BRIDGE FOUNDATION INVESTIGATION REPORT

West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven, DeKalb County, Georgia

PREPARED FOR:
Heath and Lineback Engineers, Inc.
2390 Canton Road, Building 200
Marietta, Georgia 30066

NOVA Project No. 2022116

December 23, 2022

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December 23, 2022

HEATH AND LINEBACK ENGINEERS, INC.
2390 Canton Road, Building 200
Marietta, Georgia 30066

Attention: Brian K. Adams, P.E., S.E.
Vice President/Structures Division Director

Subject: Bridge Foundation Investigation Report
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven, DeKalb County, Georgia
NOVA Project No. 2022116

Dear Mr. Adams:

NOVA Engineering and Environmental, LLC (NOVA) has completed the referenced Bridge Foundation Investigation. The results of the Bridge Foundation Investigation with supporting documents are included with this letter.

We appreciate being part of the Heath and Lineback Engineer's team for this important project. If you have questions, please contact us.

Thank you.

Sincerely,
NOVA Engineering and Environmental, LLC

Naveen Thakur

Naveen S. Thakur
Senior Geotechnical Engineer

Eric K. Tay, P.E.
Senior Engineer
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Bridge Foundation Investigation
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven, DeKalb County, Georgia
NOVA Project No. 2022116
December 23, 2022

LOCATION The approximate location of West Nancy Creek Drive over Nancy Creek Tributary is indicated on the attached Figures 1 and 2.

GENERAL INFORMATION

GEOLOGIC FORMATION Button Micaceous Schist formation of Georgia Piedmont Region. See Figure 3.

SUBSURFACE FEATURES The subsurface information was obtained from two (2) soil test borings, B-1 and B-2 performed on August 15 and 16, 2022. Soil test boring depths ranged from 68 feet to 78 feet below the existing roadway pavement. Rock core samples were obtained in B-1 and B-2 upon auger refusal.

Fill materials described as silty SAND and/or sandy SILT were encountered in test borings B-1 and B-2 to an approximate depth of 13 feet (approximate elevation, EL. 855 feet). Standard Penetration Test (SPT) resistances ranged from 6 to 12 blows per foot (bpf).

Beneath the fill, alluvial soils described as silty SAND and/or sandy SILT were encountered to depths ranging from 18 to 23 feet (approximately from EL. 850 to 845 feet). SPT resistances ranged from 7 to 11 bpf.

Below the alluvial soils, residual soils described as silty SAND were encountered to depths ranging from 23 to 56.5 feet (approximate EL. 845 to 811.5 feet). In test boring B-2, a lens of Partially Weathered Rock (PWR) described as silty SAND was encountered within the residual soils between depths of 48 and 53 feet (approximately from EL. 820 to 815 feet).

PWR was encountered in test borings B-1 and B-2 to depths ranging from 28 to 58 feet (approximately from EL. 840 to EL. 810 feet). PWR is a transitional material between soil and underlying parent rock that is defined locally as materials that exhibit a SPT resistance exceeding 100 bpf.

Auger refusal was encountered in B-1 at an approximate depth of 28 feet (approximate EL. 840 feet) and in B-2 at an approximate depth of 58 feet (approximate EL. 810 feet). Auger refusal materials are very hard or very dense material, frequently boulders or the upper surface of the bedrock, which cannot be penetrated by a power auger.

Upon auger refusal, rock coring was performed in test borings. In boring B-1, rock core recoveries (REC) ranged from 3 to 94 percent, and Rock Quality Designation (RQD) ranged from 0 to 68 percent. In boring B-2, REC ranged from 97 to 100 percent, and RQD ranged from 53 to 93 percent.

Groundwater during drilling was observed in borings at an approximate depth of 13 feet (approximate EL. 855 feet).

For additional information see Figure 4 – Boring Location Plan (Appendix B) and attached Test Boring Records (Appendix C).

SITE CLASSIFICATION We recommend a Site Class of D per AASHTO LRFD 8th Edition (2017), 3.10.3.1.

1.0 – FOUNDATION RECOMMENDATIONS

Bents	Pile Bent (Type)
1 & 2	Micropile (Type A)

1.1 – MICROPILE PROPERTIES

Pile Type	Steel Reinforcement Casing Size (in)	Steel Reinforcement Casing Wall Thickness (in)	Bond Zone Diameter In Rock (in)
Micropile	9.625 (O.D.)	0.472	8 (O.D.)

Note: Steel reinforcement casing shall be permanently left in place to provide added pile reinforcement.

1.2 – DESIGN LOADS

Bents	Service Load (kips)
1 & 2	380

1.3 – MICROPILE FOUNDATION LOADS

Bents [1]	Bond Zone Diameter In Rock (in)	Downdrag (kips)	Ultimate Unit Grout-to-Ground Bond Strength (ksf)	Micropile Bonded Length In Rock (feet)	Ultimate Axial Capacity (kips)	Allowable Axial Capacity (kips)**
1 & 2	8	0	20[2]	19[3]	796	398[4]

Note [1]: At Bents 1 & 2, the bottom of micropile footing is estimated to be at elevation of 862 feet.
Note [2]: Micropile side resistance calculation was estimated based on ultimate grout-to-ground bond strength values from FHWA-NHI-05-039 (2005), Table 5-3, for Granite and Basalt (fresh-moderate fracturing, little to no weathering).

Note [3]: Micropile bonded length in rock was calculated based on micropiles embedded into competent rock. At Bent 1, side resistance in the upper 20 feet of rock (approximately from EL. 840 to 820 feet) is neglected due to poor quality rock encountered in test boring B-1. The top of competent rock at Bent 1 is at approximate EL. 820 feet and at Bent 2 is at approximate EL. 810 feet.

Note [4]: Allowable Axial Capacity was estimated based on a Factor of Safety of 2.

2.0 – FOUNDATION ELEVATIONS

Bents	Reference Borings	Bottom of Micropile (feet-NAVD 88)
1[1]	B-1	801 or below
2[2]	B-2	791 or below

Note [1]: At Bent 1, the estimated bottom elevation of reinforcement steel casing to be at 840 feet (top of rock).

Note [2]: At Bent 2, the estimated bottom elevation of reinforcement steel casing to be at 810 feet (top of rock).

3.0 – GENERAL NOTES

Elevations All elevations are based on Benchmark CD#G102 (PK Nail) located near Station 105+67, 13 feet Right at an elevation of 868.65 feet.

As Built Foundation Information The as built foundation information should be forwarded to the Geotechnical Engineering Bureau upon completion of the foundation system.

4.0 – MICROPILE FOUNDATION NOTES

Micropile Foundation End bents 1 and 2 will be supported by micropiles and constructed utilizing low overhead equipment due to overhead electrical power lines and neighborhood homes which may otherwise be subjected to vibrations due to pile driving operations. Micropile foundations shall be evaluated and designed by a specialty contractor consistent with GDOT Special Provision 999 – Micropile Foundations, AASHTO Standard Specifications for Highway Bridges (17th Edition), and FHWA-NHI-05-039 (2005). Micropile design is usually controlled by structural considerations. The micropile designer should evaluate the potential for buckling and lateral resistance in the micropile foundation system design.

At Bent 1, for all micropiles, a minimum 19-foot micropile bonded length into competent/sound rock from EL. 820 feet to EL. 801 feet for a minimum bond zone diameter of 8-inches will be required. Side resistance was neglected in the upper 20 feet of rock from EL. 840 feet to EL. 820 feet due to poor-quality rock encountered in test boring B-1.

Recommended soil profile for design of micropiles at Bent 1 (Boring B-1) is shown below:

Material	Approximate Elevation (ft.)	Uncorrected N-value (blows/ft.)	Total Unit Weight (pcf)
Loose to Medium Dense Fill	868 – 855	9 – 12	110
Medium Dense Alluvium	855 – 850	11	120
Dense Residuum	850 – 845	32	130
Partially Weathered Rock	845 – 840	100+	140
Poor Rock Quality (RQD 0 to 10%)	840 – 820	Auger Refusal	160
Competent/ Sound Rock	820 – 800	Auger Refusal	175

At Bent 2, for all micropiles, a minimum 19-foot micropile bonded length into competent/sound rock from EL. 810 feet to EL. 791 feet for a minimum bond zone diameter of 8-inches will be required.

Recommended soil profile for design of micropiles at Bent 2 (Boring B-2) is shown below:

Material	Approximate Elevation (ft.)	Uncorrected N-value (blows/ft.)	Total Unit Weight (pcf)
Loose to Medium Dense Fill	868 – 855	6 – 12	110
Loose Alluvium	855 – 845	7	110
Loose to Medium Dense Residuum	845 – 825	9 – 22	120
Medium Dense to Very Dense Residuum	825 – 811	29 – 54	140
Partially Weathered Rock	811 – 810	100+	140
Competent/ Sound Rock	810 – 790	Auger Refusal	175

Static Load Tests: Perform two (2) axial tension static load tests on non-production micropiles (one at Bent 1 and one at Bent 2) with a minimum applied load of 760 kips in accordance with Special Provision 999 – Micropile Foundations.

Proof Tests: Perform axial tension proof tests on two (2) micropiles (one at Bent 1 and one at Bent 2) or 5% of the total micropiles, whichever is greater. Proof testing should be loaded to the maximum design load of 380 kips in accordance with Special Provision 999 – Micropile Foundations.

Special Problems: Erratic micropile lengths are to be expected.

5.0 – QA / QC

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APPENDIX A

GDOT SPECIAL PROVISIONS

- GDOT Special Provision 999 – Micropile Foundations

January 19, 2018

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA

SPECIAL PROVISION

West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven, DeKalb County, Georgia

SECTION 999 – MICROPILE FOUNDATIONS

999.1 General Description

This work consists of furnishing all labor, materials, equipment, tools and other incidental items to design and construct micropile foundations and includes all incidentals and additional work in conjunction therewith.

999.1.01 Definitions

Admixture:	Substance added to the grout to either control bleed and/or shrinkage, improve flowability, reduce water content, retard setting time, or resist washout.
Casing:	Steel tube introduced during the drilling process in overburden soil to temporarily stabilize the drill hole. This is usually withdrawn as the pile is grouted, although in certain types of micropiles, some casing is permanently left in place to provide added pile reinforcement.
Centralizer:	A device to support and position the reinforcing steel in the drill hole and/or casing to provide a minimum grout cover.
Coupler:	The means by which the load can be transmitted from one partial length of reinforcement to another.
Bond Length:	The length of the micropile that is bonded to the ground and which is conceptually used to transfer the applied axial loads to the surrounding soil or rock.
Free Length:	The designed length of the micropile that is not bonded to the surrounding ground or grout during testing.
Micropile:	A small diameter, bored, cast-in-place pile, in which most of the applied load is resisted by the steel reinforcement.
Micropile Casing:	Steel pipe introduced during the drilling process to temporarily stabilize the drill hole and/or perform as a permanent structural component if left in place. Use as a permanent structural component is a designated assignment in the Plans or by the Engineer.

Micropile Type Class:	Micropiles are assigned a type class based on the method of installation. The method of installation is discussed in detail in the *Micropile Design and Construction Manual* (Sabatini, et al., 2005). Designer assumes a type class for designing the micropile. Final type class will be assigned by the Contractor.
Positive circulation/flush:	A method of progressing and cleaning out a hole for a micropile where drilling fluid is injected into the hole and returns upward along the outside of the drill casing.
Post-grouting:	The injection of additional grout into the bond length of a micropile after the Primary grout has set. Also known as re-grouting or secondary grouting.
Preloading:	The principle whereby load is applied to the micropile, prior to the micropile's connection to the structure, to minimize any structural movement in service.
Pressure grouting:	A method used to develop pile capacity wherein pressure is applied continuously to the top of the fluid grout column through the drill head as the casing is removed from the bond zone.
Primary Grout:	Portland cement based grout injected into the micropile hole prior to or after the installation of the reinforcement to provide the load transfer to the surrounding ground along the micropile and affords a degree of corrosion protection.
Production pile:	A pile which will be incorporated into the structure's foundation as a load-bearing element.
Proof Test:	Incremental loading of a micropile, recording the total movement at each increment.
Reinforcement:	The steel component of the micropile which accepts and/or resists applied loadings.
Spacer:	A device to separate elements of multiple-element reinforcement.
Static Pile Load Test:	A test to verify design assumptions and the adequacy of the contractor's installation methods.
Temporary Casing:	Steel pipe introduced during the drilling process to temporarily stabilize the drill hole.
Test Load (TL):	The maximum load to which the micropile is subjected during testing.
Tremie Grouting:	The placement of grout in a borehole via a grout pipe introduced to the bottom of the hole. During grouting, the exit of the pipe is kept at least 10 feet below the level of the grout in the hole.

999.1.02 Related References

A. Standard Specifications

General Provisions 101 through 150

Section 500 – Concrete Structures

Section 511 – Reinforcement Steel

Section 830 – Portland Cement

Section 831 – Admixtures

Section 880 – Water

B. Referenced Documents

AASHTO M31-10

AASHTO M85-09

AASHTO M275

ASTM A 252

ASTM D 1143 (D 1143M)

999.1.03 Submittals

A. Proof of Ability

Submit to the Engineer for review and approval the following proof of ability at least 30 days prior to beginning micropile construction:

- Documented qualifications of at least one Registered Professional Engineer licensed to perform work in the State of Georgia employed for the overall charge of the Work and a supervising Engineer for the Project with at least 5 years of experience in constructing micropiles.
- Documented qualifications of the Micropile Design Engineer as a Registered Professional Engineer licensed to perform work in the State of Georgia with working experience on a minimum of five projects designing and constructing micropiles.
- Evidence of successfully completing at least five projects similar in concept and scope to the proposed design. Include names, addresses and telephone numbers of the owner's representatives for verification.
- Résumés of foremen, superintendents, and drilling operators to be employed on this project. Show the type, length, and number of micropiles each has installed or tested within the past five years
- Evidence of experience in load testing. Persons performing load testing must list previous load testing projects within the past five years
- Documented qualifications of certified welder and a specialized welding plan for the micropile casing, if applicable.

The Department is the sole judge of the qualifications of the foreman, drilling operator, and testing personnel. Do not begin construction on foundations until the Engineer has approved the documented proof of ability.

B. Micropile Final Plan, Geotechnical Calculations, and Sequence of Construction

Submit Micropile Final Bridge Plans, Geotechnical Calculations, and Sequence of Construction to the Department for review and approval 90 working days before beginning construction on the foundations:

1. Final Plans will include shop drawings, detailed renderings, failure criteria for tensile load tests, failure criteria for proof testing, and/or any other representations necessary for accurate determination of micropile construction.
2. Calculations will include engineering assumptions and/or all data necessary for accurate determination of micropile construction specifications.
3. Sequence of construction will include a detailed sequence for micropile work describing all materials, methods and equipment to be used, including, but not limited to the following:

- List and sizes of proposed equipment including micropile drilling rigs and tools, tremies and grouting equipment.
- Detailed sequence of micropile construction and step-by-step description of micropile installation methods including details of casing installation, drilling methods and flushing.
- List of reinforcement and casings including grades or yield strengths and sizes.
- Methods for placing reinforcement with procedures for supporting and positioning the reinforcement including centralizers.
- Grout placement details including how the grout will be placed in the drill hole and ranges for grout pressure and volumes. Equipment and procedures for monitoring and recording grout levels, pressures and volumes with calibration certificates within one year of submittal date. Pressure grouting requirements are dictated by Micropile Type classification as outlined in *Micropile Design and Construction Manual* (Sabatini, et al., 2005). Check the classification to assess whether pressure grouting, staging, or specialized equipment is needed for the construction sequence. This information must be included for assessment for applicable submittals.
- Procedures for containment and disposal of drilling spoils, drill flush and excess waste grout.
- Grout mix design including laboratory test results, ranges for grout flow and density, any sand content, and any admixture to be used.
- Other information related to micropiles shown on the Plans or as requested by the Engineer.

Do not begin micropile foundation construction until the Plan, calculations, and sequence of construction have been approved in writing by the Engineer.

If alternate installation procedures are proposed or become necessary, provide a revised installation plan to the Engineer. If the work deviates from the accepted submittal the Engineer may suspend micropile construction until a revised plan is submitted and approved.

The time required for Plan, calculation, and sequence preparation and review will be charged to the allowable Contract time. The Department has 30 working days for Micropile Final Bridge Plan, Calculations, and Sequence of Construction review after receiving the complete submittal package at the Geotechnical Bureau.

New submittals from the Contractor showing corrections from the Department's review or changes to ease construction or to correct field errors have a 30-day review. The Department is the sole judge of information adequacy.

The Department's review and approval of the final Plan and construction methods does not relieve the Contractor from successfully completing the work. Time extensions are not granted for Contractor delays from untimely submissions and insufficient information.

C. Admixture Literature

Submit to the Engineer the manufacturer's literature, before using an admixture for review and approval. Indicate the admixture type and the manufacturer's recommendations for mixing the admixtures with grout.

D. Structural Steel

Submit to the Construction Project Manager the mill test reports for each heat or lot of prestressing material used to fabricate micropiles. Store this information as part of permanent recordkeeping for the project.

E. Calibration Data

Submit to the Engineer for review and approval calibration data for each test jack, pressure gauge and master pressure gauge. Provide calibration tests that have been performed by an independent testing laboratory within 180 days of the date of the submittal

The Engineer will approve or reject the calibration data within seven calendar days after receipt of the data. Do not begin testing until the Engineer has approved the jack, pressure gauge and master pressure gauge calibrations.

F. Reports

Submit micropile installation logs and drilling logs to the Construction Project Engineer not later than 48 hours after drilling for acceptance. If the submitted logs are not accepted by the Construction Project Engineer then the Department should be contacted.

Submit a report to the Engineer within 30 days after completion of the micropile work containing:

- As-built drawings showing the locations and lengths of the micropiles
- Detailed drilling records including depth to hard rock
- Grouting records indicating the cement type, and quantity injected
- Micropile test results and graphs

999.2 Materials

Ensure materials meet the requirements of the Specifications with the following exceptions:

A. Cement

Use Type I, II or III cement conforming to AASHTO M85-09 for the grout mixture. Do not add sand to the grout unless approved by the Engineer.

B. Admixtures

Admixtures to control bleed, improve flowability, reduce water content and retard set may be used in the grout subject to the approval of the Engineer. Use admixtures compatible with manufacturer's recommendation.

C. Water

Use potable water for mixing grout that meets the requirements of Specifications Section 880.

D. Micropile Steel Components

1. Reinforcing Casings

Use steel casings with the minimum wall thickness shown on the Plans and outside diameters ranging from the minimum diameter shown on the Plans to 3 inches (75 mm) larger. Provide casings meeting the tensile requirements of ASTM A252-98, Grade 3, except with a minimum elongation of 15% and minimum yield strength of 80 ksi (550 MPa) unless otherwise noted on the Plans.

2. Reinforcing Bars

   Use deformed steel bars meeting the requirements of AASHTO M31-10, Grade 60 or 75 (420 or 520) or M275.

E. Centralizers

Fabricate bar centralizers from schedule 40 polyvinyl chloride (PVC) plastic pipe or tube, steel, or other material not detrimental to steel reinforcement.

F. Grout

Produce cement grout using Portland cement conforming to AASHTO M85-09, Type I, II or III and potable water. Use fresh cement free of lumps and hydration. Do not use admixtures with chemicals that will be harmful to the reinforcing steel or cement. If approved by the Engineer, use admixtures that will impart low water content, flowability, and minimum bleeding in the cement grout.

999.2.01 Delivery, Storage, and Handling

A. Micropile steel components

Store steel reinforcement on blocking a minimum of 12 inches (300 mm) above the ground and protect the reinforcement at all times from damage.

999.3 Construction Requirements

999.3.01 Personnel

A. Contractor

Ensure personnel meet qualification requirements and have been approved by the Engineer in compliance with Subsection 999.1.03.A.

B. Micropile Design Engineer

Ensure Design Engineer meets qualifications requirements and has been approved by the Engineer in compliance with Subsection 999.1.03.A and is available at any time during the Contract to discuss the design of the micropiles with the Department.

999.3.02 Equipment

A. Drilling Rig

Use micropile drilling rigs capable of drilling through whatever materials are encountered to the dimensions and elevations required.

B. Grout Pump

Use a pump equipped with a pressure gauge to monitor grout pressures capable of measuring pressure of at least 150 psi (1035 kPa) or twice the actual grout pressures, whichever is greater.

999.3.03 Construction

A. Micropile Steel Components

When placing reinforcement in the drill hole, make sure the reinforcement is free from dirt, dust, loose mill scale, loose rust, paint, oil or other foreign materials.

B. Centralizers

Size centralizers to position reinforcement within 1 inch (25 mm) of the drill hole center and allow a tremie pipe to be inserted to the bottom of the drill hole. Use centralizers that do not interfere with grout placement or flow around the reinforcement.

C. Drilling and Reinforcement Installation

Install reinforcing casings to the tip elevations noted on the Plans.

Use a drilling method that results in a minimum clearance of 1 inch (25 mm) between the casing and soil or rock. Ensure the annulus between the casing and soil or rock is filled with grout.

Install micropiles to the location and inclination as specified in the Plans. Do not drill within 10 pile diameters, center to center, or 10 feet (3.0 m) whichever is greater, of any adjacent micropiles until the grout in all adjacent micropiles has been in place a minimum of 12 hours or has attained a minimum of 500 psi (3450 kPa) compressive strength.

Stabilize drill holes with casing from the beginning of drilling through grouting if unstable material is anticipated or encountered. After drilling, flush drill holes with water or air to remove drill cuttings and other loose material.

Use centralizers to center reinforcement bars in the drill hole. Securely attach bar centralizers at maximum 10 feet (3.0 m) intervals along the reinforcing bar. Attach upper and lowermost centralizers 5 feet (1.5m) from the top and bottom of micropiles.

Place reinforcing bars before the grouting operation. Do not vibrate or drive reinforcement. If reinforcement bars can only be partially inserted, redrill or clean drill hole to permit complete insertion.

D. Grouting

Use a neat cement grout, or a sand-cement grout, as approved in 999.1.03 "Submittals, with a minimum 28 day unconfined compressive strength of 3,500 psi (24100 kPa). Use cement free from lumps or other indications of hydration. Approved grout composition with sand content and admixtures as approved by the Engineer in the 999.1.03 section "Submittals" to be used on the project. If admixtures are used, mix in accordance with the manufacturer's recommendation.

Produce grout free of lumps and undispersed cement. Place the grout in one continuous operation. Use a mixer capable of continuously agitating the grout.

Inject grout from the lowest point of the drill hole. Record the quantity of the grout used on the project. Record the grout pressures measured, if required, due to Micropile Type Classification as outlined in *Micropile Design and Construction Manual* (Sabatini, et al., 2005). Control the grout pressures and grout takes to prevent heave. Fill the entire micropile with grout.

Upon completion of grouting, if the grout tube is to remain in the hole, fill the tube with grout. For load testing and proof testing, do not load the micropile intended for testing until the grout has reached a minimum cure break strength of 1750 psi (12050 kPa).

E. Meetings

A preconstruction meeting will be scheduled by the Construction Project Engineer a minimum 1 week prior to the load test. Discussion at the meeting will cover the construction sequence, delineation of responsibilities between the Prime Contractor and subcontractors on site, any necessary design considerations, and any additional information that the Department or the Contractor brings to the meeting about the construction and testing of the micropile foundations.

F. Load Testing

Furnish all labor, equipment and materials necessary to conduct an axial tensile test on 2 non-production piles per site and provide a written report to the Department. Monitor the installation of instrumentation and record all data using personnel experienced in this type of work or obtain the services of an experienced sub-contractor to perform this work. The report must contain site descriptions, calibration data, visual representation of the load test data, pile installation data, calibration reports for hydraulic jacks/load cells, applicable material certifications, comparison of the data with the failure criteria, and a statement clearly accepting or rejecting the load test data based on the failure criteria. Failure criteria will be formally established on this project based through the acceptance of 999.1.03 “Submittals” by the Geotechnical Environmental Pavement Bureau.

Perform all tests in accordance with ASTM D3689-07, “Standard Test Methods for Deep Foundations under Static Axial Tensile Load”, Paragraph 8.1.2 – Quick Pile Test.

Do not install production piles prior to the completion and approval of the test piles.

Load test results must be furnished to the Department for review and approval prior to beginning production piles. The Department has 14 working days from receipt of the results at the Geotechnical Bureau for review and approval. Approval for the load test report will be issued from the Department in writing.

If the load tests fail to meet the design requirements the Contractor will redesign, replace and test additional micropile(s) at no expense to the Department.

E. Proof Testing

Furnish all labor, equipment and materials necessary to conduct proof testing on 1 pile per substructure unit or 5% of the total piles on the project, whichever is the greater numerical value. Perform proof testing to the Plan required Maximum Foundation Load per Micropile. Proof testing is not to exceed the Maximum Foundation Load per Micropile for this test section. Perform this work on production piles and in accordance with ASTM D3689-07, “Standard Test Methods for Deep Foundations under Static Axial Tensile Load”, Paragraph 8.1.2 – Quick Pile Test.

Failure criteria will be formally established on this project based through the acceptance of 999.1.03 “Submittals” by the Geotechnical Environmental Pavement Bureau. Submit the proof test report to the Department within 2 calendar weeks. Provided the data in the proof test report in a manner consistent with the load test report, as stated in 999.3.03. D. “Load Test.” The Department has 7 working days for review and approval from when the proof test results are received at the Geotechnical Environmental Pavement Bureau. Approval for the proof test report will be issued from the Department in writing.

On projects with accepted load test results, proof testing may be reduced based on the load test showing higher than the theoretical calculated resistance factor and if there are no other mitigating geotechnical concern present on the project. Proof Tests will not be reduced for projects designed in karst environments or for projects designed without tip resistance in rock.

999.3.04 Quality Acceptance

Micropile acceptance is based on the following criteria:

- Micropile is within 3 inches (75 mm) of plan location and 2% of plumb or required inclination. Top of micropile is within 1 inch (25 mm) below and 3 inches (75 mm) above the top of micropile elevation shown on the Plans.
- Reinforcement is properly placed and inclination and top of reinforcement is within tolerances shown above for micropiles. Center of reinforcement is within 0.75 inch (19 mm) of center of the micropile. Tip of reinforcing casing is no higher than that noted in the Plans and casing penetrates into hard rock a minimum of 10 feet (3.0 m) or as noted on the Plans.
- Grout pressures, volumes, flow and densities are within acceptable ranges. Grout is in accordance with the contract and does not have any evidence of segregation, intrusions, contamination, structural damage or inadequate consolidation (honeycombing) and the Engineer verifies grout flow return around the reinforcing elements.

999.4 Measurement

1. **Micropile:** The length of accepted micropile foundation is measured in linear feet (meters) of micropile in place in the completed work. The length is measured from the final approved bottom elevation to 1 foot (305 mm) above the bottom of the footing cap where micropiles are used in a footing or to the top of the micropile elevation detailed in the Plans.
2. **Load Test:** Micropile load test is measured per each satisfactory load test on non-production micropiles, complete and accepted by the Engineer. No separate measurement will be made for unsatisfactory load tests on non-production micropiles, anchor piles, instrumentation, labor, equipment, materials, report preparation or any other incidentals needed to complete the work.
3. **Proof Test:** Micropile proof test is measured per each satisfactory load test on production micropiles, complete and accepted by the Engineer. No separate measurement will be made for unsatisfactory load tests on production micropiles, anchor piles, instrumentation, labor, equipment, materials, report preparation or any other incidentals needed to complete the work.

999.5 Payment

Micropile foundations are paid for at the unit price bid per linear feet (meters) complete and in place as specified. The payment is full compensation for all excavation, furnishing and placement of reinforcing steel and grout in the micropile, all temporary and/or permanent casing, disposal of excavated materials, and the cost of furnishing all tools, safety devices, labor, equipment and all other necessary items to complete the work.

Micropile load test is paid for at the unit bid price per each satisfactory load test on non-production micropiles, complete and accepted by the Engineer. No additional payment will be made for unsatisfactory load tests on non-production micropiles, anchor piles instrumentation, labor, equipment, materials, report preparation or any other incidentals needed to complete the work.

Micropile proof test is measured per each satisfactory load test on production micropiles, complete and accepted by the Engineer. No separate measurement will be made for unsatisfactory load tests on production micropiles, anchor piles, instrumentation, labor, equipment, materials, report preparation or any other incidentals needed to complete the work.

Payment will be made under:

Item No. 999	Micropile 8 in (mm) diameter	Per linear foot (meter)
Item No. 999	Load Test Micropile8 in (mm) diameter	Per each
Item No. 999	Proof Test Micropile8in (mm) diameter	Per each

APPENDIX B

FIGURES

- Figure 1 Site Location Map
- Figure 2 Topographic Map
- Figure 3 Regional Geologic Map
- Figure 4 Boring Location Plan
- Figures 5A through 5E - Rock Core Photographs

FIGURE 1
SITE LOCATION MAP
SOURCE: Google Earth Aerial Photos
SCALE: Not to scale

NOVA
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Heath & Lineback Engineers, Inc.
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven
Dekalb County, Georgia
NOVA Project No. 2022116

FIGURE 2 TOPOGRAPHIC MAP SOURCE: USGS National Map Advanced Viewer 7.5 Minute Topo, Chamblee, Georgia 2017 SCALE: Not to scale	NOVA PROFESSIONAL | PRACTICAL | PROVEN	Heath & Lineback Engineers, Inc. West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement City of Brookhaven Dekalb County, Georgia NOVA Project No. 2022116

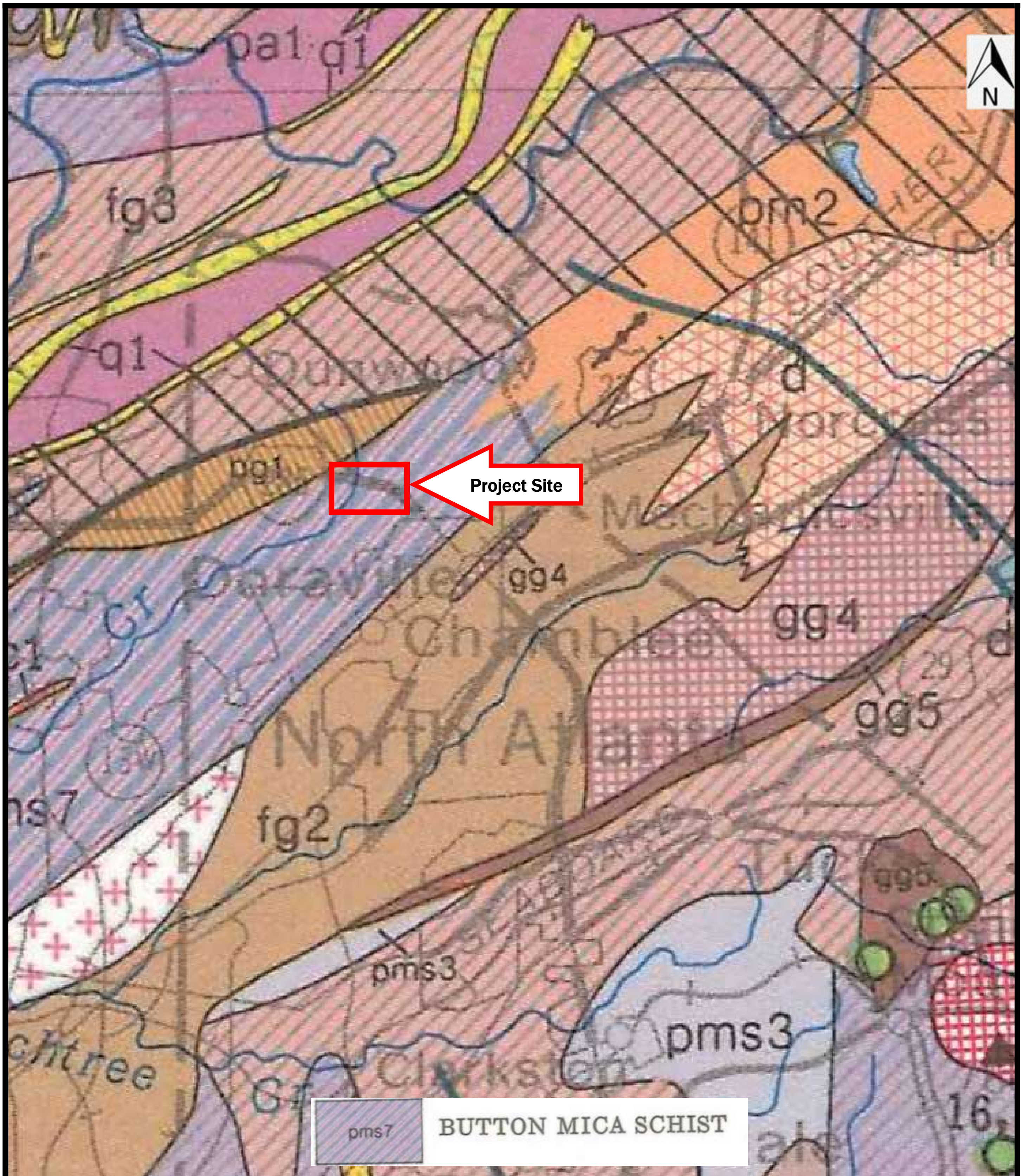

FIGURE 3
REGIONAL GEOLOGIC MAP
SOURCE: Geologic Map of Georgia, Georgia Department of Natural Resources, 1976
SCALE: Not to Scale

Heath & Lineback Engineers, Inc.
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven
Dekalb County, Georgia
NOVA Project No. 2022116

APPROXIMATE LOCATIONS OF NOVA SOIL TEST BORINGS. IT SHOULD BE NOTED THAT DUE TO THE EXISTING JERSEY BARRIERS, TEST BORINGS B-1 AND B-2 WERE DRILLED APPROXIMATELY 25 FEET WEST AND EAST OF THEIR RESPECTIVE BENTS 1 AND 2.

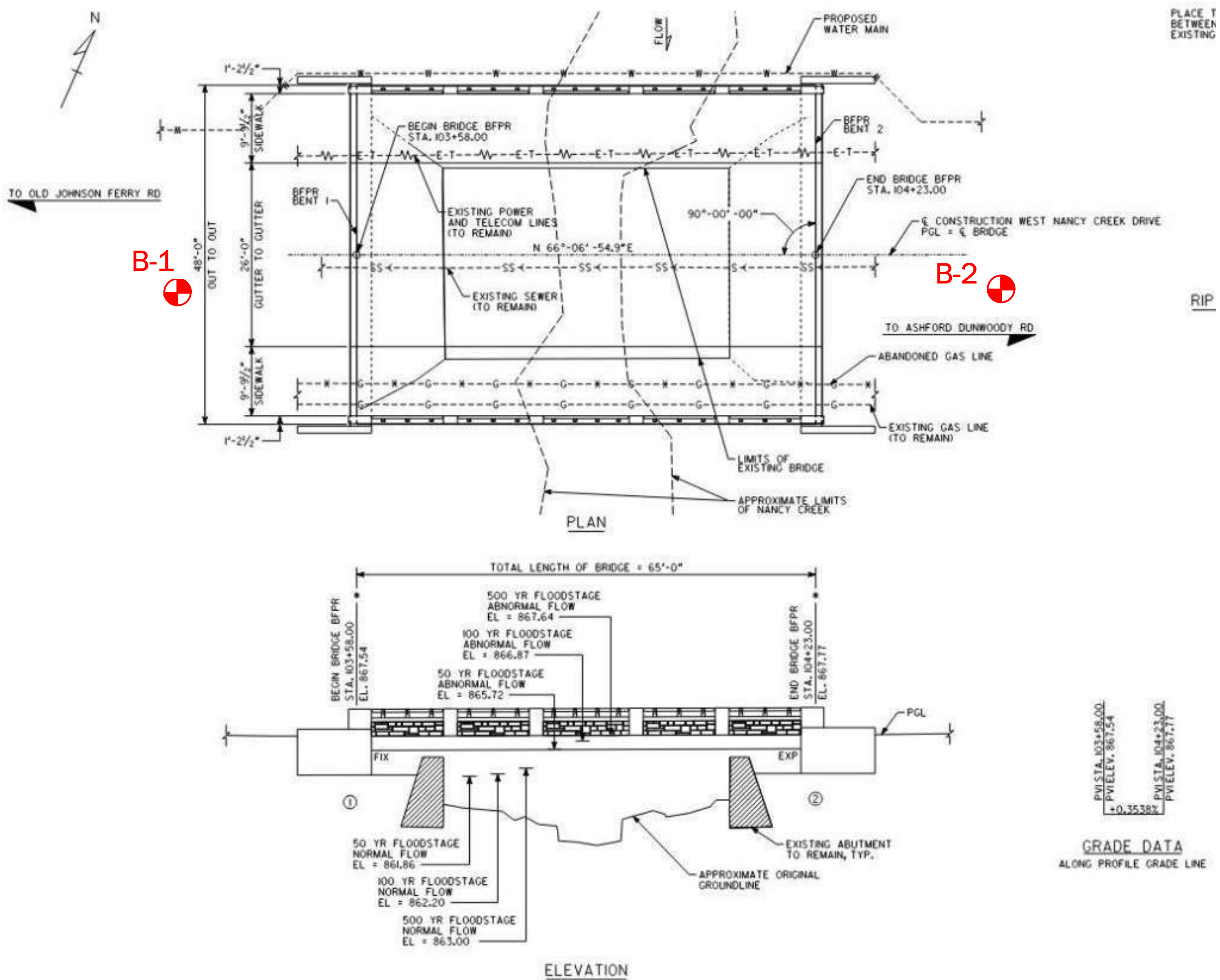

FIGURE 4
BORING LOCATION PLAN
PROVIDED BY: Heath and Lineback Engineers, Inc
DATE: November 2022
SCALE: Not to Scale

NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Heath and Lineback Engineers, Inc
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven
DeKalb County, Georgia
NOVA Project Number 2022116

B-1 (Core Run 1)
28 TO 38 FT BGS
EL. 839.6 TO EL. 829.6 FT NAVD88
REC: 12% RQD: 10%

B-1 (Core Run 2)
38 TO 48 FT BGS
EL. 829.6 TO EL. 819.6 FT NAVD88
REC: 3% RQD: 0%

28 FT BGS/EL. 839.6 FT NAVD88

48 FT BGS/EL. 819.6 FT NAVD88

FT BGS – FEET BELOW GROUND SURFACE
FT NAVD88- FEET NORTH AMERICAN VERTICAL DATUM OF 1988

ROCK CORE PHOTOGRAPH – FIGURE 5A
BORING B-1 (Core Run 1 & Core Run 2)
SOURCE: NOVA Engineering
SCALE: Not to Scale

NOVA
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Heath & Lineback Engineers, Inc.
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven
Dekalb County, Georgia
NOVA Project No. 2022116

B-1 (Core Run 3)
48 TO 58 FT BGS
EL. 819.6 TO EL. 809.6 FT NAVD88
REC: 55% RQD: 28%

48 FT BGS/EL. 819.6 FT NAVD88

58 FT BGS/EL. 809.6 FT NAVD88

FT BGS – FEET BELOW GROUND SURFACE
FT NAVD88- FEET NORTH AMERICAN VERTICAL DATUM OF 1988

ROCK CORE PHOTOGRAPH – FIGURE 5B
BORING B-1 (Core Run 3)
SOURCE: NOVA Engineering
SCALE: Not to Scale

NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Heath & Lineback Engineers, Inc.
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven
Dekalb County, Georgia
NOVA Project No. 2022116

B-1 (Core Run 4)
58 TO 68 FT BGS
EL. 809.6 TO EL. 799.6 FT NAVD88
REC: 94% RQD: 68%

58 FT BGS/EL. 809.6 FT NAVD88

68 FT BGS/EL. 799.6 FT NAVD88

FT BGS – FEET BELOW GROUND SURFACE
FT NAVD88- FEET NORTH AMERICAN VERTICAL DATUM OF 1988

ROCK CORE PHOTOGRAPH – FIGURE 5C
BORING B-1 (Core Run 4)
SOURCE: NOVA Engineering
SCALE: Not to Scale

NOVA
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Heath & Lineback Engineers, Inc.
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven
Dekalb County, Georgia
NOVA Project No. 2022116

B-2 (Core Run 1)
58 TO 68 FT BGS
EL. 809.8 TO EL. 799.8 FT NAVD88
REC: 97% RQD: 53%

58 FT BGS/EL. 809.8 FT NAVD88

68 FT BGS/EL. 799.8 FT NAVD88

FT BGS – FEET BELOW GROUND SURFACE
FT NAVD88- FEET NORTH AMERICAN VERTICAL DATUM OF 1988

ROCK CORE PHOTOGRAPH – FIGURE 5D
BORING B-2 (Core Run 1)
SOURCE: NOVA Engineering
SCALE: Not to Scale

NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Heath & Lineback Engineers, Inc.
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven
Dekalb County, Georgia
NOVA Project No. 2022116

B-2 (Core Run 2)
68 TO 78 FT BGS
EL. 799.8 TO EL. 789.8 FT NAVD88
REC: 100% RQD: 93%

68 FT BGS/EL. 799.8 FT NAVD88

78 FT BGS/EL. 789.8 FT NAVD88

FT BGS – FEET BELOW GROUND SURFACE
FT NAVD88- FEET NORTH AMERICAN VERTICAL DATUM OF 1988

ROCK CORE PHOTOGRAPH – FIGURE 5E
BORING B-2 (Core Run 2)
SOURCE: NOVA Engineering
SCALE: Not to Scale

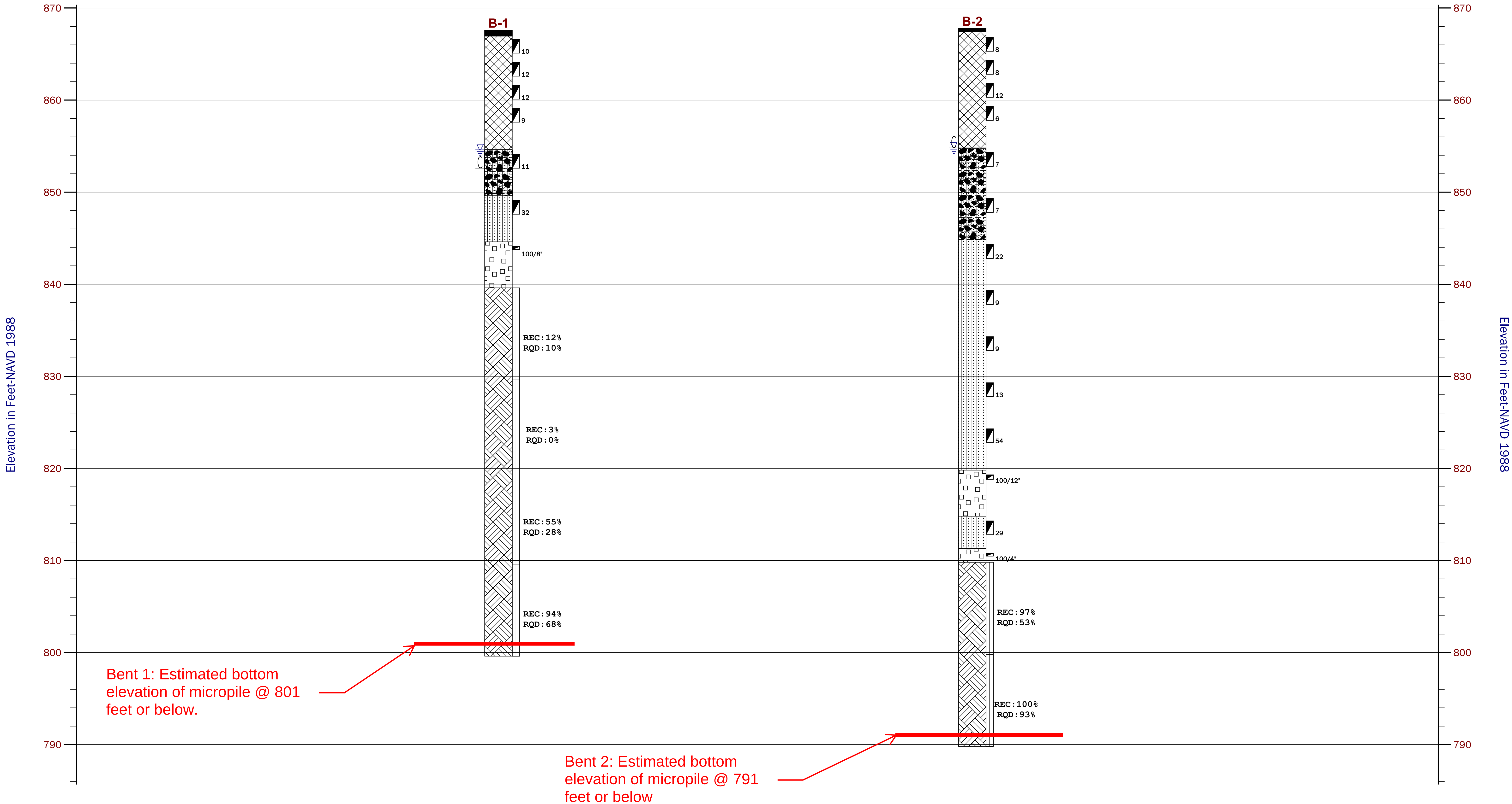
NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Heath & Lineback Engineers, Inc.
West Nancy Creek Drive over Nancy Creek Tributary – Bridge Replacement
City of Brookhaven
Dekalb County, Georgia
NOVA Project No. 2022116

APPENDIX C

TEST BORING RECORDS

- Subsurface Profile
- Test Boring Records Summary
- Key to Symbols and Classifications
- Test Boring Records



AR - Auger Refusal ; BT - Boring Termination

Paving
Fill
Alluvium
Silty sand
Partially Weathered Rock (PWR)
Rock
Water table during drilling
Depth to caving
Standard penetration test
Rock core

NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Heath & Lineback Engineers, Inc.
West Nancy Creek Drive over Nancy Creek Tributary - Bridge Replacement
City of Brookhaven, DeKalb County, Georgia
NOVA Project No: 2022116

BORING SUMMARY

Heath & Lineback Engineers, Inc.

West Nancy Creek Drive over Nancy Creek Tributary - Bridge Replacement

City of Brookhaven, DeKalb County, Georgia

NOVA Project Number 2022116

BORING	EXISTING BORING ELEVATION (FT)	GROUND WATER		FILL		ALLUVIUM		RESIDUUM		PARTIALLY WEATHERED ROCK		AUGER REFUSAL		ROCK CORES			
		DEPTH (FT)	ELEVATION (FT)	DEPTHS (FT)	ELEVATION (FT)	DEPTHS (FT)	ELEVATION (FT)	DEPTHS (FT)	ELEVATION (FT)	DEPTHS (FT)	ELEVATION (FT)	DEPTH (FT)	ELEVATION (FT)	DEPTHS (FT)	ELEVATION (FT)	RECOVERY (%)	RQD (%)
B-1	867.6	13.0	854.6	0.6 - 13.0	867.0 - 854.6	13.0 - 18.0	854.6 - 849.6	18.0 - 23.0	849.6 - 844.6	23.0 - 28.0	844.6 - 839.6	28.0	839.6	28.0 - 38.0	839.6 - 829.6	12	10
														38.0 - 48.0	829.6 - 819.6	3	0
														48.0 - 58.0	819.6 - 809.6	55	28
														58.0 - 68.0	809.6 - 799.6	94	68
B-2	867.8	13.0	854.8	0.4 - 13.0	867.4 - 854.8	13.0 - 23.0	854.8 - 844.8	23.0 - 56.5	844.8 - 811.3	56.5 - 58.0	811.3 - 809.8	58.0	809.8	58.0 - 68.0	809.8 - 799.8	97	53
														68.0 - 78.0	799.8 - 789.8	100	93

Elevations and Depths should be considered as approximate.

N/E: Not encountered

KEY TO SYMBOLS AND CLASSIFICATIONS

DRILLING SYMBOLS

◩	Split Spoon Sample
▯	Undisturbed Sample (UD)
●	Standard Penetration Resistance (ASTM D1586-67)
▼	Water Table at least 24 Hours after Drilling
▽	Water Table 1 Hour or less after Drilling
100/2”	Number of Blows (100) to Drive the Spoon a Number of Inches (2)
NX, NQ	Core Barrel Sizes: 2⅛- and 2-Inch Diameter Rock Core, Respectively
REC	Percentage of Rock Core Recovered
RQD	Rock Quality Designation – Percentage of Recovered Core Segments 4 or more Inches Long
▶	Loss of Drilling Water
MC	Moisture Content Test Performed
N/E	Not Encountered
N/M	Not Measured
C	Caving

CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

	Number of Blows, “N”	Approximate Relative Density
	0 – 4	Very Loose
	5 – 10	Loose
SANDS	11 – 30	Medium Dense
	31 – 50	Dense
	Over 50	Very Dense

	Number of Blows, “N”	Approximate Consistency
	0 – 2	Very Soft
	3 – 4	Soft
SILTS	5 – 8	Firm
and	9 – 15	Stiff
CLAYS	16 – 30	Very Stiff
	31 – 50	Hard
	Over 50	Very Hard

DRILLING PROCEDURES

Soil sampling and standard penetration testing performed in accordance with ASTM D1586-67. The standard penetration resistance is the number of blows of a 140 pound hammer falling 30 inches to drive a 2-inch O.D., 1⅜-inch I.D. split spoon sampler one foot. Core drilling performed in accordance with ASTM D2113-08. The undisturbed sampling procedure is described by ASTM D1587-08 (2012). Unless other arrangements are made, NOVA will dispose of all soil and rock samples remaining at the time of report submission.

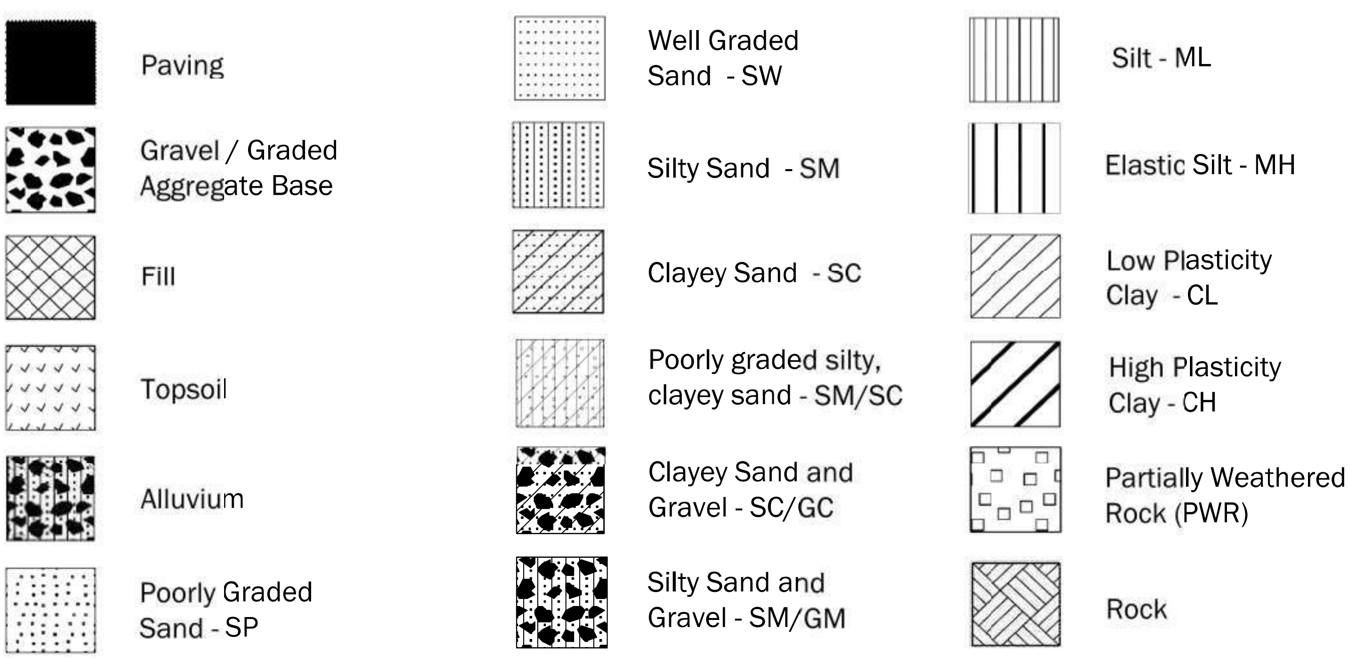
SOIL CLASSIFICATION CHART

COARSE GRAINED SOILS	**GRAVELS**	Clean Gravel less than 5% fines	GW	Well graded gravel
			GP	Poorly graded gravel
		Gravels with Fines more than 12% fines	GM	Silty gravel
			GC	Clayey gravel
	SANDS	Clean Sand less than 5% fines	SW	Well graded sand
			SP	Poorly graded sand
		Sands with Fines more than 12% fines	SM	Silty sand
			SC	Clayey sand
FINE GRAINED SOILS	**SILTS AND CLAYS** Liquid Limit less than 50	Inorganic	CL	Lean clay
			ML	Silt
		Organic	OL	Organic clay and silt
	SILTS AND CLAYS Liquid Limit 50 or more	Inorganic	CH	Fat clay
			MH	Elastic silt
		Organic	OH	Organic clay and silt
HIGHLY ORGANIC SOILS		Organic matter, dark color, organic odor	PT	Peat

PARTICLE SIZE IDENTIFICATION

GRAVELS	Coarse	¾ inch to 3 inches
	Fine	No. 4 to ¾ inch
SANDS	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40
SILTS AND CLAYS		Passing No. 200

STRATA SYMBOLS



NOVA

TEST BORING RECORD B-1

PROJECT: West Nancy Creek Dr. over Nancy Creek Tributary
CLIENT: Heath & Lineback Engineers, Inc.
PROJECT LOCATION: Brookhaven, DeKalb County, Georgia
LOCATION: Bent 1 - STA. 103+33, 25 feet RT.
DRILLER: Piedmont Drilling (D-50/SN 442)
DRILLING METHOD: Hollow Stem Auger % ENERGY: 82.0
DEPTH TO - WATER> INITIAL: 13 feet AFTER 24 HOURS: N/M CAVING> 15 feet

PROJECT NO.: 2022116
LATITUDE: 33.903697
LONGITUDE: -84.338086
ELEVATION: 867.6 feet
LOGGED BY: Eman Woods
DATE: 8-16-2022

Depth (feet)	Elevation (ft NAVD88)	Description	Groundwater	Uncorrected N-Value
0–0.7		ASPHALT: 8 INCHES		
0.7–3.5		FILL: Loose gray red silty medium to fine SAND		10 (2.5 ft)
3.5–8.5	865	Medium dense gray red silty medium to fine SAND		12 (5 ft); 12 (7.5 ft)
8.5–13.5	860	Loose brown gray silty fine SAND	Initial: 13 feet	9 (10 ft)
13.5–18.5	855	ALLUVIUM: Medium dense gray silty coarse to fine sand	Caving: 15 feet	11 (15 ft)
18.5–23.5	850	RESIDUUM: Dense light brown silty medium to fine SAND		32 (20 ft)
23.5–28	845	PARTIALLY WEATHERED ROCK: Sampled as very dense gray red silty coarse to fine SAND with rock fragments and mica		100/8" (25 ft)
28	840	Auger Refusal at 28 feet. Begin Rock Coring.		
28–36	835	Highly weathered moderately soft very intensely fractured gray micaceous SCHIST		REC 12% RQD 10%

Graphic Depiction

- BLOW COUNT
- NATURAL MOISTURE
- PLASTIC LIMIT ├──┤ LIQUID LIMIT

Scale: 10, 20, 30, 40, 60, 100

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

24-hour groundwater reading was not recorded due to the boring being backfilled on the same day of drilling.

NOVA

TEST BORING RECORD B-1

PROJECT: West Nancy Creek Dr. over Nancy Creek Tributary
PROJECT NO.: 2022116
CLIENT: Heath & Lineback Engineers, Inc.
LATITUDE: 33.903697
PROJECT LOCATION: Brookhaven, DeKalb County, Georgia
LONGITUDE: -84.338086
LOCATION: Bent 1 - STA. 103+33, 25 feet RT.
ELEVATION: 867.6 feet
DRILLER: Piedmont Drilling (D-50/SN 442)
LOGGED BY: Eman Woods
DRILLING METHOD: Hollow Stem Auger **% ENERGY:** 82.0
DATE: 8-16-2022
DEPTH TO - WATER> INITIAL: 13 feet **AFTER 24 HOURS:** N/M **CAVING>** 15 feet

Depth (feet)	Elevation (ft NAVD88)	Description	Graphic	Groundwater	SPT Sample	Shelby Tube Sample	Uncorrected N-Value
40, 45	830, 825, 820	Highly weathered very soft very intensely fractured gray micaceous SCHIST					REC 3% RQD 0%
50, 55	815, 810	Moderately weathered moderately hard intensely to moderately fractured olive gray micaceous SCHIST					REC 55% RQD 28%
60, 65	805, 800	Slightly weathered to fresh very hard slightly fractured bluish gray biotitic GNEISS					REC 94% RQD 68%
70		Rock Coring Terminated at 68 feet.					

Graphic Depiction

● BLOW COUNT
▲ NATURAL MOISTURE
PLASTIC LIMIT |———| LIQUID LIMIT
10 20 30 40 60 100

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

24-hour groundwater reading was not recorded due to the boring being backfilled on the same day of drilling.

NOVA

TEST BORING RECORD B-2

PROJECT: West Nancy Creek Dr. over Nancy Creek Tributary
CLIENT: Heath & Lineback Engineers, Inc.
PROJECT LOCATION: Brookhaven, DeKalb County, Georgia
LOCATION: Bent 2 - STA. 104+48, 25 feet RT.
DRILLER: Piedmont Drilling (D-50/SN 442)
DRILLING METHOD: Hollow Stem Auger % ENERGY: 82.0
DEPTH TO - WATER> INITIAL: 13 feet AFTER 24 HOURS: N/M CAVING> 13 feet

PROJECT NO.: 2022116
LATITUDE: 33.903791
LONGITUDE: -84.337828
ELEVATION: 867.8 feet
LOGGED BY: Eman Woods
DATE: 8-15-2022

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

Depth (feet)	Elevation (ft NAVD88)	Description	Uncorrected N-Value
0		ASPHALT: 5 INCHES	
		FILL: Loose brown silty medium to fine SAND	8
5	865		8
		Medium dense brown silty medium to fine SAND	12
	860	FILL: Firm gray fine sandy SILT	
10			6
	855	ALLUVIUM: Loose gray silty medium to fine SAND with rock fragments	
15			7
	850	ALLUVIUM: Loose gray orange fine sandy SILT	
20			7
	845	RESIDUUM: Medium dense brown silty medium to fine SAND	
25			22
	840	Loose brown gray silty medium to fine SAND	
30			9
	835		
35			9

Graphic Depiction: BLOW COUNT (●); NATURAL MOISTURE (▲); PLASTIC LIMIT |——| LIQUID LIMIT; scale 10, 20, 30, 40, 60, 100

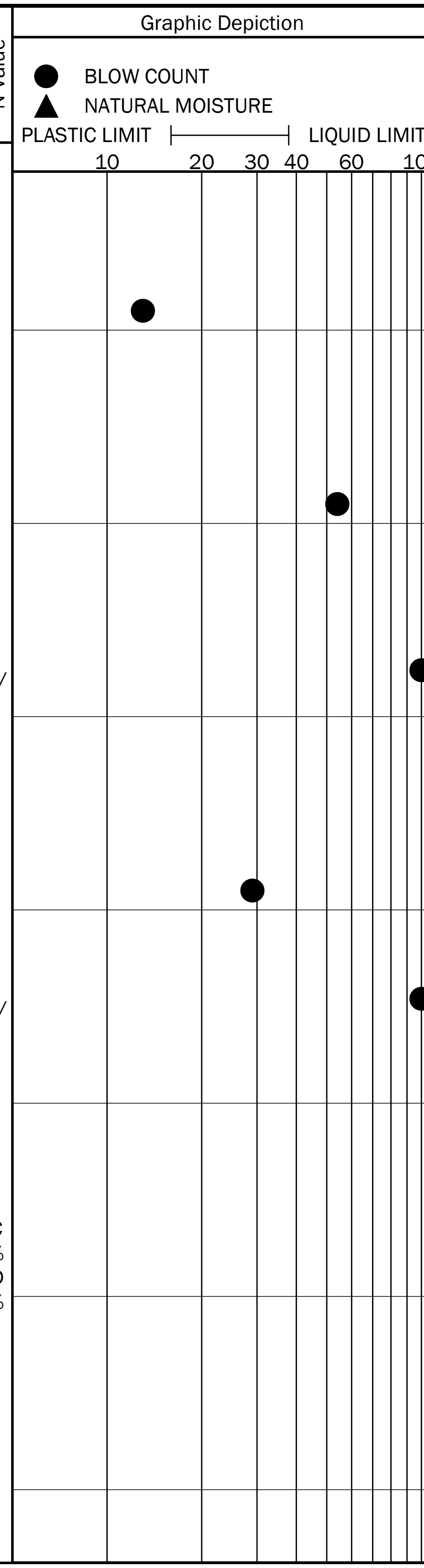
24-hour groundwater reading was not recorded due to the boring being backfilled on the same day of drilling.

NOVA

TEST BORING RECORD B-2

PROJECT:	West Nancy Creek Dr. over Nancy Creek Tributary	PROJECT NO.:	2022116
CLIENT:	Heath & Lineback Engineers, Inc.	LATITUDE:	33.903791
PROJECT LOCATION:	Brookhaven, DeKalb County, Georgia	LONGITUDE:	-84.337828
LOCATION:	Bent 2 - STA. 104+48, 25 feet RT.	ELEVATION:	867.8 feet
DRILLER:	Piedmont Drilling (D-50/SN 442)	LOGGED BY:	Eman Woods
DRILLING METHOD:	Hollow Stem Auger % ENERGY: 82.0	DATE:	8-15-2022
DEPTH TO - WATER> INITIAL:	13 feet AFTER 24 HOURS: N/M	CAVING>	13 feet

Depth (feet)	Elevation (ft NAVD88)	Description	Uncorrected N-Value
38	830	Medium dense brown silty coarse to fine SAND with rock fragments	13
43	825	Very dense brown silty coarse to fine SAND with rock fragments	54
48	820	PARTIALLY WEATHERED ROCK (LENS): Sampled as very dense gray silty medium to fine with mica	100/ 12"
53	815	RESIDUUM: Medium dense gray silty medium to fine SAND	29
56.5		PARTIALLY WEATHERED ROCK: Sampled as very dense gray silty medium to fine with mica	100/ 4"
58	810	Auger Refusal at 58 feet. Begin Rock Coring.	
		Moderately weathered moderately hard to hard intensely to moderately fractured bluish gray biotitic GNEISS with interbedded layers of micaceous SCHIST	REC 97% RQD 53%
68	800		



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

24-hour groundwater reading was not recorded due to the boring being backfilled on the same day of drilling.

TEST BORING RECORD B-2

NOVA

PROJECT:	West Nancy Creek Dr. over Nancy Creek Tributary
PROJECT NO.:	2022116
CLIENT:	Heath & Lineback Engineers, Inc.
LATITUDE:	33.903791
PROJECT LOCATION:	Brookhaven, DeKalb County, Georgia
LONGITUDE:	-84.337828
LOCATION:	Bent 2 - STA. 104+48, 25 feet RT.
ELEVATION:	867.8 feet
DRILLER:	Piedmont Drilling (D-50/SN 442)
LOGGED BY:	Eman Woods
DRILLING METHOD:	Hollow Stem Auger
% ENERGY:	82.0
DATE:	8-15-2022
DEPTH TO - WATER> INITIAL:	13 feet
AFTER 24 HOURS:	N/M
CAVING>	13 feet

Depth (feet)	Elevation (ft NAVD88)	Description	Graphic	Groundwater	SPT Sample	Shelby Tube Sample	Uncorrected N-Value
73–78	795–790	Slightly weathered to fresh very hard slightly fractured bluish gray biotitic GNEISS					REC 100% RQD 93%
78	790	Rock Coring Terminated at 78 feet.					

Graphic Depiction: BLOW COUNT; NATURAL MOISTURE; PLASTIC LIMIT — LIQUID LIMIT; 10 20 30 40 60 100

Depth scale: 75, 80, 85, 90, 95, 100, 105. Elevation scale: 795, 790, 785, 780, 775, 770, 765.

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

24-hour groundwater reading was not recorded due to the boring being backfilled on the same day of drilling.

APPENDIX D

LABORATORY TEST RESULTS

- Laboratory Tests Summary
- Atterberg Limits and Mechanical Sieve Analyses Results
- Unconfined Compressive Strength of Intact Rock Core Specimens Results

LABORATORY TESTS SUMMARY

Heath and Lineback Engineers, Inc
West Nancy Creek Drive over Nancy Creek Tributary - Bridge Replacement
City of Brookhaven, DeKalb County, Georgia
NOVA Project Number 2022116

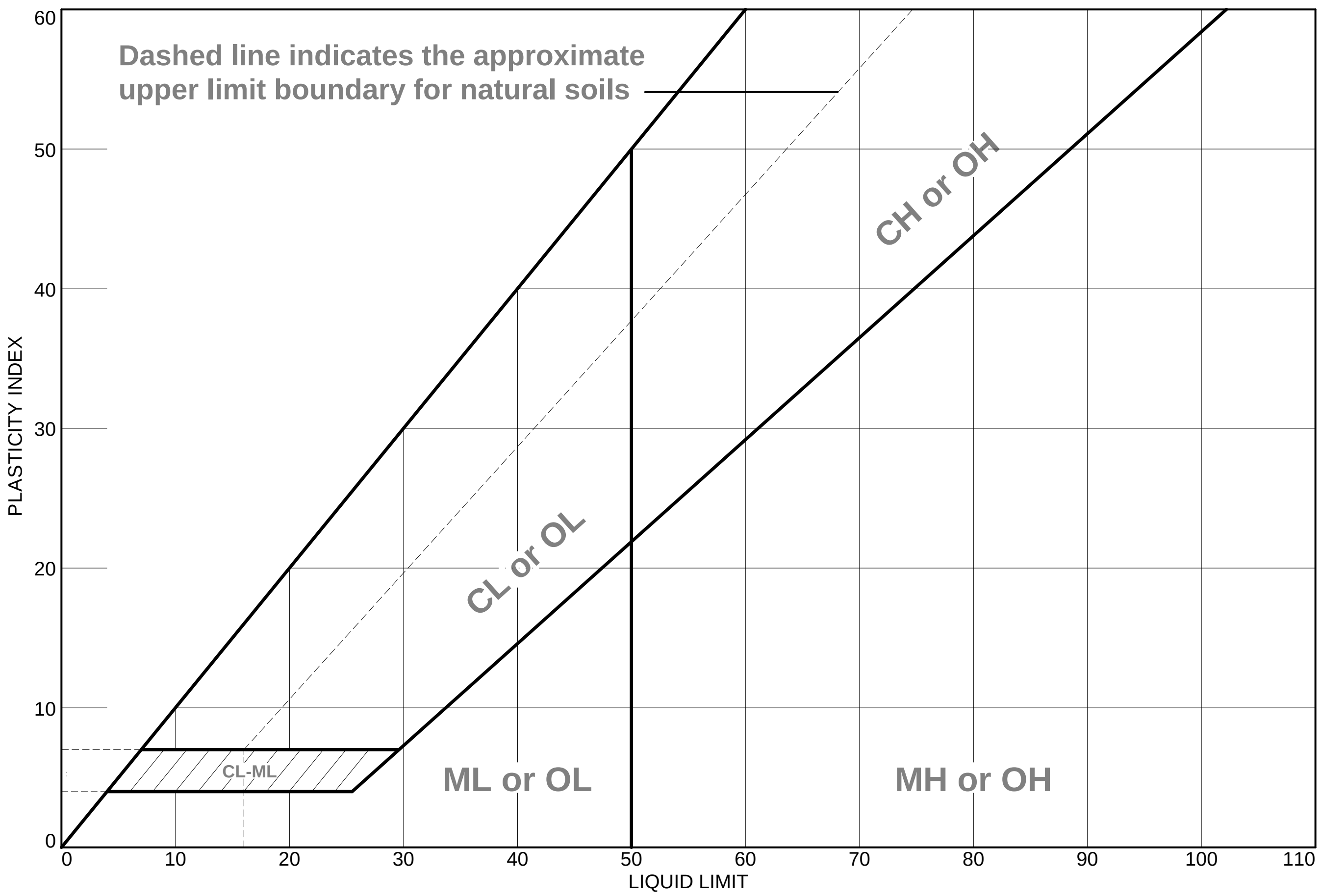
MECHANICAL SIEVE ANALYSES AND ATTERBERG LIMITS TEST RESULTS

BORING NO.	SAMPLE DEPTH (ft.)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	PERCENT FINER #40	PERCENT FINER #200	USCS CLASSIFICATION	MOISTURE CONTENT (%)
B-1	8.5-10.0	NP	NP	NP	97.3	33.2	SM	20.1
B-2	28.5-30.0	NP	NP	NP	86.8	46.1	SM	31.4

ROCK COMPRESSIVE STRENGTH TEST RESULTS

BORING NO.	SAMPLE DEPTH (ft.)	ELEVATIONS (ft.-NAVD88)	STRENGTH (psi)	YOUNG MODULUS (psi)	UNIT WEIGHT (pcf)
B-1	28.0-29.0	839.6-838.6	3,011	230,045	161.4
B-2	63.5-64.0	804.3-803.8	7,268	530,033	176.0

LIQUID AND PLASTIC LIMITS TEST REPORT ASTM D4318



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Yellowish brown and gray silty fine SAND	NP	NP	NP	97.3	33.2	SM

Project No. 2022116 **Client:** Heath & Lineback Engineers, Inc.

Project: West Nancy Creek Dr. over Nancy Creek Tributary - Bridge Replacement

● **Source of Sample:** B-1 **Depth:** 8.5-10 ft. **Sample Number:** 4

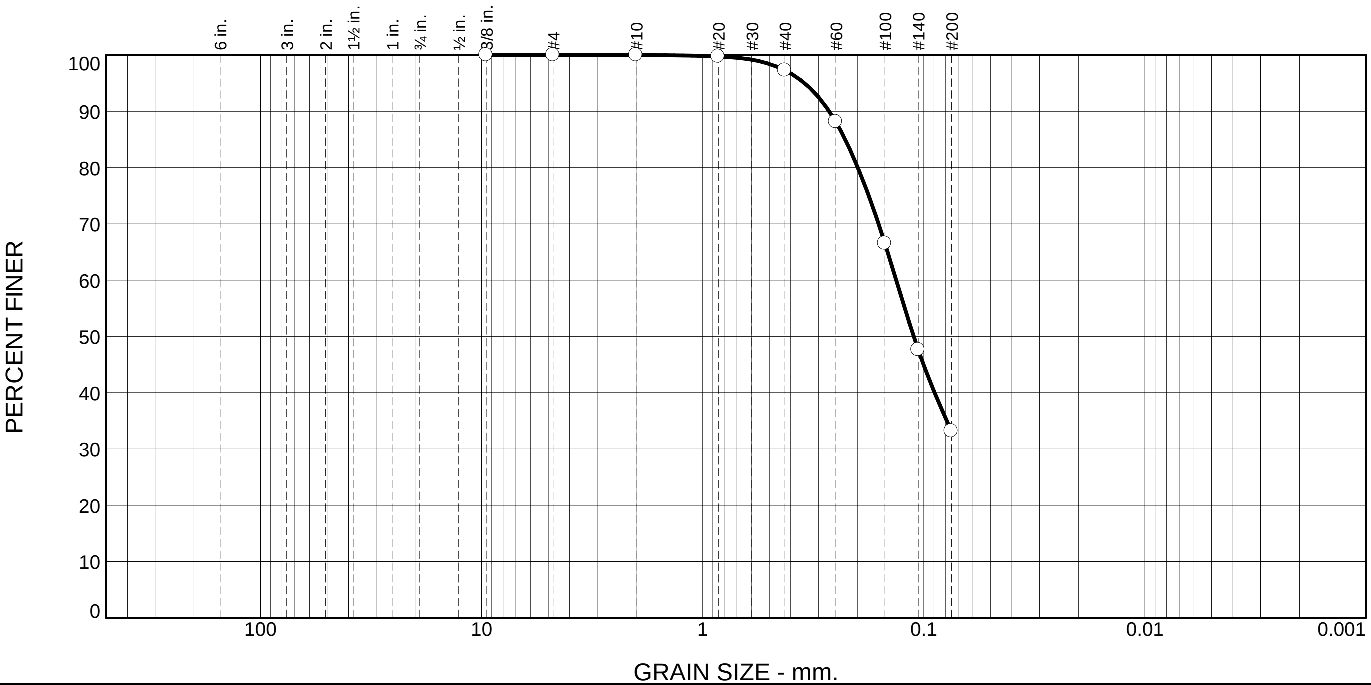
Remarks:

NOVA ENGINEERING
Kennesaw, Georgia
770-425-0777

Dated Tested 9/7/22

Tested By: MLS

Particle Size Distribution Report - ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	2.7	64.1	33.2	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8	100.0		
#4	100.0		
#10	100.0		
#20	99.7		
#40	97.3		
#60	88.1		
#100	66.5		
#140	47.6		
#200	33.2		

* (no specification provided)

Material Description

Yellowish brown and gray silty fine SAND

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NP **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

$D_{90}=$ 0.2678 $D_{85}=$ 0.2268 $D_{60}=$ 0.1333
$D_{50}=$ 0.1112 $D_{30}=$ $D_{15}=$
$D_{10}=$ $C_u=$ $C_c=$

Remarks

NMC = 20.1%

Date Received: **Date Tested:** 9/7/22

Tested By: BLM

Checked By: MLS

Title:

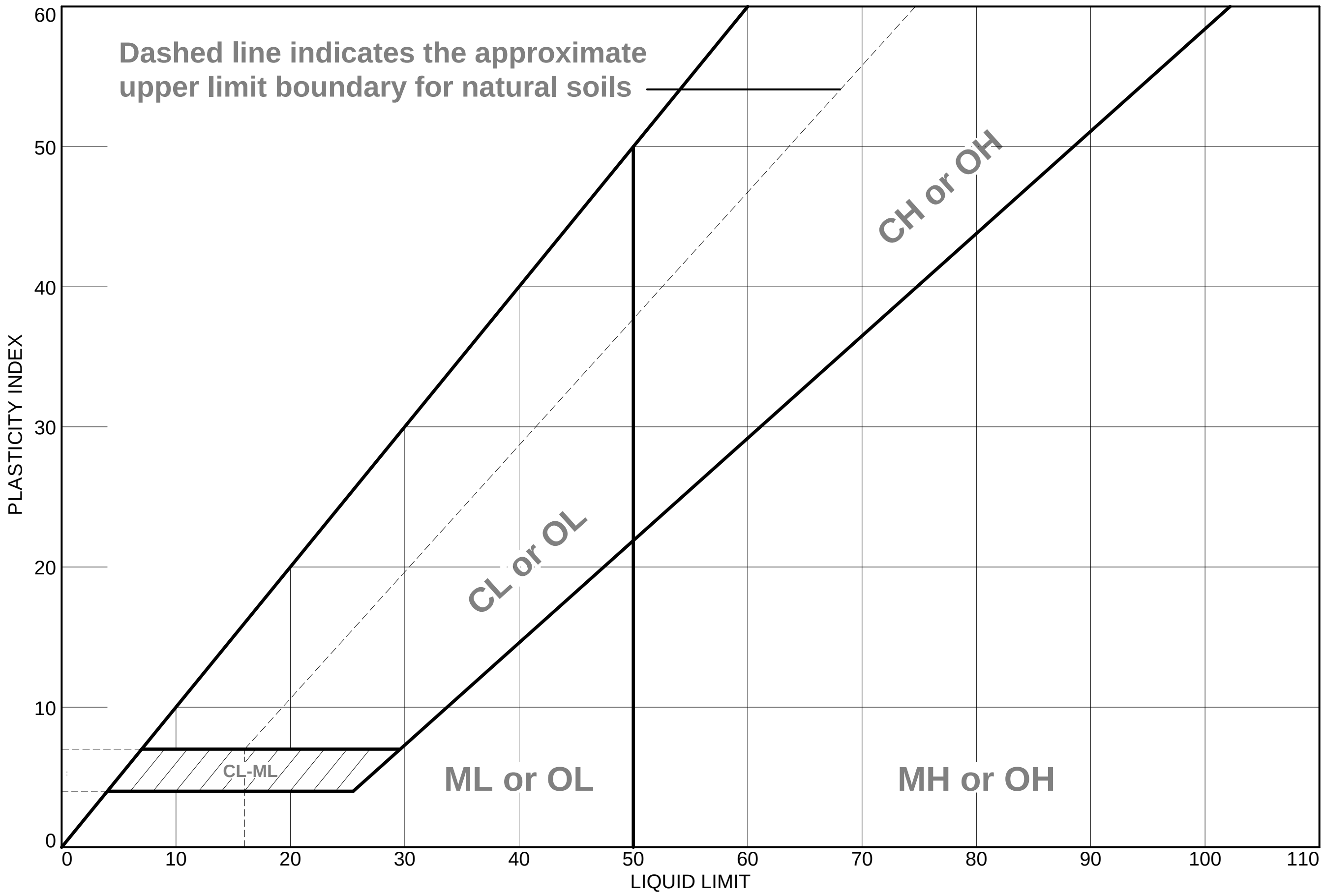
Source of Sample: B-1 **Depth:** 8.5-10 ft. **Date Sampled:**
Sample Number: 4

NOVA ENGINEERING
Kennesaw, Georgia
770-425-0777

Client: Heath & Lineback Engineers, Inc.
Project: West Nancy Creek Dr. over Nancy Creek Tributary - Bridge Replacement

Project No: 2022116 **Dated Tested** 9/7/22

LIQUID AND PLASTIC LIMITS TEST REPORT ASTM D4318



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grayish brown silty medium to fine SAND	NP	NP	NP	86.8	46.1	SM

Project No. 2022116 **Client:** Heath & Lineback Engineers, Inc.

Project: West Nancy Creek Dr. over Nancy Creek Tributary - Bridge Replacement

● **Source of Sample:** B-2 **Depth:** 28.5-30 ft. **Sample Number:** 8

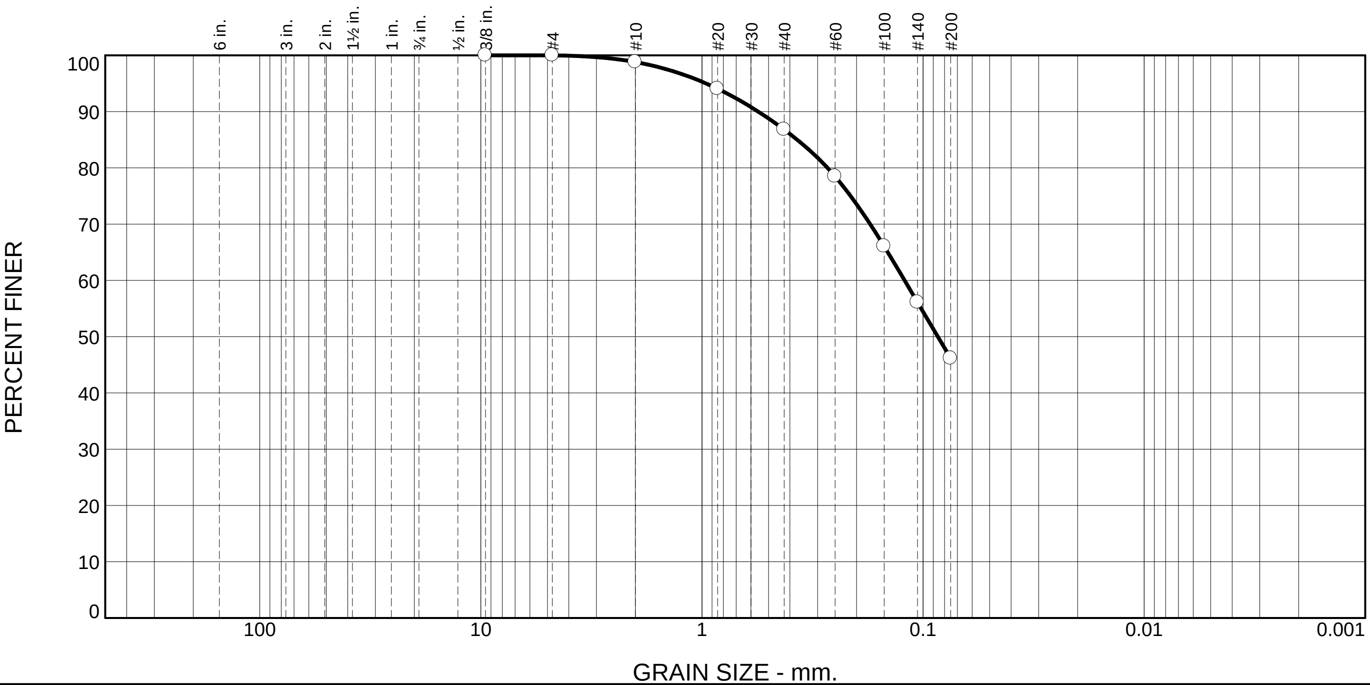
Remarks:

NOVA ENGINEERING
Kennesaw, Georgia
770-425-0777

Dated Tested 9/7/22

Tested By: MLS

Particle Size Distribution Report - ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.2	12.0	40.7	46.1	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8	100.0		
#4	100.0		
#10	98.8		
#20	94.1		
#40	86.8		
#60	78.5		
#100	66.1		
#140	56.1		
#200	46.1		

* (no specification provided)

Material Description

Grayish brown silty medium to fine SAND

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NP **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.5568 D_{85}= 0.3710 D_{60}= 0.1212
D_{50}= 0.0858 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

NMC = 31.4%

Date Received: **Date Tested:** 9/7/22
Tested By: BLM
Checked By: MLS
Title:

Source of Sample: B-2 **Depth:** 28.5-30 ft. **Date Sampled:**
Sample Number: 8

NOVA ENGINEERING
Kennesaw, Georgia
770-425-0777

Client: Heath & Lineback Engineers, Inc.
Project: West Nancy Creek Dr. over Nancy Creek Tributary - Bridge Replacement
Project No: 2022116 **Dated Tested** 9/7/22

TIMELY ENGINEERING SOIL TESTS, LLC

1874 Forge Street Tucker, GA 30084
Phone: 770-938-8233
Fax: 770-923-8973
Web: www.test-llc.com

Tested By	IH
Date	09/22/22
Checked By	

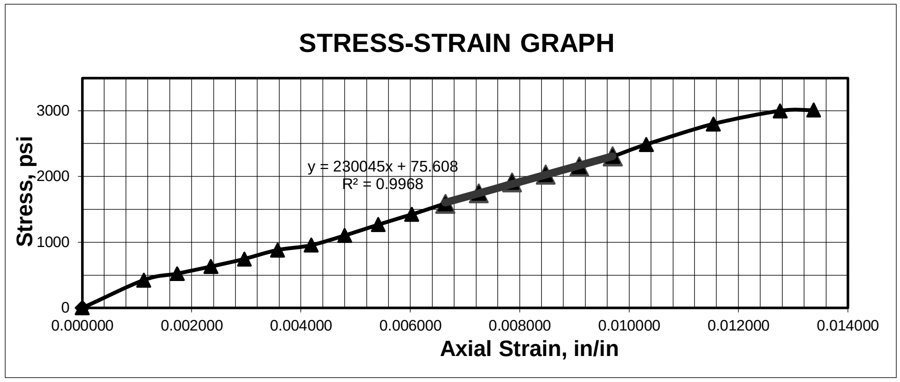
Client Pr. #	2022116	Lab. PR. #	2204A-28-1
Pr. Name	West Nancy Creek Drive over Nancy Creek Tributary	S. Type	Rock Core
Sample ID	43344/B-1	Depth/Elev.	28-29'
Subsample	-	Add. Info	-

ASTM D 7012 Method D

Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens

SAMPLE DATA

Initial Height, in	4.081
Initial Diameter, in	1.846
Height-to-Diameter Ratio	2.21
Initial Area, in^2	2.68
Initial Volume, in^3	10.92
Mass of Sample, g	462.70
Wet Density, pcf	161.4
Dry Density, pcf	161.1
Test Time, min	3.00
Temperature, C^o	26.0



Constant Rate of loading was selected for Failure during 2-15 min of compression.

TEST DATA

Balance ID	400/597	Oven ID	495/496
Load Cell ID	266/367	Caliper ID	370/458
Apparatus ID	267/366	Def. Indicator ID	1056

WATER CONTENT DETERMINATION (after test)

Mass of Wet Sample and Tare, g	594.10
Mass of Dry Sample and Tare, g	593.60
Mass of Tare, g	139.50
Moisture, %	0.1

Corrected Total Strain, in/in	Deformation w/t correction (inch)	Axial Load (lb)	Corrected Total Strain, %	UCS, (kPa)	UCS, (psi)
0.000000	0.0000	0	0.00	0	0
0.001124	0.0025	1131	0.11	2914	423
0.001737	0.0050	1397	0.17	3599	522
0.002350	0.0075	1685	0.23	4341	630
0.002962	0.0100	1995	0.30	5140	745
0.003575	0.0125	2355	0.36	6067	880
0.004187	0.0150	2554	0.42	6580	954
0.004800	0.0175	2952	0.48	7605	1103
0.005413	0.0200	3393	0.54	8741	1268
0.006025	0.0225	3813	0.60	9823	1425
0.006638	0.0250	4246	0.66	10939	1586
0.007250	0.0275	4675	0.73	12044	1747
0.007863	0.0300	5113	0.79	13172	1910
0.008476	0.0325	5425	0.85	13976	2027
0.009088	0.0350	5763	0.91	14847	2153
0.009701	0.0375	6171	0.97	15898	2306
0.010313	0.0400	6657	1.03	17150	2487
0.011539	0.0450	7494	1.15	19306	2800
0.012764	0.0500	8029	1.28	20684	3000
0.013376	0.0525	8059	1.34	20762	3011

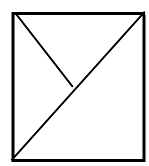
UNCONFINED COMPRESSIVE STRENGTH AT FAILURE, c (psi)	**3011**
UNCONFINED SHEAR STRENGTH AT FAILURE, s_u (psi)	1506
STRAIN AT FAILURE, %	1.3

REMARKS

Corrections (based on calibration) for Apparatus's deformation were applied to initial readings of deformation for calculations of Strain

Failure Code: 1

Failure Sketch



Failure Type: Cone

***Note: Young's modulus is calculated per Fig. 2 by method of Average Modulus of Linear Portion of Axial Stress-Strain Curve**

NOTE

Test specimen was prepared in accordance with ASTM D4543 (Procedures S1, P1 and FP-1)

USCS (ASTM D2487: D2488): NA

Young's modulus, psi*: **230045**

	TIMELY ENGINEERING SOIL TESTS, LLC	1874 Forge Street Tucker, GA 30084 Phone: 770-938-8233 Fax: 770-923-8973 Web: www.test-llc.com	

Tested By	IH
Date	09/22/22
Checked By	*(initials)*

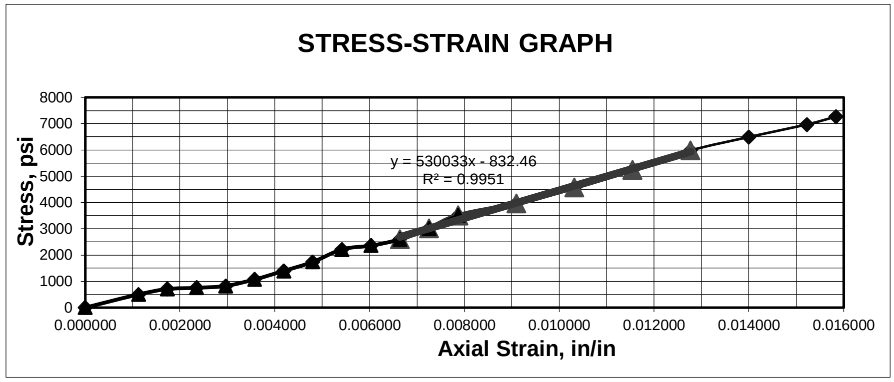
Client Pr. #	2022116	Lab. PR. #	2204A-28-1
Pr. Name	West Nancy Creek Drive over Nancy Creek Tributary	S. Type	Rock Core
Sample ID	43343/B-2	Depth/Elev.	63.5-64'
Subsample	-	Add. Info	-

ASTM D 7012 Method D

Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens

SAMPLE DATA

Initial Height, in	4.078
Initial Diameter, in	1.854
Height-to-Diameter Ratio	2.20
Initial Area, in^2	2.70
Initial Volume, in^3	11.01
Mass of Sample, g	508.50
Wet Density, pcf	176.0
Dry Density, pcf	175.7
Test Time, min	5.50
Temperature, C^o	26.0



Constant Rate of loading was selected for Failure during 2-15 min of compression.

WATER CONTENT DETERMINATION (after test)

Mass of Wet Sample and Tare, g	638.60
Mass of Dry Sample and Tare, g	638.20
Mass of Tare, g	140.90
Moisture, %	0.1

TEST DATA

Balance ID	400/597	Oven ID	495/496
Load Cell ID	266/367	Caliper ID	370/458
Apparatus ID	267/366	Def. Indicator ID	1056

Corrected Total Strain, in/in	Deformation w/t correction (inch)	Axial Load (lb)	Corrected Total Strain, %	UCS, (kPa)	UCS, (psi)
0.000000	0.0000	0	0.00	0	0
0.001125	0.0025	1352	0.11	3453	501
0.001738	0.0050	1906	0.17	4868	706
0.002351	0.0075	2033	0.24	5192	753
0.002964	0.0100	2223	0.30	5678	823
0.003577	0.0125	2899	0.36	7404	1074
0.004191	0.0150	3775	0.42	9641	1398
0.004804	0.0175	4695	0.48	11991	1739
0.005417	0.0200	5974	0.54	15258	2213
0.006030	0.0225	6370	0.60	16269	2360
0.006643	0.0250	7028	0.66	17950	2603
0.007256	0.0275	8139	0.73	20787	3015
0.007869	0.0300	9459	0.79	24159	3504
0.009095	0.0350	10698	0.91	27323	3963
0.010321	0.0400	12350	1.03	31542	4575
0.011547	0.0450	14167	1.15	36183	5248
0.012773	0.0500	16157	1.28	41265	5985
0.013999	0.0550	17518	1.40	44741	6489
0.015225	0.0600	18820	1.52	48067	6971
0.015838	0.0625	19620	1.58	50110	7268

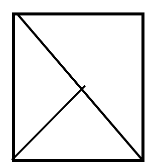
UNCONFINED COMPRESSIVE STRENGTH AT FAILURE, ((psi)	**7268**
UNCONFINED SHEAR STRENGTH AT FAILURE, s_u (psi)	3634
STRAIN AT FAILURE, %	1.6

REMARKS

Corrections (based on calibration) for Apparatus's deformation were applied to initial readings of deformation for calculations of Strain

Failure Code: 1

Failure Sketch



Failure Type: Cone

***Note: Young's modulus is calculated per Fig. 2 by method of Average Modulus of Linear Portion of Axial Stress-Strain Curve**

NOTE

Test specimen was prepared in accordance with ASTM D4543 (Procedures S1, P1 and FP-1)

USCS (ASTM D2487: D2488): NA

Young's modulus, psi*: 530033

APPENDIX E

SEISMIC SITE CLASS CALCULATION

SEISMIC SITE CLASS CALCULATION
(AASHTO LRFD 3.10.3.1 - Method B)

West Nancy Creek Drive over Nancy Creek Tributary - Bridge Replacement

NOVA Project No. 2022116

Boring No.	B-1	% Energy:	82.00
Location/Station:	STA. 103+33, 25' RT.	Boring Elevation (feet):	867.60
Depth Neglected * (feet):	0.00	SC Reference Elev. (feet):	867.60

Elevation (feet)	Boring Depth (feet) **	N (bl/ft)	SC Depth (feet) ***	N_{60} (bl/ft)	d_i (feet)	d_i/N_i
865	3	10	2.5	14	2.5	0.18293
863	5	12	5	16	2.5	0.15244
860	8	12	7.5	16	2.5	0.15244
858	10	9	10	12	2.5	0.20325
853	15	11	15	15	5.0	0.33259
848	20	32	20	44	5.0	0.11433
844	24	100	24	100	4.0	0.04000
840	28	100	28	100	4.0	0.04000
768	100	100	100	100	72.0	0.72000

* Boring log depth neglected for Site Class. ** Boring log depth to bottom of layer. *** Site Class depth to bottom of layer.

Average N	51.6
Site Class	C

SEISMIC SITE CLASS CALCULATION
(AASHTO LRFD 3.10.3.1 - Method B)

West Nancy Creek Drive over Nancy Creek Tributary - Bridge Replacement
NOVA Project No. 2022116

Boring No.	B-2	% Energy:	82.00
Location/Station:	STA. 104+48, 25' RT.	Boring Elevation (feet):	867.80
Depth Neglected * (feet):	0.00	SC Reference Elev. (feet):	867.80

Elevation (feet)	Boring Depth (feet) **	N (bl/ft)	SC Depth (feet) ***	N_{60} (bl/ft)	d_i (feet)	d_i/N_i
865	3	8	3	11	2.5	0.22866
863	5	8	5	11	2.5	0.22866
860	8	12	8	16	2.5	0.15244
858	10	6	10	8	2.5	0.30488
853	15	7	15	10	5.0	0.52265
848	20	7	20	10	5.0	0.52265
843	25	22	25	30	5.0	0.16630
838	30	9	30	12	5.0	0.40650
833	35	9	35	12	5.0	0.40650
828	40	13	40	18	5.0	0.28143
823	45	54	45	74	5.0	0.06775
818	50	100	50	100	5.0	0.05000
813	55	29	55	40	5.0	0.12616
811	57	100	57	100	2.0	0.02000
768	100	100	100	100	43.0	0.43000

* Boring log depth neglected for Site Class. ** Boring log depth to bottom of layer. *** Site Class depth to bottom of layer.

Average N	25.5
Site Class	D

APPENDIX F

Micropile Foundation Design Loads

Naveen Thakur

From: Gary Lineback <glineback@heath-lineback.com>
Sent: Wednesday, November 23, 2022 10:03 AM
To: Naveen Thakur
Cc: Ed Tavera
Subject: RE: West Nancy Creek Drive over Nancy Creek Tributary (Single Span Bridge) BFR

Naveen,

Yes. It is:

1. 3 piles each bent.
2. 17th Edition.
3. Service load is 380 kips/pile.

The factored loads are for our design of the cap. FS=2 for the service load is good.

Thanks,

Gary B. Lineback, P.E. | **Sr. Vice President / Chief Engineer**
Voice: 770.424.1668 x 105
glineback@heath-lineback.com

From: Naveen Thakur <NThakur@usanova.com>
Sent: Monday, November 21, 2022 1:02 PM
To: Gary Lineback <glineback@heath-lineback.com>
Cc: Ed Tavera <etavera@usanova.com>
Subject: RE: West Nancy Creek Drive over Nancy Creek Tributary (Single Span Bridge) BFR

Hello Gary,
We are wrapping up the BFI Report but just wanted to make sure the following:

- We understand that 3-pile group will be used per bent.
- Bridge will not be per LRFD and will be based on AASHTO 17th Edition.
- Maximum Service Load per pile = 380 kips.

Since the bridge will not be based on LRFD, there will not be any factored loads, right? However, in your table below, you have indicated LF = 580 kips. I am trying to understand what that means. Is it Factored Load or? In our report, we are going to base it on Factor of Safety of 2.

3-pile	~~290~~	~~19~~	~~15.2~~	
	380 (LF=580)	25	15.6	

APPENDIX G

Micropile Design Calculation

West Nancy Creek Drive over Nancy Creek Tributary - Bridge Replacement
City of Brookhaven, DeKalb County, Georgia
NOVA Project No. 2022116

Micropile Design	Bent 1	Bent 2
Service Load	380	380
Bond Zone Diameter in Rock, d_b (in)	8	8
Micropile Bonded Length In Rock, L_b (ft)	19	19
Ultimate Unit Grout-to-Ground Bond Strength, α_b (ksf)	20	20
Ultimate Grout-to-Ground Bond Capacity, R_s(kips)	796	796
Factor of Safety	2.0	2.0
Ultimate Tip Capacity, R_p (kips)	0	0
Allowable Axial Capacity of a Mircopile, R_R (kips)	398	398

Note [1]: At Bents 1 & 2, the bottom of micropile footing is estimated to be at elevation of 862 feet.
Note [2]: Micropile side resistance calculation was estimated based on ultimate grout to ground bond strength values from FHWA-NHI-05-039 (2005), Table 5-3, for Granite and Basalt (fresh-moderate fracturing, little to no weathering).
Note [3]: Micropile bonded length in rock was calculated based on micropiles embedded into competent rock. At Bent 1, side resistance in the upper 20 feet of rock (approximately from EL. 840 to 820 feet) is neglected due to poor quality rock encountered in test boring B-1. The top of competent rock at Bent 1 is at approximate EL. 820 feet and at Bent 2 is at approximate EL. 810 feet.
Note [4]: Allowable Axial Capacity was estimated based on a Factor of Safety of 2.

APPENDIX H

DRILL RIG CALIBRATION

SPT Automatic Hammer Energy Measurement Report

Drill Rig Model: Diedrich D-50
Serial Number: D50-442
Piedmont Environmental Drilling Drill Rig Asset Number: 442
March 2, 2022

Photograph depicts Piedmont Environmental Drilling Rig D-50-442 and crew.

Prepared for:
Piedmont Environmental Drilling, Inc.
Norcross, Georgia

Prepared by:
Terracon Consultants, Inc.
Exploration Services Group

March 2, 2022

Piedmont Environmental Drilling, Inc.
2722 Simpson Circle
NW, Norcross, GA 30071

Attn: Mr. Drew Roach
E: dmr@piedmontdrilling.com

Re: SPT Automatic Hammer Energy Measurement Report
Piedmont Environmental Drilling Asset Number: 442
Drill Serial Number: 442
Drill Rig Make and Model: Diedrich D-50
Terracon Project Number: 49205253A

Dear Mr. Roach:

This report provides the Energy Transfer Ratio (ETR) for the SPT automatic hammer found on drill rig model Diedrich D-50; Piedmont Environmental Drilling Asset Number 442 (Serial Number: D50-442).

Table 1: Hammer Measurement Summary

Drill Rig Model	Serial No.	Drill Rig Year	Drill Rig No.	Energy Transfer Ratio (ETR)	Hammer Efficiency Correction (C_E)
Diedrich D-50	D50-442	2019	442	82.0% ± 2.9%	1.37

If you have any questions concerning this summary, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

James Smith
National Exploration Manager

Marie Maher, P.G.
Regional Exploration Manager

R.L. (Levi) Denton II, P.E.
National Director Exploration Services

Attachments:
Exhibit A: Measurement Information
Exhibit B: PDA SPT Analyzer Results

Exhibit A
Measurement Information

MEASUREMENT INFORMATION

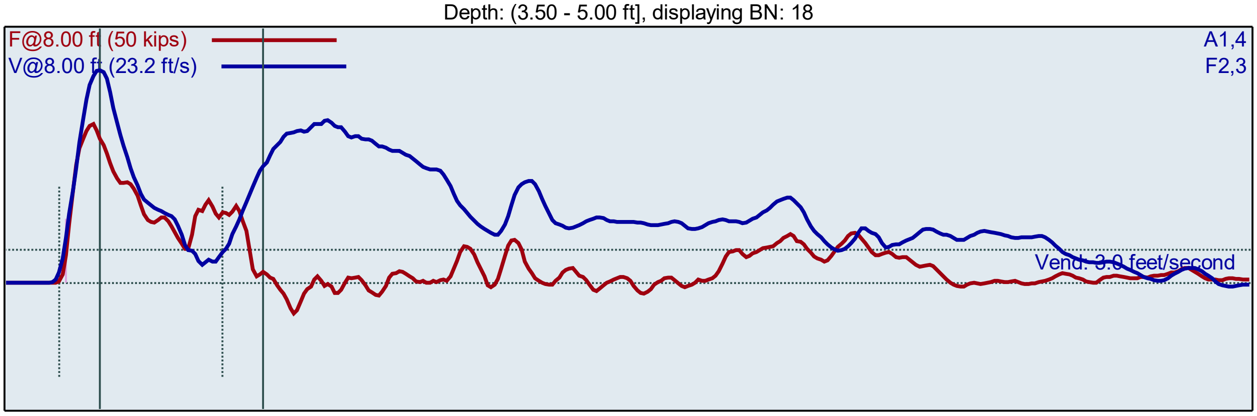
ITEM	DESCRIPTION
Drill Rig Identification	Drill Rig Model: Diedrich D-50 Drill Rig Year: 2019 Drill Rig Asset No.: 442; Serial No. D50-442
Drill Rig Owner	Piedmont Environmental Drilling, Inc. – Norcross, GA
Drill Rig Operator	Mark Warner; Piedmont Environmental Drilling
Testing Date	02/18/2022
Testing Location	Norcross, GA Office
Boring Identification	B-1
Hammer Type	140 pounds (automatic)
Boring Method	Hollow Stem Auger
Drill Rods	■ AWJ ■ 1 3/4" outside diameter ■ 3/16" wall thickness
Testing Equipment	■ 2-foot AWJ rod instrumented w/ 2 strain gauges and 2 accelerometers ■ Model SPT Analyzer™ (PDA)
ASTM Methods Used	**ASTM D1586**, Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils **ASTM D4633-16**, Standard Method for Energy Measurement for Dynamic Penetrometers
Personnel	Jim Smith – National Exploration Manager - Terracon Consultants, Inc.

Exhibit B
PDA SPT ANALYZER RESULTS

PED-442_1 3.5-5
Jim Smith Test date: 2/18/2022

AR: 1.21 in^2 SP: 0.492 k/ft3
LE: 8.00 ft EM: 30000 ksi
WS: 16807.9 ft/s

Depth: (3.50 - 5.00 ft], displaying BN: 18



F2 : [454AWJ2] 201.47 PDICAL (1) FF1
F3 : [454AWJ1] 202.29 PDICAL (1) FF1
A1 (PR): [K10492] 441.86 mv/6.4v/5000g (1) VF1
A4 (PR): [K4483] 410.187 mv/6.4v/5000g (1) VF1

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
1	2	26	19.5	1.9	247	70.5
2	2	30	20.3	44.5	329	94.0
3	7	31	19.8	44.3	313	89.4
4	7	30	18.8	46.1	308	88.1
5	7	30	18.7	46.3	309	88.4
6	7	31	19.0	46.3	307	87.7
7	7	29	19.0	46.2	297	84.9
8	7	29	18.6	46.6	312	89.1
9	7	29	17.8	46.4	286	81.6
10	11	29	18.5	46.2	290	82.8
11	11	29	18.0	46.3	294	83.9
12	11	30	18.5	46.3	287	82.0
13	11	28	18.1	46.3	291	83.3
14	11	30	18.7	46.3	295	84.4
15	11	30	18.8	46.4	286	81.6
16	11	30	18.7	46.4	289	82.7
17	11	30	18.7	46.3	295	84.4
18	11	31	19.2	46.3	301	86.0
19	11	30	18.7	46.4	295	84.2
20	11	31	19.5	46.2	297	85.0
	Average	30	18.7	46.2	297	85.0
	Std Dev	1	0.5	0.5	9	2.5
	Maximum	31	19.8	46.6	313	89.4
	Minimum	28	17.8	44.3	286	81.6

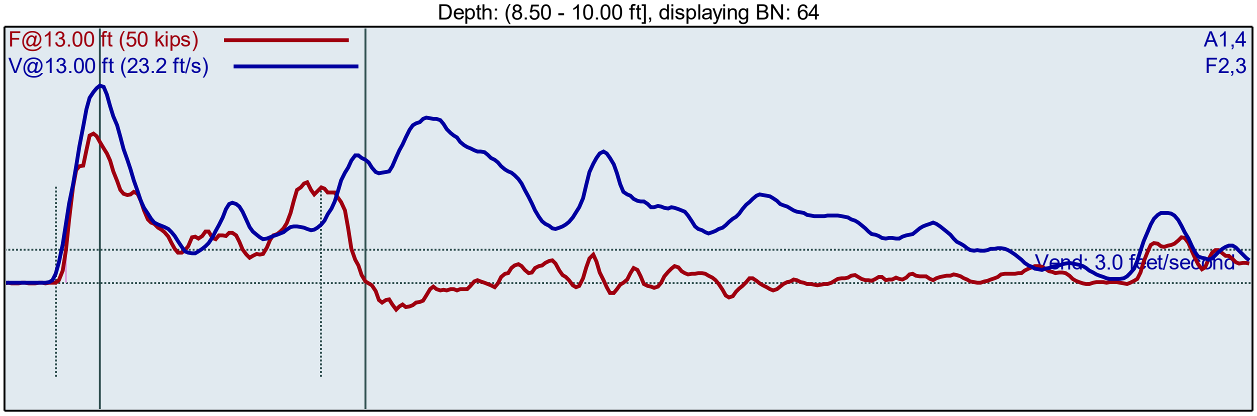
N-value: 18

Sample Interval Time: 24.70 seconds.

PED-442_1 3.5-5
Jim Smith Test date: 2/18/2022

AR: 1.21 in^2 SP: 0.492 k/ft3
LE: 13.00 ft EM: 30000 ksi
WS: 16807.9 ft/s

Depth: (8.50 - 10.00 ft], displaying BN: 64



F2 : [454AWJ2] 201.47 PDICAL (1) FF1
F3 : [454AWJ1] 202.29 PDICAL (1) FF1
A1 (PR): [K10492] 441.86 mv/6.4v/5000g (1) VF1
A4 (PR): [K4483] 410.187 mv/6.4v/5000g (1) VF1

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
21	4	32	20.9	1.9	304	86.8
22	4	34	21.3	45.8	312	89.2
23	4	33	21.1	45.5	322	91.9
24	4	32	19.9	45.5	297	84.8
25	26	33	20.4	45.6	301	86.1
26	26	34	20.1	45.5	301	86.0
27	26	34	20.1	45.4	299	85.3
28	26	32	18.9	45.4	295	84.2
29	26	31	18.3	45.5	288	82.4
30	26	32	18.4	45.3	282	80.5
31	26	31	18.1	45.6	272	77.7
32	26	29	17.2	45.3	282	80.7
33	26	30	18.1	45.5	288	82.2
34	26	31	18.2	45.4	290	82.8
35	26	30	18.8	45.4	293	83.7
36	26	30	18.0	45.4	298	85.3
37	26	30	18.5	45.6	297	84.9
38	26	31	18.2	45.4	295	84.2
39	26	31	18.2	45.4	297	84.8
40	26	29	18.1	45.5	285	81.5
41	26	29	18.4	45.4	292	83.5
42	26	31	18.2	45.5	292	83.3
43	26	29	17.5	45.4	288	82.3
44	26	30	18.2	45.4	284	81.0
45	26	30	18.1	45.6	279	79.6
46	26	30	17.7	45.5	284	81.1
47	26	29	17.8	45.2	287	82.0
48	26	29	17.6	45.5	283	80.9
49	26	30	18.0	45.5	280	80.1
50	26	30	18.1	45.4	280	80.1
51	16	29	17.6	45.4	290	82.7
52	16	30	18.0	45.4	287	81.9

53	16	29	17.5	45.6	292	83.4
54	16	28	17.3	45.3	279	79.8
55	16	29	17.7	45.6	283	80.7
56	16	31	18.4	45.3	282	80.5
57	16	29	17.8	45.6	279	79.8
58	16	29	18.0	45.5	280	80.0
59	16	30	18.1	45.4	284	81.1
60	16	29	17.9	45.5	287	82.0
61	16	29	17.9	45.4	281	80.2
62	16	30	18.2	45.5	286	81.8
63	16	29	18.0	45.5	273	78.0
64	16	29	17.9	45.3	269	76.9
65	16	30	18.0	45.5	280	80.1
66	16	30	17.9	45.4	280	80.1
	Average	30	18.2	45.4	286	81.8
	Std Dev	1	0.7	0.1	8	2.2
	Maximum	34	20.4	45.6	301	86.1
	Minimum	28	17.2	45.2	269	76.9

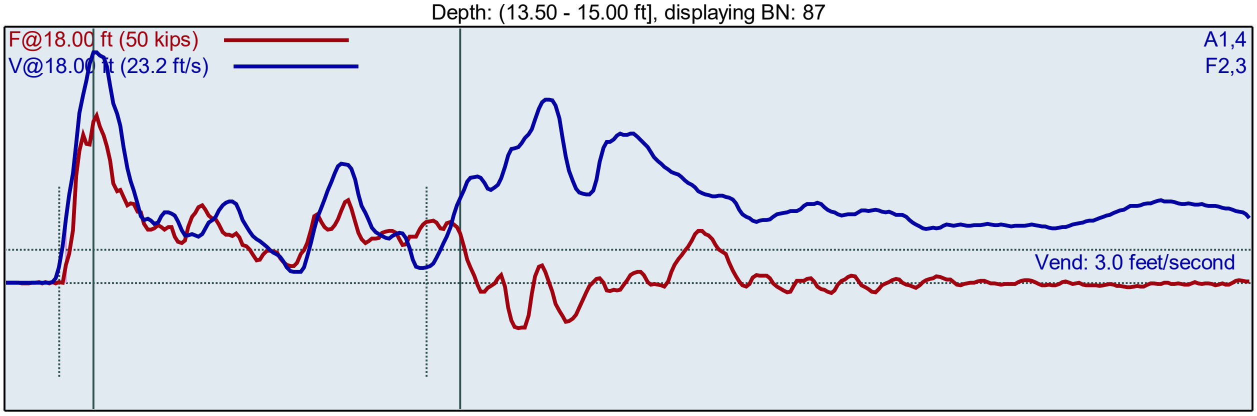
N-value: 42

Sample Interval Time: 59.41 seconds.

PED-442_1 3.5-5
Jim Smith Test date: 2/18/2022

AR: 1.21 in^2 SP: 0.492 k/ft3
LE: 18.00 ft EM: 30000 ksi
WS: 16807.9 ft/s

Depth: (13.50 - 15.00 ft], displaying BN: 87



F2 : [454AWJ2] 201.47 PDICAL (1) FF1
F3 : [454AWJ1] 202.29 PDICAL (1) FF1
A1 (PR): [K10492] 441.86 mv/6.4v/5000g (1) VF1
A4 (PR): [K4483] 410.187 mv/6.4v/5000g (1) VF1

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
67	6	34	21.4	1.9	273	77.9
68	6	33	20.3	46.7	298	85.1
69	6	32	19.8	46.5	290	82.8
70	6	31	19.5	46.3	294	84.1
71	6	31	19.7	46.4	291	83.1
72	6	30	19.1	46.4	302	86.2
73	8	32	20.6	46.3	309	88.2
74	8	30	19.2	46.3	290	83.0
75	8	30	19.5	46.4	285	81.5
76	8	30	19.2	46.5	282	80.5
77	8	29	18.8	46.2	288	82.3
78	8	30	19.1	46.3	291	83.1
79	8	30	19.4	46.6	300	85.7
80	8	30	19.3	46.2	296	84.5
81	9	31	19.4	46.5	289	82.5
82	9	30	18.9	46.2	288	82.3
83	9	32	19.9	46.4	291	83.3
84	9	30	18.9	46.5	286	81.6
85	9	31	19.8	46.3	292	83.4
86	9	31	19.2	46.3	297	85.0
87	9	33	20.9	46.4	304	86.8
88	9	32	20.1	46.3	300	85.6
89	9	31	19.5	46.4	287	82.0
	Average	31	19.5	46.4	293	83.6
	Std Dev	1	0.6	0.1	7	2.0
	Maximum	33	20.9	46.6	309	88.2
	Minimum	29	18.8	46.2	282	80.5

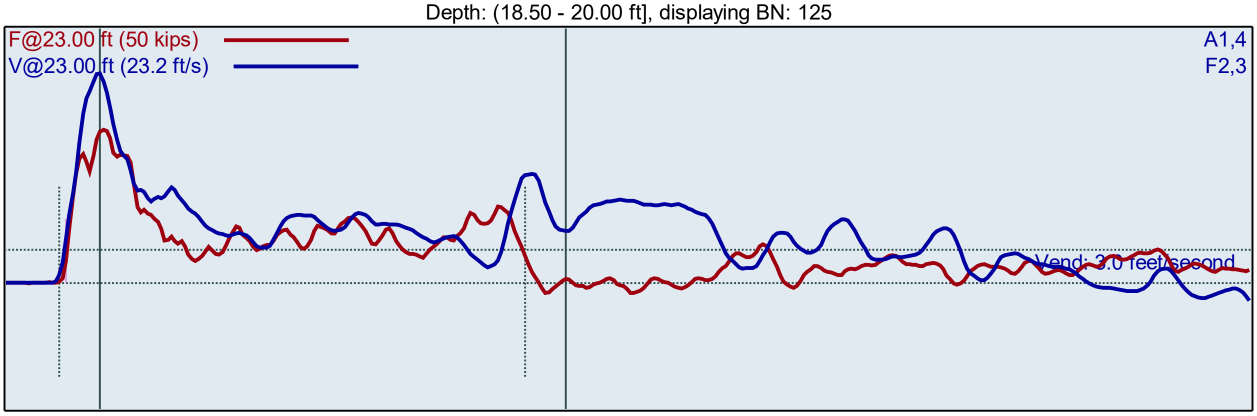
N-value: 17

Sample Interval Time: 28.45 seconds.

PED-442_1 3.5-5
Jim Smith Test date: 2/18/2022

AR: 1.21 in^2 SP: 0.492 k/ft3
LE: 23.00 ft EM: 30000 ksi
WS: 16807.9 ft/s

Depth: (18.50 - 20.00 ft], displaying BN: 125



F2 : [454AWJ2] 201.47 PDICAL (1) FF1
F3 : [454AWJ1] 202.29 PDICAL (1) FF1
A1 (PR): [K10492] 441.86 mv/6.4v/5000g (1) VF1
A4 (PR): [K4483] 410.187 mv/6.4v/5000g (1) VF1

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
90	8	26	23.8	1.9	239	68.3
91	8	31	17.6	43.3	276	78.8
92	8	29	18.2	43.4	276	78.8
93	8	28	19.4	43.2	273	78.1
94	8	28	18.5	43.3	262	74.7
95	8	27	18.2	43.2	259	73.9
96	8	30	18.3	43.3	283	80.9
97	8	32	18.6	43.2	292	83.5
98	13	30	18.4	43.2	294	84.0
99	13	29	18.3	43.3	278	79.4
100	13	31	19.1	43.3	284	81.1
101	13	29	17.8	43.4	282	80.7
102	13	28	19.5	43.3	278	79.6
103	13	29	18.8	43.3	279	79.7
104	13	28	18.1	43.2	274	78.4
105	13	29	18.8	43.4	275	78.6
106	13	29	18.2	43.3	278	79.5
107	13	28	18.1	43.1	270	77.1
108	13	28	18.2	43.3	264	75.4
109	13	28	18.1	43.2	263	75.1
110	13	28	18.5	43.4	271	77.4
111	17	30	18.9	43.2	289	82.6
112	17	29	18.5	43.3	280	80.1
113	17	29	18.6	43.2	283	80.7
114	17	29	18.9	43.3	281	80.3
115	17	30	18.3	43.3	272	77.8
116	17	29	18.2	43.3	278	79.4
117	17	30	19.0	43.2	279	79.8
118	17	30	18.9	43.4	283	80.8
119	17	30	18.8	43.3	275	78.7
120	17	30	18.7	43.2	286	81.8
121	17	30	18.9	43.3	285	81.3

122	17	30	18.9	43.3	272	77.7
123	17	31	19.1	43.3	267	76.3
124	17	30	18.6	43.2	287	82.0
125	17	30	18.9	43.3	284	81.3
126	17	30	18.7	43.2	285	81.5
127	17	30	19.0	43.3	284	81.3
	Average	29	18.6	43.3	279	79.6
	Std Dev	1	0.4	0.1	7	2.1
	Maximum	31	19.5	43.4	294	84.0
	Minimum	28	17.8	43.1	263	75.1

N-value: 30

Sample Interval Time: 51.30 seconds.

Summary of SPT Test Results

Project: PED-442_1, Test Date: 2/18/2022

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
8.00	**2-7-11**	**18**	**24**	30	18.7	46.2	297	85.0
13.00	**4-26-16**	**42**	**57**	30	18.2	45.4	286	81.8
18.00	**6-8-9**	**17**	**23**	31	19.5	46.4	293	83.6
23.00	**8-13-17**	**30**	**41**	29	18.6	43.3	279	79.6
			Overall Average Values:	30	18.6	45.1	287	82.0
			Standard Deviation:	1	0.7	1.2	10	2.9
			Overall Maximum Value:	34	20.9	46.6	313	89.4
			Overall Minimum Value:	28	17.2	43.1	263	75.1

Certificate of Calibration

Pile Dynamics, Inc. certifies that the

Pile Driving Analyzer®, Model SPT

Serial Number: 4535 TB

was calibrated on 8 NOVEMBER 2021
using a PDA Calibration Box whose output was calibrated with test equipment traceable to NIST.
This certificate is valid for 2 years from above date.

Tested by: MQ

Pile Dynamics, Inc.
30725 Aurora Road
Cleveland, Ohio 44139 USA

Accelerometer Calibration Certificate
Pile Dynamics, Inc.

Calibrated by Pile Dynamics, Inc.
Calibration performed on 26Oct2021

Serial No:	K4484	**Temperature:**	22.4 °C
Model:	PR	**Humidity:**	44%

Calibrated on: Channel 3 on 8G 5161 LE

Ref Acc 1:	69096! 978 g's/volt	**Cal on:**	27Jan2021
Ref Acc 2:	69132! 960 g's/volt	**Cal on:**	09Feb2021

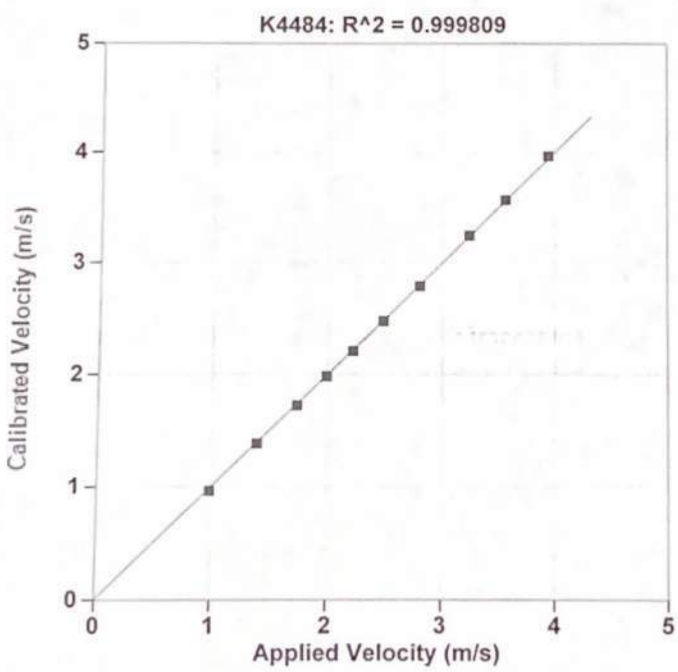
Reference accelerometer calibrations are traceable to the United States National Institute of Standards and Technology (NIST).

PDA CALIBRATION FACTOR

356.2 mv/5000g
(71.2 µv/g)
R^2: 0.999809 [Chip programmed]

Operator: William Johnson

Signed



Reference Velocity m/s	S/N K4484 Velocity m/s
0.986	0.965
1.393	1.383
1.737	1.725
1.992	1.985
2.219	2.209
2.482	2.477
2.800	2.790
3.235	3.241
3.558	3.562
3.937	3.962

Maximum Acceleration: 868 g's

Accelerometer Calibration Certificate
Pile Dynamics, Inc.

Calibrated by Pile Dynamics, Inc.
Calibration performed on 26Oct2021

Serial No:	K10492	**Temperature:**	22.4 °C
Model:	PR	**Humidity:**	44%

Calibrated on: Channel 3 on 8G 5161 LE

Ref Acc 1:	69096! 978 g's/volt	**Cal on:**	27Jan2021
Ref Acc 2:	69132! 960 g's/volt	**Cal on:**	09Feb2021

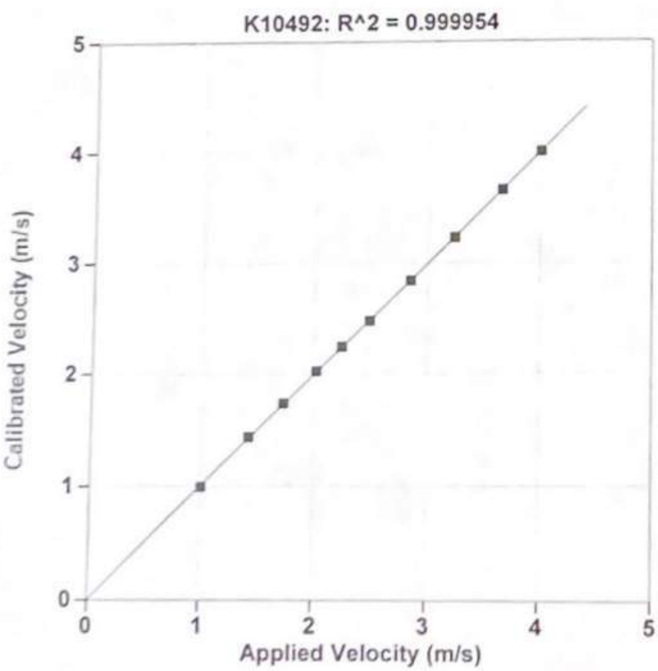
Reference accelerometer calibrations are traceable to the United States National Institute of Standards and Technology (NIST).

PDA CALIBRATION FACTOR

441.9 mv/5000g
(88.4 μv/g)
R^2: 0.999954 [Chip programmed]

Operator: William Johnson

Signed



Reference Velocity m/s	S/N K10492 Velocity m/s
1.007	0.995
1.424	1.432
1.727	1.736
2.015	2.021
2.240	2.244
2.481	2.478
2.841	2.841
3.232	3.228
3.658	3.657
4.005	4.002

Maximum Acceleration: 885 g's

Accelerometer Calibration Certificate
Pile Dynamics, Inc.

Calibrated by Pile Dynamics, Inc.
Calibration performed on 26Oct2021

Serial No:	K4483	**Temperature:**	22.1 °C
Model:	PR	**Humidity:**	45%

Calibrated on: Channel 3 on 8G 5161 LE

Ref Acc 1:	69096! 978 g's/volt	**Cal on:**	27Jan2021
Ref Acc 2:	69132! 960 g's/volt	**Cal on:**	09Feb2021

Reference accelerometer calibrations are traceable to the United States National Institute of Standards and Technology (NIST).

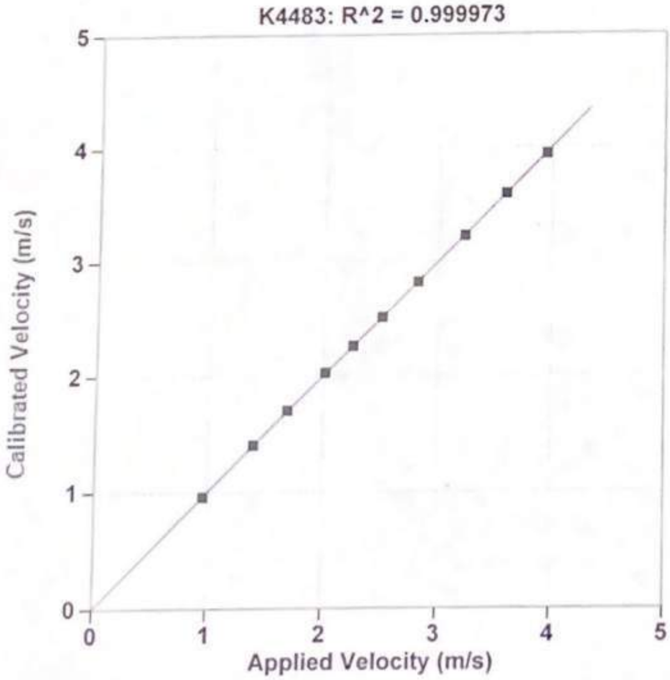
PDA CALIBRATION FACTOR

410.2 mv/5000g
(82.0 µv/g)
R^2: 0.999973 [Chip programmed]

Operator: William Johnson

Signed

K4483: R^2 = 0.999973



Reference Velocity m/s	S/N K4483 Velocity m/s
0.964	0.962
1.399	1.401
1.691	1.700
2.014	2.022
2.254	2.257
2.507	2.508
2.815	2.814
3.226	3.220
3.590	3.591
3.947	3.941

Maximum Acceleration: 874 g's

APPENDIX I

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled*. No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed*. The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

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