LAKE METROPARKS

11211 SPEAR ROAD
CONCORD TOWNSHIP, OHIO 44077

ADDENDUM NO. 1

Addendum Date: September 3, 2026

Project Name: Hemlock Ridge Park Bridge and Abutments Installation
Bid Advertised Date: August 19, 2026
Bid Opening Date: September 8, 2026
Bid Number: 2026-049

This addendum supplements and amends the original Bid Form and is hereby made a part of the Contract Drawings. Receipt of this Addendum must be noted on the "Bid Form".

ITEM 1: On Sheet 2 of 8 (S-101) in the drawings, note 1 should read Pedestrian bridge to be supplied by Lake Metroparks and installed by contractor as part of the base bid.

ITEM 2: Material storage will be on-site near the project location.

ITEM 3: Areté FRP Truss Bridge components will be unloaded and taken to a storage site near the project location by Lake Metroparks staff.

ITEM 4: Excavated materials will be disposed of on-site, exact location to be determined after award of contract.

ITEM 5: Lake Metroparks will remedy damage caused to the gravel access trail only (after construction is completed). Any damage beyond the gravel will be the contractor's responsibility.

ITEM 6: Geotechnical Report has been attached for review.

ITEM 7: (2) Control points have been marked within the vicinity of the bridge.

All questions concerning the Project and Addendum shall be referred to the Lake Metroparks Representative as listed in the Supplemental Instructions to Bidder.

GPD GROUP
Glaus, Pyle, Schomer, Burns & DeHaven, Inc.

GEOTECHNICAL ENGINEERING REPORT

HEMLOCK RIDGE PEDESTRIAN BRIDGE
LAKE COUNTY, OHIO

Prepared For:
Lake Metro Parks

GPD Project No. 2026415.01
May 29, 2026

STATE OF OHIO
JORDAN S. KIRKENDOLL
E-92610
REGISTERED
PROFESSIONAL ENGINEER
05/29/2026

Jordan Kirkendoll, PE
Project Manager

Contents

SECTION 1

1.0 Introduction

GPD Group is pleased to submit this Geotechnical Report for the aforementioned project. The purpose of this study was to obtain information on the subsurface conditions at the proposed project site and based on this information, to provide geotechnical recommendations regarding the design and construction of foundations for the proposed pedestrian bridge. A total of one (1) boring extending to a depth of approximately 7 feet below the existing surface was drilled at the site. Additionally, a dynamic cone penetrometer (wildcat) test was taken to a depth of about 3 feet below grade. A Location Plan and the boring log as well as the wildcat log is attached.

1.1 Project Description

A pedestrian bridge is proposed to be built on land west off of Vrooman Road and north of interstate I-90. This bridge will span a creek and connect a pedestrian hiking trail. No bridges or established trails have been constructed at the time of the geotechnical investigation.

Structural loading conditions for the bridge were not available for this writing; however, we anticipate maximum foundation loads will be the self-weights of structures and pedestrian loadings.

1.2 Purpose and Scope

The purposes of this report were to investigate subsurface conditions within the site and to provide geotechnical engineering recommendations for earthwork and foundation design. Specifically, the scope of work included the following:

- Conducting a field exploration program consisting of site reconnaissance and drilling a sample boring at a selected location within the proposed project location to explore subsurface conditions and collect soil samples.
- Conducting geotechnical engineering laboratory tests on sampled soils to assist with soil classifications and estimation of engineering properties.
- Develop geotechnical engineering recommendations for the design and construction of foundations and earthwork for site grading.

SECTION 2

2.0 Subsurface Exploration Program

The subsurface exploration consisted of drilling and sampling one (1) boring at the site to a depth of approximately 7.0 feet below existing grade. Additionally, a wildcat test was conducted on the east side of the creek. The boring locations were laid out by GPD personnel using a Reach RX Land Survey Unit. The location of the boring should be considered accurate only to the degree implied by the means and methods used to define them.

The boring was drilled with a track-mounted Acker rotary drill rig using hollow stem augers and an automatic SPT hammer to advance the borehole. Representative soil samples were obtained by the split-barrel sampling procedure in general accordance with the appropriate ASTM standards. In the split-barrel sampling procedure, the number of blows required to advance a standard 2-inch O.D. split-barrel sampler the last 12 inches of the typical total 18-inch penetration by means of a 140-pound hammer with a free fall of 30 inches, is the standard penetration resistance value (N-Value). This value is used to estimate the in-situ relative density of cohesion-less soils and the consistency of cohesive soils. The sampling depths and penetration distance, plus the standard penetration resistance values, are shown on the boring log. The samples were sealed and returned to the laboratory for testing and classification.

The field log was prepared by the drill rig crew. This log included visual classifications of the materials encountered during drilling as well as the driller's interpretation of the subsurface conditions between samples. The final boring log included with this report represent an interpretation of the field log and include modifications based on observations made by a Geotechnical Engineer and the results of laboratory testing.

An automatic SPT hammer was used to advance the split-barrel sampler in the boring performed for this site. A significantly greater efficiency is achieved with the automatic hammer compared to the conventional safety hammer operated with a cathead and rope. This higher efficiency has an appreciable effect on the standard penetration resistance blow count (N) values. The effect of the automatic hammer's efficiency has been considered in the interpretation and analysis of the subsurface information for this report.

Dynamic cone testing (wildcat) was performed by advancing a cone having a 10 square centimeter projected end area into the subsoils using a controlled dynamic energy produced by the drop of a 35-pound hammer through a height of 15 inches. The data was recorded as the number of blows required to advance the cone through each succeeding 10 centimeters of penetration. The field cone penetration data was reduced by computer, with the results as shown on the attached Wildcat Dynamic Cone Logs. Soil samples are not obtained during dynamic cone penetration testing. Thus, identification of soil-type is not possible except for obtaining a subgrade sample from directly under the base stone. The logs show the relative density of the soil being penetrated for each 10-centimeter increment if the soil being penetrated were sand or silt; and the stiffness of the soil being penetrated if it was clay. The value shown on the log as "N'" for each test interval is the approximate equivalent standard penetration blow count for the soil being tested, i.e., the equivalent blows per foot required to advance a standard split spoon sampler into that soil using a 140-pound hammer, freely falling from a height of 30 inches. This correlation between the two types of testing becomes inaccurate under high penetration resistance conditions, thus, the equivalent blow count is not given where such conditions exist.

Table 1: Boring Locations

Boring Number	Latitude	Longitude
B-1	41.706914°	-81.183127°
WC-1	41.706934°	-81.182908°

2.1 Laboratory Testing

The samples were classified in the laboratory based on visual observation, texture, and plasticity. The descriptions of the soils indicated on the boring logs are in accordance with the enclosed General Notes and the Unified Soil Classification System. A brief description of this classification system is attached to this report.

Information from these tests was used in conjunction with field penetration test data to evaluate soil strength in-situ, volume change potential, and soil classification. Results of these tests are attached and provided on the boring logs.

2.2 Subsurface Conditions

Topsoil – It was found that a organic layer of topsoil was present to a depth of 12" below grade. Below the topsoil layer, severely weathered shale was found.

Bedrock –Beneath the topsoil, a severely weathered, brown & gray to gray shale was found. Blow counts for the shale began at 18 blows per foot which is indicative of a medium dense shale rock, and became a very dense shale at 3 feet below grade with a blow count of 50/4". Auger refusal was then reached at 7 feet below grade.

WC-1 –Cone testing at WC-1 showed that a similar profile was present on the east side of the creek. The wildcat found that very loose material was present to 1 foot below grade. A medium dense likely severely weathered shale bedrock was then reached which became a very dense shale at 3 feet below grade.

2.2.1 Groundwater Conditions

The boring was monitored while drilling and immediately after completion for the presence and level of groundwater. At this time, groundwater was not observed in the soil boring.

Fluctuations of the groundwater level can occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the borings were performed. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

SECTION 3

3.0 Engineering Recommendations

The following engineering recommendations are based on information provided to GPD Group regarding the design of the proposed bridge, the field and laboratory testing performed on the soil encountered at this site, and other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, GPD should be immediately notified so that further evaluation and supplemental recommendations can be provided.

3.1 Geotechnical Considerations

Based on the information obtained during the course of this study, the following geotechnical considerations should be taken into account during the planning, design, and construction phases of the project. These geotechnical considerations are provided as a summary of the primary issues we believe are associated with this site. This report must be read in its entirety for a full description of our geotechnical recommendations:

- Due to the proximity of the stream to the proposed foundations, groundwater should be expected as part of this project for excavations extending below the stream water level. Conventional groundwater control measures should be anticipated as well as the potential use of permanent drains to move the water from the excavations.

- It should be noted that during site visits, the stream bed was observed to be composed of shale bedrock.

- It was expressed that helical piles would be the preferred foundation type. Due to the presence of shallow bedrock which can't typically be penetrated with helical piles, this system would not be appropriate for this site. Shallow foundations excavated into the shale bedrock would be considered appropriate. The bedrock is composed of a frost susceptible severely weathered shale. Excavations should extend below the frost level, and also below any potential scour depth. Excavation of the bedrock will likely require the use of specialized equipment such as an excavator equipped with rock ripping teeth or a hoe ram.

- Prior to the placement of steel or concrete, the excavation should be inspected by a geotechnical engineer or their representative to ensure proper bearing material and frost depth was successfully reached. All foundations should bear on the shale bedrock below the frost layer which is anticipated at 42 inches below finished grade. Foundations may need to extend deeper if the potential scour depth controls.

- The exposed bedrock excavation should not be left uncovered overnight. In the event that pouring the foundations does not occur the day of the excavation, the shale be over-excavated and a 4 inch mud mat should be placed at the bottom of the excavation.

- Contingent upon proper site preparation and thorough evaluation of the foundation excavations, it is our opinion that the proposed bridge can be supported on shallow foundations.

The following report sections provide detailed recommendations regarding the geotechnical considerations presented above. In the event changes in the project design occur, GPD Group must review this report to determine if modifications to our recommendations are warranted.

3.2 Site Preparation

All organic-containing soils, and any soft or otherwise unsuitable materials should be removed from the pedestrian bridge foundation limits.

Proof-rolling with heavy construction equipment such as a loaded tandem axle dump truck (approximately 60,000-pound gross) is recommended to aid in locating unstable subgrade materials prior to placement of fills, and again prior to placement of base stone. Areas where proof-rolling is not possible should be evaluated by other means. Proof-rolling is also recommended in cut areas, and areas left near existing grade after rough grading is completed. Unstable materials located by proof rolling should be removed and replaced with suitable compacted fill material. Areas of loose sand should be densified with a smooth drum vibratory roller.

The native soils sampled in our boring are moisture-sensitive and could become unstable beneath repetitive construction traffic or after a period of wet weather. Areas of unsuitable soil identified during proofrolling or subsequent construction operations will need to be stabilized. In the event that subgrade stabilization is required, we recommend select undercutting and backfilling with crushed stone as follows:

Crushed Stone - The use of crushed stone or gravel could be used to improve subgrade stability. The thickness and type of crushed stone will depend upon the conditions encountered and the location of the area to be improved. GPD's on-site Quality Control representative will provide this information as needed. Typical undercut depths would range from 1/2 foot to 1-1/2 feet below finished subgrade elevation. The use of high modulus geotextiles (i.e., geogrid) could also be considered. Equipment should not be operated above the geogrid until one full lift of crushed stone fill is placed above it. The maximum particle size of granular material placed over geotextile fabric or geogrid should not exceed 1-1/2 inches.

3.3 Fill Material

Any fill or backfill required within the bridge limits should be select material, as approved by a qualified geotechnical engineer. For all filling operations, the following should be observed:

- Prior to use, the approved fill material should be tested as outlined in ASTM D-698 to determine the maximum dry density and optimum moisture content for silty or cohesive soils, or ASTM D-4253 and D-4254 for clean granular soils. For each change in borrow material, additional tests will be required.

- For all fill or backfill used, the fill material should be placed on the approved subgrade in controlled lifts, with each lift compacted to a stable condition, and to a minimum of 98% maximum dry density per ASTM D-698 at a moisture content within 1.5% of optimum for cohesive or silty borrow. Controlled lifts of granular material should be compacted to 80% relative density per ASTM D-4254.

- All filling operations should be observed by a qualified soils technician with field density tests made, to assure compaction to specification.

Proper moisture control of fine grained silty soils is critical in attaining the required compaction. It should be noted that both in-situ soils and new fill composed of fine grained soils are susceptible to disturbance by construction equipment traffic when wet. Thus, construction operations should be planned to prevent such disturbance and the resulting weakening of the subgrade soils. Such precautions would include, but not be limited to grading the site to prevent ponding of water, sealing the subgrade soils at the end of operations each day, and allowing wet subgrades to dry before operating heavy equipment on the soil.

Compaction equipment and techniques will be dependent on the type of material being used as fill. A sheepsfoot roller should provide adequate compaction for cohesive (clayey) soils. A vibratory type compactor such as a drum roller will be required for non-cohesive (sandy) soils. It is our opinion that a sheepsfoot roller would provide the most optimal compaction results for the on-site soils.

3.4 Foundation Systems

Foundations comprised of conventional spread footings bearing on the shale bedrock suitable native soils may be sized using a maximum net allowable soil bearing pressure of **5,000 psf**. The recommended net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. Wall bearing footings should have a minimum width of 18 inches to preclude local shear failure. Footings should bear at least 3.5 feet below lowest adjacent finished grade for frost protection.

The foundation settlement will depend upon the variations within the subsurface soil profile, the structural loading conditions, the embedment depth of the footings, the thickness of compacted fill, and the quality of the earthwork operations. Provided that footing construction is performed in accordance with our recommendations, it is our opinion that total settlement will be about 0.5 inches or less. Differential settlement on the order of 2/3 to 3/4 of the total settlement should be anticipated.

The base of all foundation excavations should be free of water and loose soil prior to placing concrete. Concrete should be placed as soon as possible after excavating to minimize bearing soil disturbance. Should the soils at bearing level become excessively dry, saturated, disturbed, or otherwise altered; the affected soil should be removed prior to placing concrete. Given the shallow groundwater conditions associated with the creek encountered at this site, it may be desirable to stabilize the bottom of excavations with a relatively coarse and well-graded crushed stone or gravel, or a lean concrete mud mat.

Some water seepage into foundation excavations could occur. Any seepage into the anticipated excavations should be minor and dewatering, if required, should be possible with sump pits and pumps.

All footing excavations should be observed and tested by GPD Group personnel. Testing should include dynamic cone penetrometer tests and/or other testing deemed necessary by GPD Group. Where unsuitable bearing soils are encountered in the footing excavations, the excavations should be extended deeper to suitable soils where the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations.

3.5 Dewatering Recommendations

Shallow groundwater seepage could be encountered in the trench excavations. In the event groundwater is encountered during excavation, the contractor should dewater the excavation (discharge either on or off the construction right-of-way) in a manner that does not cause erosion and does not result in silt-laden water flowing into any body of water or other areas that could be negatively impacted. The Contractor shall submit an erosion and sediment control plan to the owner and GPD prior to construction in accordance with Federal, State and local codes.

The Contractor shall provide and maintain, at all times during construction, proper equipment necessary to promptly remove and dispose of all water entering excavations. Dewatering shall be performed by methods that will provide a dry excavation base and preserve the final lines and grades of the excavations. Dewatering methods may include the use of well points and/or sump pumps. The cost of all dewatering activities shall be borne by the Contractor.

3.6 Excavations

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person" as defined in "CFR Part 1926," should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

If the excavations are left open and exposed to the elements for a significant length of time, desiccation of the clays may create minute shrinkage cracks which could allow large pieces of clay to collapse or slide into the excavation. Materials removed from the excavation should not be stockpiled immediately adjacent to the excavation, as this load may cause a sudden collapse of the embankment.

We are providing this information solely as a service to our client. GPD is not assuming responsibility for construction site safety or the contractor's activities; No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others.

SECTION 4

4.0 Seismic Considerations

Based on the subsurface profile found in the test borings, a Seismic Site Classification "C" should be used for design of the structures according to the "International Building Code and Related Codes, Section 1613.5.2 Site Class Definitions.

4.1 Surface Drainage

Adequate drainage should be provided at the site to minimize any increase in moisture content of the pavement materials. All pavements should be sloped away from the structures to prevent ponding of water. Drains should be piped or channeled away from any slopes to minimize erosion and soil strength loss that could result in slope failures. Water should not be allowed to pond adjacent to the pavement edges.

4.2 Subsurface Drainage

Conventional dewatering methods, such as pumping from sumps, should be adequate for temporary removal of any groundwater encountered during excavation at the site. If springs or other significant groundwater is exposed above the stream during the excavation process, it may be necessary to install permanent trench drains to remove this water away from structures and pavements. The location and design of any trench drains should be determined at the time of construction, if warranted.

4.3 General Comments

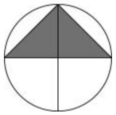
GPD Group should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. GPD should also be retained to provide testing and observation during site preparation and fill placement operations as well as during the foundation construction phases of the project.

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

The scope of services for this project does not include either specifically or by implication any environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the owner is concerned about the potential for such contamination, other studies should be undertaken.

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

LOCATION PLAN



Legend

Soil Boring Location:

PROJECT: LMP Hemlock Ridge Bridge

PROJECT NUMBER: 2026415.01 **DATE**: 05/18/2026

LOCATION: Painesville, Ohio

520 S Main St, Suite 2531 Akron, Ohio 44311

Boring Number: B-1

CLIENT Lake Metro Parks
PROJECT NAME Hemlock Ridge Bridge
PROJECT NUMBER 2026415.01
PROJECT LOCATION Painesville, Ohio
DATE STARTED April 24, 2026 **COMPLETED** April 24, 2026
GROUND ELEVATION 760.4 ft **HOLE SIZE**
DRILLING CONTRACTOR GPD Geotechnical Services, Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger - 2 1/4" ID
AT TIME OF DRILLING ---
LOGGED BY Dave Campana **CHECKED BY** Nathanael Channels
AT END OF DRILLING ---
NOTES Acker Recon

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		12 inches TOPSOIL.									
		Damp, medium dense, brown & gray, severely weathered SHALE.	SS 1	94	5-7-11 (18)						
2.5											
		Damp, very dense, gray, severely weathered SHALE.	SS 2	100	20-50/4"						
5.0											

Auger Refusal at 7.0 feet

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LOG NUMBER: WC-1

DATE: 4/24/26

WILDCAT DYNAMIC CONE LOG

PROJECT: LMP - Hemlock Ridge Bridge
LOCATION: Painesville, Ohio
WEATHER: Partly Cloudy, 80°F
REMARKS:

PROJECT NUMBER: 2026415.01
GROUND ELEVATION:
CREW: J.T. & D.C.

DEPTH FT	DEPTH M	BLOWS PER 10 CM	RESISTANCE KG/CM^2	CONE RESISTANCE 0 50 100 150	N'	TESTED DENSITY/CONSISTENCY NON-COHESIVE	COHESIVE
	0.1	1	4.4	*	1	VERY LOOSE	VERY SOFT
	0.2	1	4.4	*	1	VERY LOOSE	VERY SOFT
1	0.3	3	13.3	***	3	VERY LOOSE	SOFT
	0.4	10	44.4	*********	12	MEDIUM DENSE	STIFF
	0.5	8	35.5	*******	10	MEDIUM DENSE	STIFF
2	0.6	8	35.5	*******	10	MEDIUM DENSE	STIFF
	0.7	16	71.0	**************	20	MEDIUM DENSE	VERY STIFF
	0.8	20	88.8	******************	25	MEDIUM DENSE	VERY STIFF
3	0.9	60	266.4	*********************************	-	VERY DENSE	HARD

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LOG NUMBER:

DATE:

PROJECT: PROJECT:

DEPTH FT	M	BLOWS PER 10 CM	RESISTANCE KG/CM^2	CONE RESISTANCE 0 50 100 150	N'	TESTED DENSITY/CONSISTENCY NON-COHESIVE	COHESIVE
	5.6	0	0.0		0	VERY LOOSE	VERY SOFT
	5.7	0	0.0		0	VERY LOOSE	VERY SOFT
19	5.8	0	0.0		0	VERY LOOSE	VERY SOFT
	5.9	0	0.0		0	VERY LOOSE	VERY SOFT
	6.0	0	0.0		0	VERY LOOSE	VERY SOFT
20	6.1	0	0.0		0	VERY LOOSE	VERY SOFT
	6.2	0	0.0		0	VERY LOOSE	VERY SOFT
	6.3	0	0.0		0	VERY LOOSE	VERY SOFT
21	6.4	0	0.0		0	VERY LOOSE	VERY SOFT
	6.5	0	0.0		0	VERY LOOSE	VERY SOFT
	6.6	0	0.0		0	VERY LOOSE	VERY SOFT
22	6.7	0	0.0		0	VERY LOOSE	VERY SOFT
	6.8	0	0.0		0	VERY LOOSE	VERY SOFT
	6.9	0	0.0		0	VERY LOOSE	VERY SOFT
23	7.0	0	0.0		0	VERY LOOSE	VERY SOFT
	7.1	0	0.0		0	VERY LOOSE	VERY SOFT
	7.2	0	0.0		0	VERY LOOSE	VERY SOFT
24	7.3	0	0.0		0	VERY LOOSE	VERY SOFT
	7.4	0	0.0		0	VERY LOOSE	VERY SOFT
	7.5	0	0.0		0	VERY LOOSE	VERY SOFT
25	7.6	0	0.0		0	VERY LOOSE	VERY SOFT
	7.7	0	0.0		0	VERY LOOSE	VERY SOFT
	7.8	0	0.0		0	VERY LOOSE	VERY SOFT
26	7.9	0	0.0		0	VERY LOOSE	VERY SOFT
	8.0	0	0.0		0	VERY LOOSE	VERY SOFT
	8.1	0	0.0		0	VERY LOOSE	VERY SOFT
27	8.2	0	0.0		0	VERY LOOSE	VERY SOFT
	8.3	0	0.0		0	VERY LOOSE	VERY SOFT
	8.4	0	0.0		0	VERY LOOSE	VERY SOFT
28	8.5	0	0.0		0	VERY LOOSE	VERY SOFT
	8.6	0	0.0		0	VERY LOOSE	VERY SOFT
	8.7	0	0.0		0	VERY LOOSE	VERY SOFT
29	8.8	0	0.0		0	VERY LOOSE	VERY SOFT
	8.9	0	0.0		0	VERY LOOSE	VERY SOFT
	9.0	0	0.0		0	VERY LOOSE	VERY SOFT
30	9.1	0	0.0		0	VERY LOOSE	VERY SOFT
	9.2	0	0.0		0	VERY LOOSE	VERY SOFT
	9.3	0	0.0		0	VERY LOOSE	VERY SOFT
31	9.4	0	0.0		0	VERY LOOSE	VERY SOFT
	9.5	0	0.0		0	VERY LOOSE	VERY SOFT
	9.6	0	0.0		0	VERY LOOSE	VERY SOFT
	9.7	0	0.0		0	VERY LOOSE	VERY SOFT
32	9.8	0	0.0		0	VERY LOOSE	VERY SOFT
	9.9	0	0.0		0	VERY LOOSE	VERY SOFT
	10.0	0	0.0		0	VERY LOOSE	VERY SOFT
33	10.1	0	0.0		0	VERY LOOSE	VERY SOFT
	10.2	0	0.0		0	VERY LOOSE	VERY SOFT
	10.3	0	0.0		0	VERY LOOSE	VERY SOFT
34	10.4	0	0.0		0	VERY LOOSE	VERY SOFT
	10.5	0	0.0		0	VERY LOOSE	VERY SOFT
	10.6	0	0.0		0	VERY LOOSE	VERY SOFT
35	10.7	0	0.0		0	VERY LOOSE	VERY SOFT
	10.8	0	0.0		0	VERY LOOSE	VERY SOFT
	10.9	0	0.0		0	VERY LOOSE	VERY SOFT
36	11.0	0	0.0		0	VERY LOOSE	VERY SOFT
	11.1	0	0.0		0	VERY LOOSE	VERY SOFT
	11.2	0	0.0		0	VERY LOOSE	VERY SOFT
37	11.3	0	0.0		0	VERY LOOSE	VERY SOFT
	11.4	0	0.0		0	VERY LOOSE	VERY SOFT
	11.5	0	0.0		0	VERY LOOSE	VERY SOFT
38	11.6	0	0.0		0	VERY LOOSE	VERY SOFT
	11.7	0	0.0		0	VERY LOOSE	VERY SOFT
	11.8	0	0.0		0	VERY LOOSE	VERY SOFT
39	11.9	0	0.0		0	VERY LOOSE	VERY SOFT

Unified Soil Classification System

Major Divisions			Letter	Symbol	Description
Coarse-grained Soils More than ½ retained on the No. 200 Sieve	**Gravels** More than ½ coarse fraction retained on the No. 4 sieve	Clean Gravels	GW		Well-graded gravels and gravel-sand mixtures, little or no fines.
			GP		Poorly-graded gravels and gravel-sand mixtures, little or no fines.
		Gravels With Fines	GM		Silty gravels, gravel-sand-silt mixtures.
			GC		Clayey gravels, gravel-sand-clay mixtures.
	Sands More than ½ passing through the No. 200 sieve	Clean Sands	SW		Well-graded sands and gravelly sands, little or no fines.
			SP		Poorly-graded sands and gravelly sands, little or no fines.
		Sands With Fines	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sandy-clay mixtures.
Fine-grained Soils More than ½ passing through the No. 200 Sieve	**Silts and Clays** Liquid Limit less than 50%		ML		Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL		Organic clays of medium to high plasticity.
	Silts and Clays Liquid Limit greater than 50%		MH		Inorganic silts, micaceous or diatomaceous fines sands or silts, elastic silts.
			CH		Inorganic clays of high plasticity, fat clays.
			OH		Organic clays of medium to high plasticity.
Highly Organic Soils			PT		Peat, muck, and other highly organic soils.

Consistency Classification

Granular Soils

Description - Blows Per Foot (Corrected)

	MCS	SPT
Very loose	<5	<4
Loose	5 – 15	4 – 10
Medium dense	16 – 40	11 – 30
Dense	41 – 65	31 – 50
Very dense	>65	>50

Cohesive Soils

Description - Blows Per Foot (Corrected)

	MCS	SPT
Very soft	<3	<2
Soft	3 – 5	2 – 4
Firm	6 – 10	5 – 8
Stiff	11 – 20	9 – 15
Very Stiff	21 – 40	16 – 30
Hard	>40	>30

MCS = Modified California Sampler

SPT = Standard Penetration Test Sampler

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
ST: Shelby Tube - 3" O.D., except where noted.
BS: Bulk Sample
PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼,▽,▼ Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.