estimated Of Internal Friction (degrees)	Estimated Unit Weight (pcf)
	Tension	Compression				
0 to 3	Neglect	Neglect	Neglect	Neglect	Neglect	Neglect
3 to 14	550	550	3,000	300	0°	120
14 to 24	3,700	3,700	1,700 psi*	Moderately Strong	0°	145

* Rock Unconfined Compressive Strength (psi)

The reinforcing steel and concrete strength requirements for the piers should be determined by the project structural engineer. The pier excavations should be observed by an ECS engineer, engineering geologist or experienced soil technician by visually examining the exposed sidewalls and the bottom of the pier excavation.

Drilled Pier Installation Considerations

We anticipate drilling operations will penetrate into the weathered sandstone encountered below 14 feet from the existing grade. Temporary steel casing may be needed if falling rock fragments hamper the drilling/coring operations. Therefore, we recommend that the pier installer be prepared to case the pier excavations if the sidewalls are unstable, or if groundwater seepage is encountered in deep pier excavations. If water infiltration cannot be effectively controlled by casing, then slurry construction methods may be required. To reduce the potential risk of accidental fall-in of foreign materials and personnel into the excavation hole we recommend that a temporary steel casing extend at least two feet above the prevailing ground surface elevation.

Appropriate rock augers and clean-out tools should also be on site during excavation to expedite construction and reduce the potential for delays. The drill rig utilized should be suitable for rock excavation and coring and should have a minimum torque of 100,000 ft-lbs. and the capacity to exert adequate downpressure to advance the drill tools. The contractor must be prepared for loss of circulating water and should have adequate sources and reserves available to complete construction without delay. Consideration can also be given to using air rotary methods to advance the excavation. Appropriate measures should be implemented to reduce the impact of construction vibrations to any nearby structures.

Once the bearing level is reached, all loose soil, rock and any accumulated water seepage should be removed prior to placement of pier reinforcing cage and concrete. Up to 1 inch of water standing in the base of the pier is acceptable at the time of concrete placement and an inflow rate of 1 inch per 5 minutes is also acceptable. Significant groundwater seepage is not anticipated unless the drilled pier excavations extend below a depth of 20 feet from the existing grade, which could likely be encountered, may require additional control or the drilled pier concrete be placed by tremie method.

One of the most critical aspects of installation of drilled piers is removal of the temporary casing. Specifically, concrete will have a tendency to "arch" within the casing lining, creating the possibility of voids or discontinuities within the shaft of the drilled pier. During concreting operations, we recommend that special attention be paid to the pour and pull operations, to help ascertain that discontinuities are not created within the shaft of the drilled pier. Temporary steel casing can be extracted as the concreting operation progresses. A positive head of concrete (at least 5 feet) should be maintained prior to pulling out the temporary steel casing to prevent any water and soil outside the steel casing from contaminating the concrete.

The drilled pier concrete should be placed in intimate contact with undisturbed natural soil and bedrock. To reduce the potential for arching, we recommend the drilled pier concrete mix be designed for a slump of 5 to 7 inches. Provided the water seepage is minimal once the temporary casing is placed and the excavation is dewatered, our experience and current research in the field indicates that the drilled piers can be constructed by "free fall" placement of concrete without affecting the strength and quality of concrete. The concrete should "free fall" without hitting the sides of the casing or reinforcing steel. The use of a hopper or other suitable device is recommended to control concrete placement and direct it towards the center of the pier. The placement of concrete in the cased pier shaft should proceed until the concrete level is above the external fluid level and should be maintained above this level throughout casing removal. However, if significant seepage is present within the excavation, it may be necessary to place the concrete by tremie method, and we recommend a concrete slump of 7 to 9 inches for this method of concrete placement.

The pier design and construction procedures should be reviewed with the foundation contractor prior to the start of construction. If you desire, we would be pleased to review the plans and specifications for the project once they are completed so we may have the opportunity to comment on the impact of the soil and groundwater conditions on the final design.

Tower Mat Foundation

Our findings indicate that a mat foundation bearing directly in competent natural, very stiff, sandy clay with weathered sandstone encountered in our boring may be used to support the tower. Based on considerations of the general subsurface conditions indicated by the borings and the project characteristics, the actual uplift forces, the horizontal force and overturning moment will govern the design (depth and size) of the foundation. Based on our current findings, we assume that a mat foundation would bear at a minimum depth of 4 feet below the adjacent ground surface elevation. A mat foundation bearing directly in competent natural, very stiff, sandy clay with weathered sandstone can be designed for a maximum net allowable soil bearing pressure of 6,000 psf. The net allowable soil bearing pressure refers to that pressure which may be transmitted to the foundation bearing soils in excess of the final minimum surrounding overburden pressure. The mat foundation may be designed using a modulus of subgrade reaction of 150 pounds per cubic inch (pci).

Resistance to lateral loads can be provided by friction between the bottom of the mat foundation and the underlying soils and by passive resistance of soil adjacent to the mat foundation. The passive resistance should only be used in situations where the soil adjacent to the mat will not be eroded or otherwise removed in the future. A coefficient of friction of 0.40 for concrete bearing on approved soils is recommended. An ultimate equivalent fluid pressure of 350 psf may be used to calculate passive earth pressures. These values are not factored and the foundation designer should apply the appropriate factor of safety. We recommend passive resistance should not be considered within 3 feet of the ground surface due to frost.

The final mat foundation bearing elevation should be evaluated by ECS personnel to verify that the bearing soils are capable of supporting the recommended net allowable bearing pressure and suitable for foundation construction. These evaluations should include visual observations, hand rod probing or hand auger probes with in-situ vane shear testing or dynamic cone penetrometer (ASTM STP-399) testing. Any unsuitable soil should be removed from beneath the planned mat foundations. Undercut excavations beneath the mat foundation should be backfilled with lean concrete or the foundation be extended below the soft/unsuitable soils and bear in competent natural soils.

Groundwater seepage is anticipated not to be a major factor during mat foundation excavation/construction. If water seepage from runoff and/or granular seams is encountered, we believe conventional sump and pump methods should be adequate to manage seepage and maintain dry excavation during the mat foundation short construction period. Exposure to the environment may weaken the soils at the footing bearing level if the foundation excavation remains open for too long a time. If the bearing soils are softened by surface water intrusion or exposure, the softened soils must be removed from the foundation excavation bottom immediately prior to placement of concrete. A mud mat or aggregate working mat may be necessary to stabilize the subgrade. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, we recommend that a 1- to 3-inch thick "mud mat" of "lean" concrete be placed on the bearing soils before the placement of reinforcing steel.

Ancillary Structure Foundations

Based upon our findings, the ancillary structures may be supported by conventional shallow foundations and/or monolithic slabs-on-grade with foundation elements bearing in competent natural soils or on properly placed and compacted engineered fill. Shallow foundations bearing in competent natural very stiff to hard silty clay and sandy clay soils encountered in our boring or on properly compacted engineered fill can be designed for a maximum net allowable soil bearing pressure of 3,000 psf. The net allowable soil bearing pressure refers to that pressure which may be transmitted to the foundation bearing soils in excess of the final minimum surrounding overburden pressure

Shallow foundations should be designed to bear at least 3 feet below the final exterior grades to provide adequate frost cover protection. Slabs-on-grades may be designed using a modulus of subgrade reaction of 150 pounds per cubic inch (pci). All slab and foundation subgrades should be evaluated immediately prior to concrete placement by ECS to verify that the exposed subgrades are capable of satisfactorily supporting the design loads.

Subgrade Preparation

We recommend that all vegetation, topsoil, trees, roots and soft or unsuitable materials be removed from areas to receive engineered fills, foundations, slabs or access roadways. The stripped surface should be proofrolled and carefully observed by ECS at the time of construction to aid in identifying localized soft or unsuitable materials that should be removed. Any unsuitable soil should be removed from beneath areas to receive engineered fills, foundations, slabs or access roadways. Undercut areas should be backfilled with engineered fill or otherwise be satisfactorily repaired in-place. As discussed in the **Tower Mat Foundation** section of this report, undercut excavations beneath the mat foundation should be backfilled with lean concrete or the foundation be extended below the soft/unsuitable soils and bear in competent natural soils.

General Construction Considerations

Any new fill placed to raise grades beneath shallow foundations to support ancillary structures, slabs or pavements should consist of engineered fill. Engineered fill should be compacted to a minimum of 95% of the maximum dry density determined in accordance with ASTM D 698, Standard Proctor Method. The materials should be placed in lifts not exceeding 8 inches in loose lift thickness; moisture conditioned to within 3 percent of the soils optimum moisture content, and then compacted as necessary to achieve the required minimum densities. Unsuitable fill materials include topsoil, organic materials (OH, OL), and high plasticity silts or clays (MH, CH). All such materials removed during grading operations should be either stockpiled for later use in landscape fills, or placed in approved on or off-site disposal areas.

All fills should consist of an approved material, free of organic matter, debris and particles greater than 3-inches. The majority of the silty clay and sandy clay soils sampled in the test boring should be suitable for use as engineered fill, although moisture conditioning of the clayey soils may be needed prior to their use as engineered fill. Materials imported from off-site for use as engineered fill should have a minimum standard Proctor (ASTM D 698) maximum dry density of at least 95 pounds per cubic foot (pcf), a liquid limit of not greater than 40, and a plasticity index of less than 20. We recommend placement of engineered fill and compaction operations be observed by an ECS representative on a full-time basis to verify that proper compaction of the fill material is achieved.

Accumulated water or runoff water in foundation excavations or on exposed slab or pavement subgrades should be promptly removed. All loose or soft soils in the exposed subgrade should be repaired in-place or removed before placing any concrete, paving materials or engineered fill. The grading contractor shall make provisions to divert or remove collected surface water from the prepared subgrades.

All excavations should meet the requirements of the most current Occupational Safety and Health Administration (OSHA) 29 CFR Part 1926, “Occupational Safety and Health Standards-Excavations” and its appendices, as well as other applicable codes. The excavations should not only be in accordance with current OSHA excavation and trench safety standards but also with applicable local, state, and federal regulations. The contractor should shore, slope or bench the excavation sides when appropriate. In no case should excavations extend below the level of adjacent structures, utilities or pavements, unless underpinning or other adequate support is provided. Site safety is the sole responsibility of the contractor, who shall also be responsible for the means, methods and sequencing of construction operations.

CLOSING

We recommend that the construction activities be monitored by ECS to provide the necessary overview, to check the suitability of the subgrade conditions for supporting the foundations, and to verify that soils are consistent with the subsurface conditions encountered during the exploration.

This report has been prepared in order to aid in the evaluation of this property and to assist the client in the design of this project. The scope is limited to the specific project and locations described herein and our description of the project represents our understanding of the significant aspects relative to soil and foundation characteristics. In the event that any change in the nature or location of the proposed construction outlined in this report are planned, ECS should be informed so that the changes can be reviewed and the conclusions of this report modified or approved in writing by the geotechnical engineer. It is recommended that all construction operations associated with earthwork and foundations be observed by an experienced geotechnical engineer to provide information on which to base a decision as to whether the design requirements are fulfilled in the actual construction. If you wish, we would welcome the opportunity to provide field services for you during construction.

The analysis and recommendations submitted in this report are based upon the data obtained from the soil boring performed at the location indicated on the Boring Location Diagram and other information referenced in this report. In the performance of the subsurface exploration, specific information is obtained at specific locations at specific times. However, it is a well known fact that variations in soil conditions can exist and groundwater levels will vary over time. The nature and extent of variations may not become evident until the course of construction. If variations then appear evident, after performing on-site observations during the construction period and noting characteristics and variations, a re-evaluation of the recommendations for this report may be necessary.

The assessment of site environmental conditions for the presence of pollutants in the soil, rock, and groundwater of the site was beyond the scope of this exploration

In addition to geotechnical engineering services, ECS Midwest, LLC has the in-house capability to perform multiple additional services as this project moves forward. These services include the following:

- Environmental Consulting;
- Project Drawing and Specification Review; and,
- Construction Material Testing / Special Inspections

We would be pleased to provide these services for you. If you have any questions with regard to this information or need any further assistance during the design and construction of the project please feel free to contact us.

APPENDIX

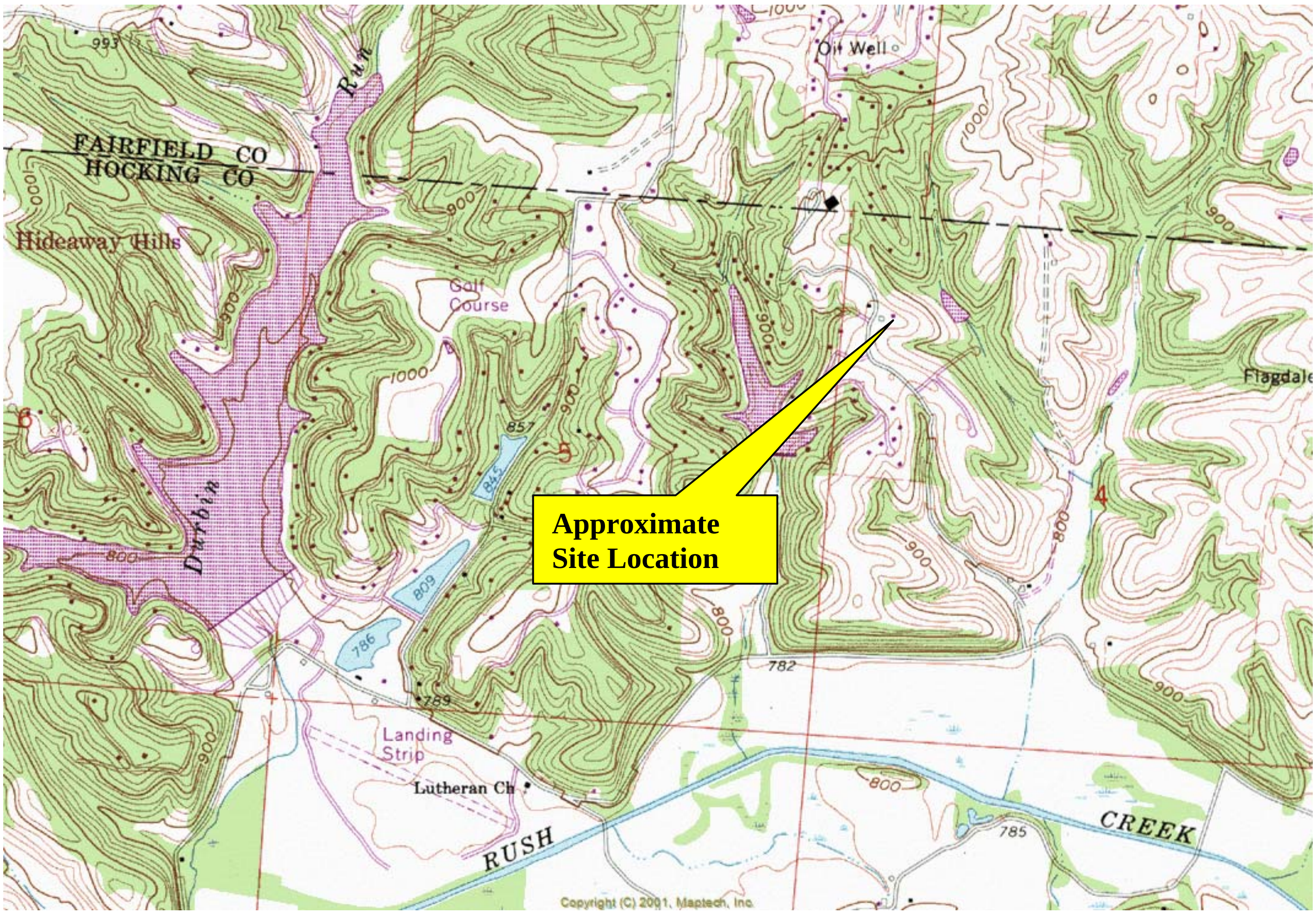
General Location Map

Boring Location Plan

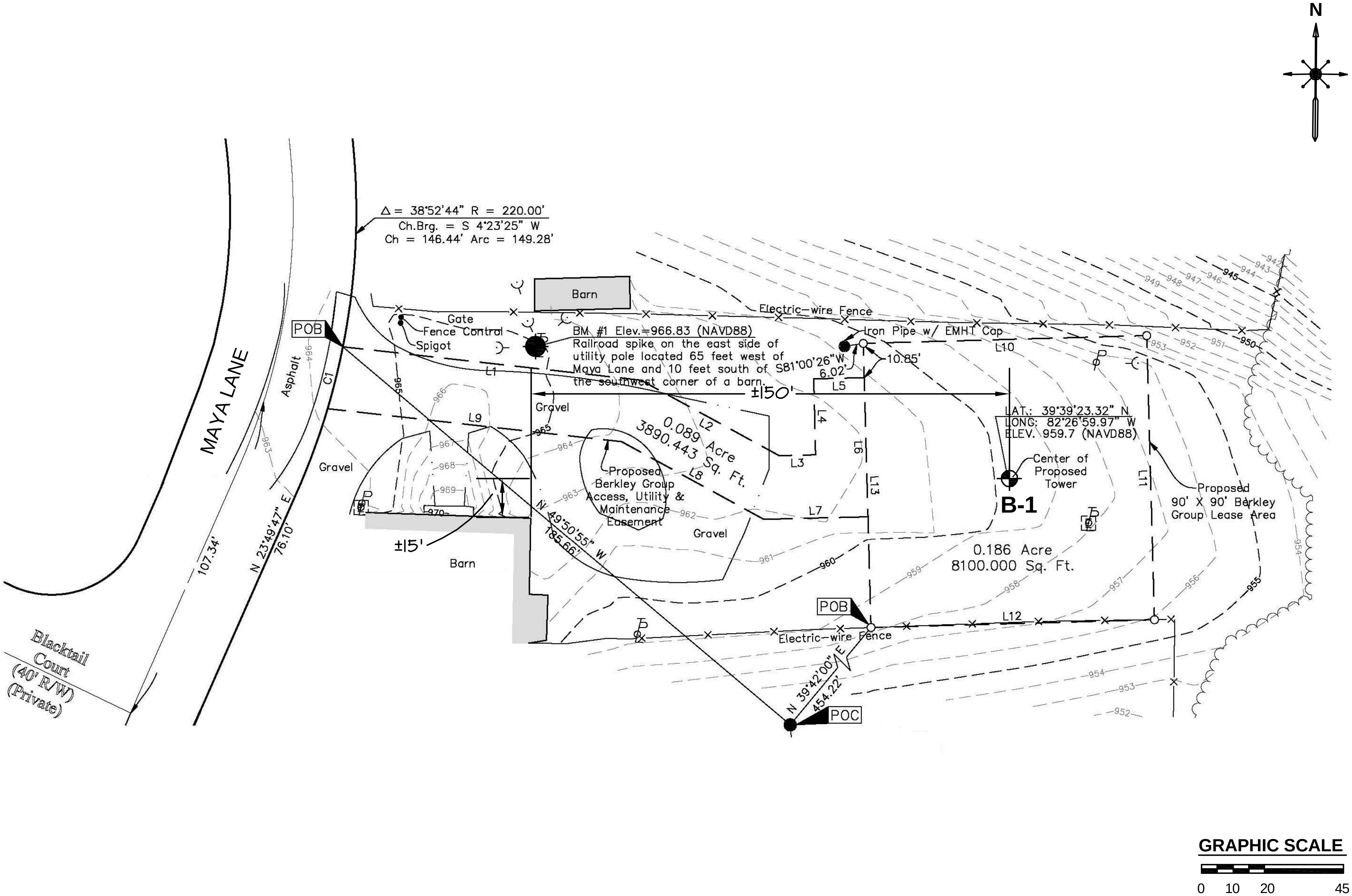
Boring Log

Unified Soil Classification System

Reference Notes For Boring Log



	GENERAL LOCATION MAP USGS Topographic Map Bremen, Ohio Quadrangle Dated 1961, Photorevised 1975 Scale: 1" = Approx. 1,800'	N	ECS Project No. 16:8597 Berkley Group Hide-Away Hills Monopole Tower Site 2119 Maya Lane, Sugar Grove Hocking County, Ohio



APPROX. SOIL BORING LOCATION

BORING LOCATION PLAN

Berkley Group – Hide Away Hills

AC&S Engineering and Surveying, Inc.

ENGINEER	SCALE
DAG	1"=45'
DRAFTING	PROJECT NO.
LGM	8597
REVISIONS	SHEET
	FIGURE 2
	DATE
	8/04/11

CLIENT	JOB #	BORING #	SHEET
AC&S Engineering and Surveying, Inc.	8597	B-1	1 OF 1

PROJECT NAME: Berkley Group - Hide Away Hills

ARCHITECT-ENGINEER: AC&S Engineering and Surveying, Inc.

SITE LOCATION: 2119 Maya Lane, Sugar Grove, OH

N39° 39' 23.32"/W82° 26' 59.97"

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION: 960

CALIBRATED PENETROMETER TONS/FT.^{2}: 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — — — REC.% ——— 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT.: 10 20 30 40 50+

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	SPT BLOWS/FT.	WATER CONTENT %	PENETROMETER TONS/FT.^{2}
0					Topsoil Depth 4"	960			
	1	SS	18	16	Silty CLAY, With Sand, Trace Gravel, Brown, Hard, (CL)		12	19.4	4.5
	2	SS	18	17	Sandy CLAY, With Weathered SANDSTONE, Brown, Very Stiff, (CL)	955	9	19.8	3.0
	3	SS	18	16			17	14.6	3.0
	4	SS	18	18		950	44	16.3	2.5
	5	SS	1	1			50/1	10.7	3.0
15	6	RC	60	41	Weathered Coarse Grained SANDSTONE, Brown, Rec = 68% RQD = 45%	945	Qu= 1,745 psi		
20	7	RC	60	36	Weathered Coarse to Fine Grained SANDSTONE, Brown and Gray, Rec = 60% RQD = 32%	940	Qu= 3,450 psi		
25					END OF BORING @ 24'	935			
30									

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

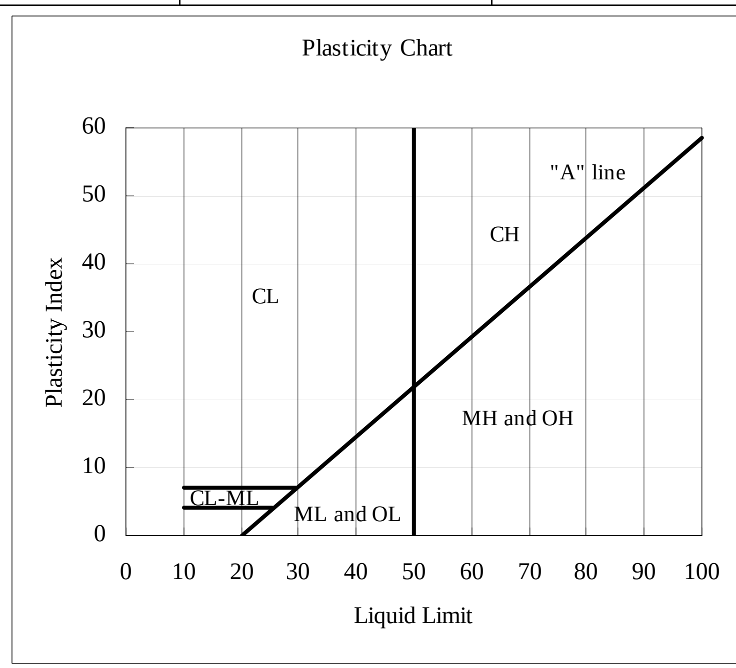
WL 22.3' — WS OR WD	BORING STARTED 07/26/11	DRILLER: Wrights Drilling, Inc.
WL(BCR) — WL(ACR) 22'	BORING COMPLETED 07/26/11	CAVE IN DEPTH @
WL	RIG CME 75 FOREMAN D. Wrights	DRILLING METHOD HSA

(08-04-11) (08-05-11) (08-08-11) (08-08-11) (08-08-11)

DGuevarra

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major Divisions			Group Symbols		Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 Sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent GW, GP, SW, SP; More than 12 percent GM, GC, SM, SC; 5 to 12 percent Borderline cases requiring dual symbols [b]	$C_u = D_{60}/D_{10}$ greater than 4 $C_c = (D_{30})^2/(D_{10} x D_{60})$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "A" line or P.I. less than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = D_{60}/D_{10}$ greater than 6 $C_c = (D_{30})^2/(D_{10} x D_{60})$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits above "A" line or P.I. less than 4	Limits plotting in CL-ML zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 Sieve)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	Plasticity Chart (see below)		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly Organic soils		Pt		Peat and other highly organic soils			



[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.
[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC,well-graded gravel-sand mixture with clay binder. (From Table 2.16 - Winterkorn and Fang, 1975)

REFERENCE NOTES FOR BORING LOGS

MATERIALS

Symbol	Material
	ASPHALT
	CONCRETE
	SUBBASE STONE / GRAVEL
	TOPSOIL
FILL	Man-placed or disturbed soils
GW	**WELL-GRADED GRAVEL** gravel-sand mixtures, little or no fines
GP	**POORLY-GRADED GRAVEL** gravel-sand mixtures, little or no fines
GM	**SILTY GRAVEL** gravel-sand-silt mixtures
GC	**CLAYEY GRAVEL** gravel-sand-clay mixtures
SW	**WELL-GRADED SAND** gravelly sand, little or no fines
SP	**POORLY-GRADED SAND** gravelly sand, little or no fines
SM	**SILTY SAND** sand-silt mixtures
SC	**CLAYEY SAND** sand-clay mixtures
ML	**SILT** non-plastic to medium plasticity
MH	**ELASTIC SILT** high plasticity
CL	**LEAN CLAY** low to medium plasticity
CH	**FAT CLAY** high plasticity
OL	**ORGANIC SILT or CLAY** non-plastic to low plasticity
OH	**ORGANIC SILT or CLAY** high plasticity
PT	**PEAT** highly organic soils
	WEATHERED ROCK
	IGNEOUS ROCK
	METAMORPHIC ROCK
	SEDIMENTARY ROCK

DRILLING SAMPLING SYMBOLS & ABBREVIATIONS

SS	Split Spoon Sampler	PM	Pressuremeter Test
ST	Shelby Tube Sampler	RD	Rock Bit Drilling
WS	Wash Sample	RC	Rock Core, NX, BX, AX
BS	Bulk Sample of Cuttings	REC	Rock Sample Recovery %
PA	Power Auger (no sample)	RQD	Rock Quality Designation
HSA	Hollow Stem Auger		

PARTICLE SIZE IDENTIFICATION

Designation		Particle Sizes
Boulders		12-inches (300-mm) or larger
Cobbles		3-inches to 12- inches (75-mm to 300-mm)
Gravel:	Coarse	¾-inch to 3-inches (19-mm to 75-mm)
	Fine	4.75-mm to 19-mm (No. 4 sieve to ¾-inch)
Sand:	Coarse	2.00-mm to 4.75-mm (No. 10 to No. 4 sieve)
	Medium	0.425-mm to 2.00-mm (No. 40 to No. 10 sieve)
	Fine	0.074-mm to 0.425-mm (No. 200 to No. 40 sieve)
Silt & Clay ("Fines")		<0.074-mm (smaller than a No. 200 sieve)

WATER LEVELS[1]

WL	Water Level (WS)(WD) (WS) While Sampling (WD) While Drilling
BCR	Before Casing Removal
ACR	After Casing Removal
WL	Water Level as stated
DCI	Dry Cave-In
WCI	Wet Cave-In

RELATIVE PROPORTIONS

Trace	<5%
Little	5% - <15%
With	15% - <30%
Adjective *(ex: "Silty")*	30% - <50%

COHESIVE SILTS & CLAYS

Unconfined Comp. Strength, Q_P[2] (tsf)	SPT[3] (bpf)	Consistency (cohesive only)
<0.25	≤2	Very Soft
0.25 - 0.49	3 - 4	Soft
0.50 - 0.99	5 - 8	Medium Stiff
1.00 - 1.99	9 - 15	Stiff
2.00 - 3.99	16 - 30	Very Stiff
4.00 - 8.00	31 - 50	Hard
>8.00	>50	Very Hard

GRAVELS, SANDS & NON-COHESIVE SILTS

SPT[3] (bpf)	Density
≤4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
51 - 99	Very Dense
≥100	Partially Weathered Rock to Intact Rock

[1]The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in granular soils. In clay and cohesive silts, the determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally taken.

[2]Typically estimated via pocket penetrometer or Torvane shear test and expressed in tons per square foot (tsf).

[3]Standard Penetration Test (SPT) refers to the number of hammer blows (blow count) of a 140 lb. hammer falling 30 inches on a 2-inch OD split-spoon sampler required to drive the sampler 12 inches (ASTM D 1586). "N-value" is another term for "blow count" and is expressed in blows per foot (bpf).

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