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Via Email

October 1, 2020
File No. 04.0191113.00

Ms. Jillian A. Semprini, P.E.
Hoyle, Tanner & Associates, Inc.
150 Dow Street
Manchester, New Hampshire 03101

Re: Geotechnical Engineering Report
Maplewood Avenue Culvert Replacement over the North Mill Pond
NHDOT Bridge No. 231/103
Portsmouth, New Hampshire

Dear Ms. Semprini:

This report presents GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical engineering recommendations for the proposed Maplewood Avenue Culvert Replacement over the North Mill Pond in Portsmouth, New Hampshire. This study has been conducted in accordance with our agreement for services with Hoyle, Tanner & Associates, Inc. (HTA) executed on June 6, 2020 and our proposal dated September 19, 2019. The contents of this report are subject to the *Limitations* set forth in **Appendix A**.

GZA's understanding of the project is based on discussions with you and the report entitled "Maplewood Ave Culvert Replacement and North Mill Pond Restoration, Culvert Replacement Alternative Analysis" prepared by the City of Portsmouth.

OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate the subsurface conditions with respect to the proposed construction, perform geotechnical engineering analyses, and develop foundation design and construction recommendations for the temporary and permanent replacement bridges. To meet these objectives, GZA completed the following Scope of Services:

- Conducted a site visit to observe surficial conditions, traffic, and site access;
- Coordinated and observed a geophysical survey of the current causeway to evaluate and identify the limits of the existing abandoned culvert openings, abandoned buried structures, and the locations of underground utilities;
- Coordinated and observed a subsurface exploration program consisting of six test borings designated B-101 through B-106 to evaluate subsurface conditions;

- Conducted a laboratory testing program to evaluate the engineering properties of the soil and bedrock encountered in the test borings;
- Conducted geotechnical engineering analyses to evaluate the impacts of subsurface conditions on the proposed construction;
- Developed geotechnical engineering recommendations for the proposed temporary and permanent replacement bridge foundations; and
- Prepared this report summarizing our findings and recommendations.

BACKGROUND

The date of original construction of the Maplewood Avenue crossing is unknown. The crossing carries Maplewood Avenue over the North Mill Pond, a tidal pond within the Piscataqua River Estuary in Portsmouth, New Hampshire, as shown on **Figure 1, Locus Plan**. The City of Portsmouth (City) was able to locate circa 1896 archive drawings of the crossing which document when the crossing was changed from five tidal openings to the current single arch configuration. It is presumed that the remaining four arch structures were filled in and that the structures still exist beneath the current causeway.

The existing bridge consists of a single, 28-foot-diameter(±), corrugated steel barrel stone arch culvert with adjacent granite block seawalls. The bridge is approximately 45 feet wide and carries two lanes of traffic and one bike lane. The culvert is reportedly supported on concrete footings. Multiple underground and overhead utility relocations and upgrades are proposed as part of the project, including the relocation of the gravity sewer main which currently crosses through the arch above the water line, replacement of a water main in the project vicinity, and the elimination of overhead utilities at the bridge crossing.

The bridge was inspected by the New Hampshire Department of Transportation (NHDOT) in November of 2008 and was found to be structurally deficient. The bridge is also on the Municipal Red List. In addition, the City conducted an inspection of the adjacent seawalls in 2007, at which time it was determined that the seawalls supporting the roadway and abutting the culvert were deficient.

In 2009 Waterfront Engineers LLC of Stratham, New Hampshire evaluated several replacement options using readily available precast concrete spans (CON/SPAN Bridge System). Four alternatives were presented to the City: 1) do nothing, 2) a single-span opening of relatively the same size, 3) a triple-span opening consisting of one 28-foot arch and two 20-foot arches, and 4) a single-span opening with one 48-foot arch. It is our understanding the project was not funded at that time.

Based on current project objectives, we understand the City intends to increase the single culvert opening at the crossing to enhance tidal flow, and that the current engineering study will include evaluations for: (1) repairs to the existing bridge/culvert, and (2) a permanent replacement bridge option. The project team is considering the construction of a temporary "jumper bridge" as part of the culvert/bridge repair option, which would span over the existing structure and allow use of the road while repairs are made to the existing culvert. The permanent replacement bridge option is anticipated to consist of a 80- to 100-foot-long, single span bridge supported by end-bearing steel piles.

PREVIOUS SUBSURFACE EXPLORATIONS

Test borings were drilled in October 2009 by Great Works Test Boring for John Turner Consulting, Inc. in the vicinity of the bridge. Three of the test borings, designated B1 through B3, were located in the vicinity of the proposed sewer siphon locations. The borings were drilled to depths of approximately 16.5 to 28.5 feet below ground surface (bgs). The locations of the three borings are shown on **Figure 2, Site and Subsurface Exploration Plan**. Logs of the previous explorations are included in **Appendix B**.

SUBSURFACE EXPLORATIONS

Subsurface investigations conducted for this project by GZA consisted of an initial geophysical survey of the current causeway to evaluate and identify the limits of the existing abandoned culvert openings and the locations of underground utilities, followed by the drilling of six test borings to evaluate the overburden soils and depth to and quality of bedrock for the permanent replacement and temporary bridge options.

GEOPHYSICAL SURVEY

GZA subcontracted Hager-Richter Geoscience, Inc. (HRGS) to conduct a geophysical survey along the causeway. The geophysical survey was conducted using complementary geophysical methods: time domain electromagnetic induction metal detection (EM), ground penetrating radar (GPR), and precision utility location (PUL) to locate subsurface utilities and other structures of interest. Multichannel analysis of surface waves (MASW) and Refraction Microtremor (ReMi) seismic data were also acquired to measure shear wave velocity and to evaluate the possible locations of the former culvert/arches in the specified area of interest.

The locations of multiple possible utilities were determined using EM, GPR and PUL data. The approximate limits of the present arch structure and opening, the locations of four possible former piers, and the locations of two possible former culvert/arches were detected using GPR data. The locations of detected possible utilities and other possible features are shown on **Figure 2**. The results of HRGS' geophysical survey are presented in their report entitled "Geophysical Survey, Maplewood Avenue Bridge, Portsmouth, New Hampshire" dated July 24, 2020, included in **Appendix C**.

TEST BORINGS

GZA completed a subsurface exploration program consisting of six test borings (designated B-101 through B-106). Two borings were drilled in the vicinity of the proposed abutments for the permanent replacement bridge (B-103 and B-104), and four borings were drilled in the vicinity of the proposed sewer siphon locations (B-101, B-102, B-105, and B-106). The borings were vacuum excavated to between 7 and 9.5 feet bgs within the understood depth range of possible underground utilities. The borings were drilled using a truck-mounted drill rig and 4-inch driven casing and drive-and-wash drilling techniques.

The borings were drilled to depths of approximately 23 to 34 feet bgs, as summarized on **Table 1, Summary of GZA Test Borings**. Standard penetration testing (SPT) and split-spoon sampling were generally performed at 5-foot intervals below the depth that was vacuum excavated. Approximately 5 to 10 feet of bedrock core was obtained at boring locations B-101 and B-103 through B-106 using an NX core barrel (2.0-inch-diameter). The borings were backfilled with drill cuttings and filter sand and finished with 6 inches of concrete below a six-inch-thick asphalt patch.

New England Boring Contractors (NEBC) of Derry, New Hampshire coordinated Dig Safe® utility clearance and provided drilling services. The vacuum excavation and drilling were completed between August 4 and 7, 2020. GZA personnel observed the drilling and prepared logs of each boring. Soil samples were visually/manually classified according to the Modified Burmister classification system. Rock descriptions were generally consistent with International Society of Rock Mechanics (ISRM) classification methods. Test boring logs prepared by GZA are included in **Appendix D**. The approximate as-drilled boring locations were located by GZA personnel using tape ties from prominent site features and are shown on **Figure 2**.

LABORATORY TESTING

Seven soil gradation analyses and moisture content tests were conducted on soil samples recovered from the subsurface explorations to confirm visual-manual field classifications and for use in our engineering analyses. Unconfined compressive strength tests were conducted on two bedrock core samples recovered from the subsurface explorations. The tests were used to estimate the engineering properties of the bedrock for use in our engineering evaluations. The gradation analyses and bedrock testing were performed by Thielsch Engineering at their Cranston, Rhode Island facility. Laboratory testing results are included in **Appendix E**.

SUBSURFACE CONDITIONS

SUBSURFACE PROFILE

Approximately 14.4 to 22.5 feet of fill was encountered in the test borings overlying bedrock. Within the fill, distinct zones of cobbles, boulders and bricks were observed in borings B-101 through B-104, ranging in thickness from approximately 1.6 to 12.2 feet. A total of approximately 11 to 14 inches of asphalt pavement was encountered in borings B-101 through B-106 in two layers separated by about 3 inches of sand fill.

Fill was generally described as loose, brown to gray, fine to medium SAND and Gravel, trace to little Silt TO medium dense to dense, GRAVEL, some fine to coarse Sand, trace to little Silt. Traces of wood and brick were observed in numerous samples. As noted above, distinct zones of cobbles and boulders were observed within the fill. Blow counts indicating dense to very dense soil may have been affected by the presence of cobbles, boulders, brick or rock and may not be representative of the soil density. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix D**.

BEDROCK

Approximately 5 to 10 feet of bedrock core was obtained at boring locations B-101 and B-103 through B-106 using an NX core barrel (2.0-inch-diameter). The top of bedrock was encountered from approximately 15.3 to 23.5 feet bgs in the test borings.

Bedrock recovered from the cores was generally described as hard, fresh, gray and white, fine grained, PHYLLITE, consistent with the mapped geology of the site. Joints were generally extremely close to close, low to high angle, planar and stepped, rough, fresh to discolored, and tight to moderately open. The Rock Quality Designation (RQD) for the bedrock cores obtained in all borings drilled at the site ranged from 0 to 51 percent, with an average RQD of 12 percent, indicating very poor rock mass quality.

Two laboratory unconfined compressive strength tests were conducted on bedrock core samples collected from the test borings, as summarized in the table below. The testing yielded unconfined compressive strengths ranging from 7.7 to 22.3 kips per square inch (ksi), with an average unconfined compressive strength of 15.0 ksi. However, as noted below, the lower test result may have been influenced by the bedrock structure.

Boring Location	Sample Number	Depth (ft)	Unconfined Compressive Strength (ksi)	Comments
B-101	C-1	17.0-17.7	22.3	Break was fresh
B-106	C-4	27.6-28.3	7.7	Break was along existing foliation plane

GROUNDWATER

The drive-and-wash cased drilling method used introduces large quantities of water during drilling. Therefore, groundwater levels observed in the test borings do not necessarily represent stabilized groundwater levels. Groundwater was observed at the completion of drilling at depths ranging from 6.8 to 9.4 feet bgs. Groundwater is anticipated to coincide approximately with the pond water elevation and will be influenced by tidal conditions.

The groundwater observations were made at the times and under the conditions stated in the boring logs. Fluctuations in groundwater levels will occur due to variations in season, precipitation, pond level, tides, and other factors. Consequently, water levels during and after construction are likely to vary from those encountered in the borings at the time the observations were made.

GEOTECHNICAL DESIGN RECOMMENDATIONS

Based on the conditions encountered in the test borings and the historic as-built plans for the bridge, the geotechnical considerations at the site include the presence loose fill soils, buried foundations from the former arch structures, and zones of cobble, boulder and brick fill presumed to have been used to raise grade in the vicinity of borings B-101 through 104.

Spread footings are considered feasible for support of a temporary bridge following intensive surface compaction or excavation and replacement of the existing sand or fill soils, as described later in this report.

Permanent support of a replacement bridge abutments using H-piles is feasible but may require predrilling through the fill soils to reach the underlying bedrock due to the potential presence of buried cobbles, boulders and bricks. Give the relatively short pile lengths expected, it is anticipated that pre-drilled H-piles would require sockets into competent bedrock and/or be backfilled with peastone or sand to achieve fixity.

GZA conducted our geotechnical engineering evaluations in general accordance with the 2020 American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LFRD) Bridge Design Specifications, 9th Edition and the NHDOT Bridge Design Manual (BDM).

LOAD AND RESISTANCE FACTORS

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), and earth surcharge (ES) loads using the load factors for permanent loads (γ_p) provided in LRFD Tables 3.4.1-2 for strength and extreme limit state foundation design. For service limit state, a load factor of 1.0 should be applied to these loads.

It is anticipated that a temporary bridge can be supported on spread footings bearing on intensively surface compacted existing sand or fill soils or on compacted structural fill. Recommended LRFD resistance factors for strength limit state design of spread footing foundations from LRFD Table 10.5.5.2.2-1 are presented in the following table.

RESISTANCE FACTORS – STRENGTH LIMIT STATE		
Foundation Resistance Type	**Method/Condition**	**Resistance Factor (ϕ)**
Bearing	Footings on Soil	0.45
Sliding	Cast-in-Place Concrete/Leveling Pad on Sand	0.80
Sliding	Cast-in-Place Concrete on Cast-in-Place Concrete Leveling Pad [1]	0.80

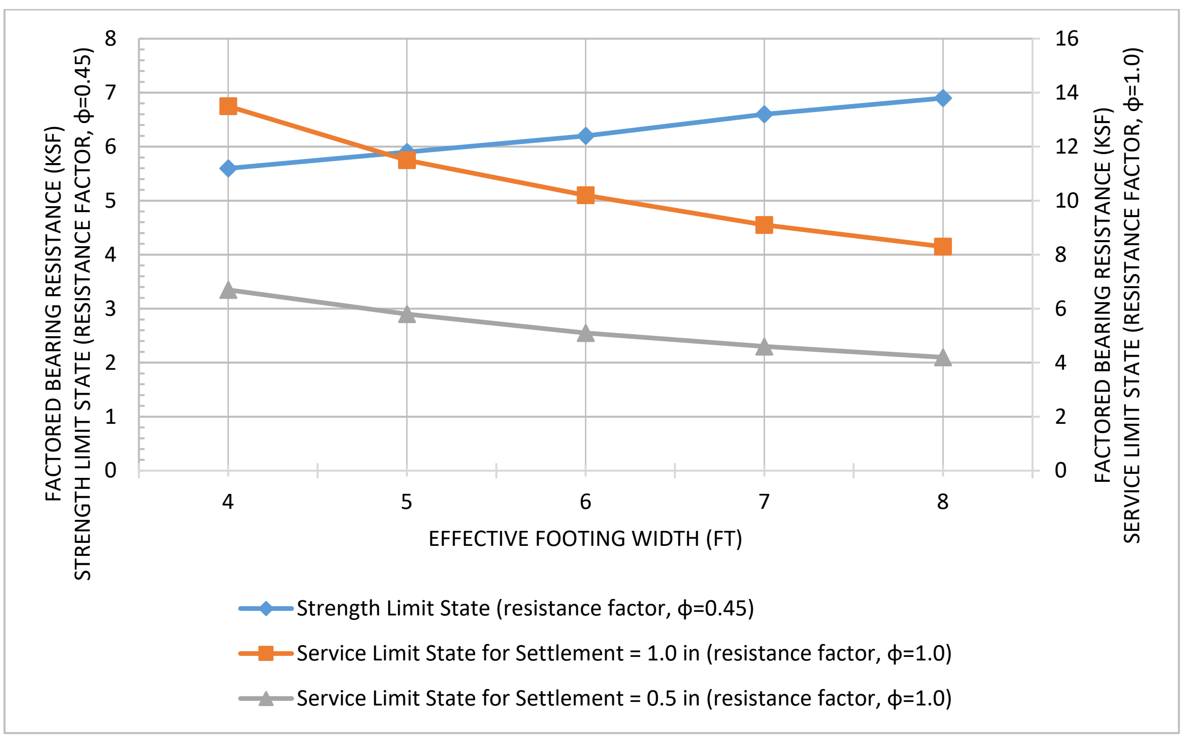
[1] Sliding resistance factor for concrete on or concrete is taken as equal to footing on sand.

Resistance factors for service and extreme limit state design should be taken as 1.0.

RECOMMENDATIONS FOR SHALLOW FOUNDATIONS

The existing sand and fill soils are generally loose and variable. To facilitate the use of spread footings, GZA recommends the sand or fill be either intensively surface compacted, or over excavate two feet of existing sand or fill, proof compact the subgrade, and replace with compacted Structural Fill (NHDOT Item 508.2.1, Crushed Gravel for Structural Fill).

Spread footings founded on intensively surface compacted existing sand or fill soils or properly compacted Structural Fill should be designed for factored bearing resistance for strength and service limit states per the chart below.



Factored Bearing Resistance Chart for Length of footing (L) = 50 feet and Depth of embedment (D_f) = 4 feet.

- For shallow foundations, lateral loads may be resisted by friction between the footing bottoms and subgrade. The recommended nominal sliding coefficient between cast-in-place concrete and properly prepared subgrades are as follows:

NOMINAL COEFFICIENT OF FRICTION VALUES	
Condition	**Friction Factor**
Concrete Cast on Sand	0.55[1]
Concrete Cast on Structural Fill	0.67

1. Reference: AASHTO Table 3.11.5.3-1

- GZA recommends that passive earth pressures not be considered for lateral resistance.
- Walls subject to vehicle loading should be designed in accordance AASHTO 3.6.1.2.1. The recommended traffic surcharge loading is HL-93 loading.
- Backfill behind new abutments should be placed in accordance with the recommendations outlined the Fill Material and Placement Recommendations section of this report.
- GZA recommends that spread footings be founded a minimum of 4 feet below the lowest adjacent ground surface to provide frost protection.

PILE DESIGN CONSIDERATIONS

HTA is considering the use of pile-supported abutments for support of the replacement bridge. In GZA's opinion, the use of H-pile supported abutments is a viable foundation system for the site, however predrilling may be required to advance the piles through the existing fill soils due to the presence of cobble, boulders, and bricks. Due to the relatively shallow depth to bedrock it is anticipated piles will need to be socketed into competent bedrock or be backfilled with peastone or sand. Although district zones of cobbles, boulders and bricks were only observed in test borings B-101 through B-103, the Contractor should be prepared to predrill if cobbles, boulders, or bricks are encountered during pile driving. The piles should consist of ASTM A572, Grade 50 steel, HP-section piles fitted with driving points to protect the tips during driving.

The piles are expected to derive their support primarily from end-bearing on bedrock. Piles may be HP 10x57, HP 12x84, or HP 14x89 as needed, depending on the magnitude of the factored design axial and lateral loads. Piles may be plumb, battered, or a combination of both. By utilizing steel H-piles driven to bedrock for support of the abutments, total and differential settlement should be limited to elastic compression of the piles and should be less than ½-inch.

The depth to the top of bedrock from existing grade ranged from approximately 20.9 to 23.5 feet bgs in borings B-103 and B-104 drilled in the vicinity of the proposed abutments. It should be noted that some variation in the top of bedrock elevation may exist laterally along each abutment. Due to the presence of cobbles, boulders and brick, piles should be pre-drilled from the bottom of the pile cap elevation through the obstructions and a minimum of 3-feet into competent bedrock to socket the piles and/or be backfilled with peastone or sand. Piles would be placed in the predrilled holes and driven with pile driving equipment to seat each pile into the competent bedrock. The piles should be anchored into bedrock with concrete or grout placed by tremie methods to provide sufficient lateral resistance. Above the bedrock, the predrilled holes would be filled with sand backfill.

Assuming a proposed bottom of pile cap at 4 feet below existing grade to provide foundations with frost protection and including a minimum 3-foot socket into competent bedrock, pile lengths on the order of 17 to 20 feet are anticipated. The estimated pile lengths do not include the pile length embedded in the pile cap or any additional footage needed to complete required testing of the piles during installation.

Piles should be designed at the strength and extreme limit states considering the structural resistance of the piles and the geotechnical resistance of the piles. For the scour extreme limit state, the design should consider the potential loss of lateral support due to scour during the design flood event if sufficient scour countermeasures cannot be implemented.

GZA conducted engineering evaluations for the estimated axial compressive resistance of the pile sizes listed above. Preliminary estimates of the factored structural axial compressive resistance for the three proposed pile sizes were calculated for both the strength limit and service limit states and are presented in the table below. The strength limit state was calculated using a resistance factor (ϕ_c) equal to 0.5 (severe driving conditions requiring the use of a driving tip) and a column slenderness factor (λ) equal to 0 assuming fully embedded piles. The service/extreme limit state was calculated using a resistance factor (ϕ) equal to 1.0 and a λ equal to 0. The nominal compressive resistance (Pn) was estimated as specified in LRFD Section 6.9.4.1.

The structural engineer should recalculate the λ for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and reduce the structural pile resistances below accordingly.

FACTORED STRUCTURAL AXIAL PILE RESISTANCE		
Pile Section	**Resistance (kips)** **Strength Limit State** $\phi_c = 0.5, \lambda=0$	**Resistance (kips)** **Service/Extreme Limit State** $\phi = 1.0, \lambda=0$
HP 10x57	420	840
HP 12x84	615	1230
HP 14x89	653	1305

Once the pile loads are finalized, a drivability analysis is recommended to assess if the piles can be installed to the required capacity without exceeding allowable driving stresses. Pile group effects were considered for axial compression, and since the piles are principally end-bearing on rock, there is no reduction recommended for group interaction in axial compression.

The drivability resistance of the piles should be calculated once maximum factored axial loads have been established. Drivability analysis should be completed in accordance with LRFD Section 10.7.8. Assuming the use of 50 ksi steel, the maximum driving stresses in the pile should be less than 45 ksi. The pile driving criteria for construction are expected to be established based on dynamic pile testing with signal matching analysis. The piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load by a resistance factor or 0.65, per LRFD Table 10.5.5.2.3-1.

LATERAL PILE RESISTANCE

Once the factored loads have been established, lateral pile resistance analyses should be performed to evaluate load-deformation behavior and to estimate combined stresses using an analysis approach that incorporates non-linear load-deformation behavior of the piles, such as L-Pile®, Group, or FB-Pier®. Proposed H-pile supported integral abutments should be analyzed for combined bending using the maximum factored axial load combined with the bending induced by thermal deflection of the deck. Driven H-piles should also be checked to confirm suitable fixity of the pile tip under the imposed pile head deformation.

Pile group interaction should be evaluated including the estimated vertical, transverse (i.e. perpendicular to the alignment) and longitudinal (i.e. along the centerline) deflections, and axial loads and bending moments for the anticipated H-pile spacing should be analyzed using GROUP or FB-Pier software.

Recommended geotechnical parameters for use in lateral pile analyses are provided in the tables below.

SOIL PARAMETERS FOR L-PILE® INPUT						
Strata Designation	Approx. Thickness (ft)	Effective Unit Weight lbs/in³ (lb/ft³)	k_s (lb/in³)	Cohesion (lb/in²)	E_{50} for clays/silts	Friction Angle
Fill or Crushed Gravel for Structural Fill above gw	5	0.069 (120)	25	-	-	30°
Fill below gw	15	0.033 (57.6)	20	-	-	30°

Note: Profile assumes bottom of pile cap is 4 feet below existing grade and depth to groundwater at the abutments is 9 feet below existing grade.

LATERAL EARTH PRESSURES

Lateral earth pressures should be calculated in accordance with the requirements of the latest edition of AASHTO and the NHDOT BDM assuming that NHDOT Item 209.20x Granular Backfill (Bridge) is used for backfill material. A unit weight (γ) equal to 120 pcf and an assumed friction angle of 34 degrees can be used to calculate earth loadings behind retaining walls.

SEISMIC CONSIDERATIONS

Evaluation of the seismic site class was based on the v_s-bar approach in accordance with AASHTO Table C3.10.3.1-1. v_s-bar is defined as the average shear wave velocity for the upper 100 feet of the soil profile ($V_{s\,100}$). For the Maplewood Avenue bridge site, the $V_{s\,100}$ measured by HRGS in four test lines ranged from 2,193 to 2,478 feet/second (ft/s); therefore, the site should be assigned to Site Class C.

Based on the site location, the recommended AASHTO Response Spectrum (Site Class C) for a 7 percent probability of exceedance in 75 years are summarized as follows:

SEISMIC DESIGN PARAMETERS Site Class C	
Parameter	Design Value
PGA	0.095 g
Ss	0.185 g
S1	0.045 g
F_{pga}	1.2
F_a	1.2
F_v	1.7
A_s	0.114 g
S_{DS} (Period = 0.2 sec)	0.222 g
S_{D1} (Period = 1.0 sec)	0.077 g

Per AASHTO Article 3.10.6, the site should be assigned to Seismic Zone 1 based on a calculated S_{D1} of 0.077 g. However, per AASHTO Article 4.7.4.2 seismic analysis is not required for single-span bridges, regardless of seismic zone.

The site was also evaluated for liquefaction potential during an earthquake. The term "liquefication" describes a phenomenon in which cohesionless soil experiences a substation reduction in effective stress during an earthquake and acquires a degree of temporary mobility sufficient to permit substantial settlement and/or loss of bearing capacity. Based on the relative density of the soils and their gradation, it is GZA's opinion that site soils are not liquefiable.

CONSTRUCTION CONSIDERATIONS

FOUNDATION SUBGRADE PREPARATION

As discussed above, GZA recommends that the existing sand or fill soils either be intensively surface compacted, or excavated and replaced with two feet of NHDOT Item 508.2.1, Crushed Gravel for Structural Fill, to provide suitable bearing surface for spread footing support of a temporary bridge. The bearing surfaces should be prepared in accordance with NHDOT Standard Specification Section 504.3.1 Preparation of a Foundation on Earth. The final excavation to bearing elevation should be performed with a smooth-edged bucket to limit disturbance to the subgrade.

Once excavation has been completed, the subgrade should be proof compacted with several passes of a vibratory roller or large plate compactor to densify soils disturbed by excavation, and provide a firm, stable subgrade. Areas exhibiting excessive weaving, soft, or unstable soils should be excavated and replaced with compacted NHDOT Item 508.2.1.1, Crushed Gravel for Structural Fill, or crushed stone wrapped in a geotextile filter fabric. Foundation subgrade preparation should be observed by a qualified geotechnical engineer to confirm that the exposed material is suitable for foundation support.

The contractor should be required to prevent freezing of subgrade soils prior to placement of fill or concrete. In the event that frost penetration occurs, the frozen soils should be removed and replaced with compacted NHDOT Item 508.2.1.1, Crushed Gravel for Structural Fill.

EXCAVATION, TEMPORARY LATERAL SUPPORT, AND DEWATERING

We anticipate that excavation for the proposed temporary abutments or permanent pile caps will encounter overburden soils consisting of sand or fill. Cobbles, boulders, and brick may be encountered within these deposits, but it is expected that these can be excavated using conventional earth moving equipment.

Temporary construction dewatering may be required to control groundwater inflow in excavations for construction of the abutment footings or pile caps. Where space and groundwater conditions permit, excavations may be achieved using sloped, open-cut techniques provided they comply with OSHA excavation safety requirements. It is anticipated that the inflow of groundwater and infiltration to excavations can be handled by open pumping from sumps installed at the bottom of excavations.

The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater should comply with all local, State, and federal regulations.

FILL MATERIAL AND PLACEMENT RECOMMENDATIONS

Backfill for proposed abutments or pile caps should consist of NHDOT Item 209.20x Granular Backfill (Bridge). Fill placed below footings or pile caps should consist of NHDOT Item 508.2.1.1, Crushed Gravel for Structural Fill. Fill should be placed systematically in horizontal layers not more than 6 inches in thickness prior to compaction. Where hand-guided compaction equipment, such as a small vibratory plate compactor is used, the loose lift thickness should not exceed six inches. The fill should be compacted to at least 98 percent of the maximum dry density determined in accordance with AASHTO T 99.

If crushed stone is used as a substitute for Granular Backfill (Bridge) or Crushed Gravel for Structural Fill, or where used for foundation drainage, then the crushed stone should be wrapped with filter fabric to limit migration of fine-grained soil particles into the crushed stone.

REUSE OF ON-SITE MATERIALS

Laboratory results included in **Appendix E** indicate that the existing on-site soils are likely not suitable for reuse as Item 209.20X Granular Backfill (Bridge) or Item 508.2.1.1, Crushed Gravel for Structural Fill. However, it is anticipated that the existing on-site soils may be used in embankment side slopes and other landscaped areas as needed on the project provided it is approved for use. GZA recommends that the excavated soils be stockpiled and assessed by a qualified geotechnical engineer to ensure conformance with the specifications prior to reuse.

CLOSING

We appreciate the opportunity to work with you on this project, and we would be pleased to work with you through design and construction. In the meantime, if you have any questions regarding the recommendations contained in this report or require additional information, please contact us.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Jennifer R. Baron
Project Manager

David G. Lamothe, P.E.
Associate Principal

Andrew R. Blaisdell
Consultant/Reviewer

JRB/DGL/ARB:jrb
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Attachments: Table 1 – Summary of GZA Test Borings
Figure 1 – Locus Plan
Figure 2 – Site and Subsurface Exploration Location Plan
Appendix A – Limitations
Appendix B – Previous Boring Logs
Appendix C – Results of Geophysical Survey
Appendix D – Test Boring Logs
Appendix E – Laboratory Test Results

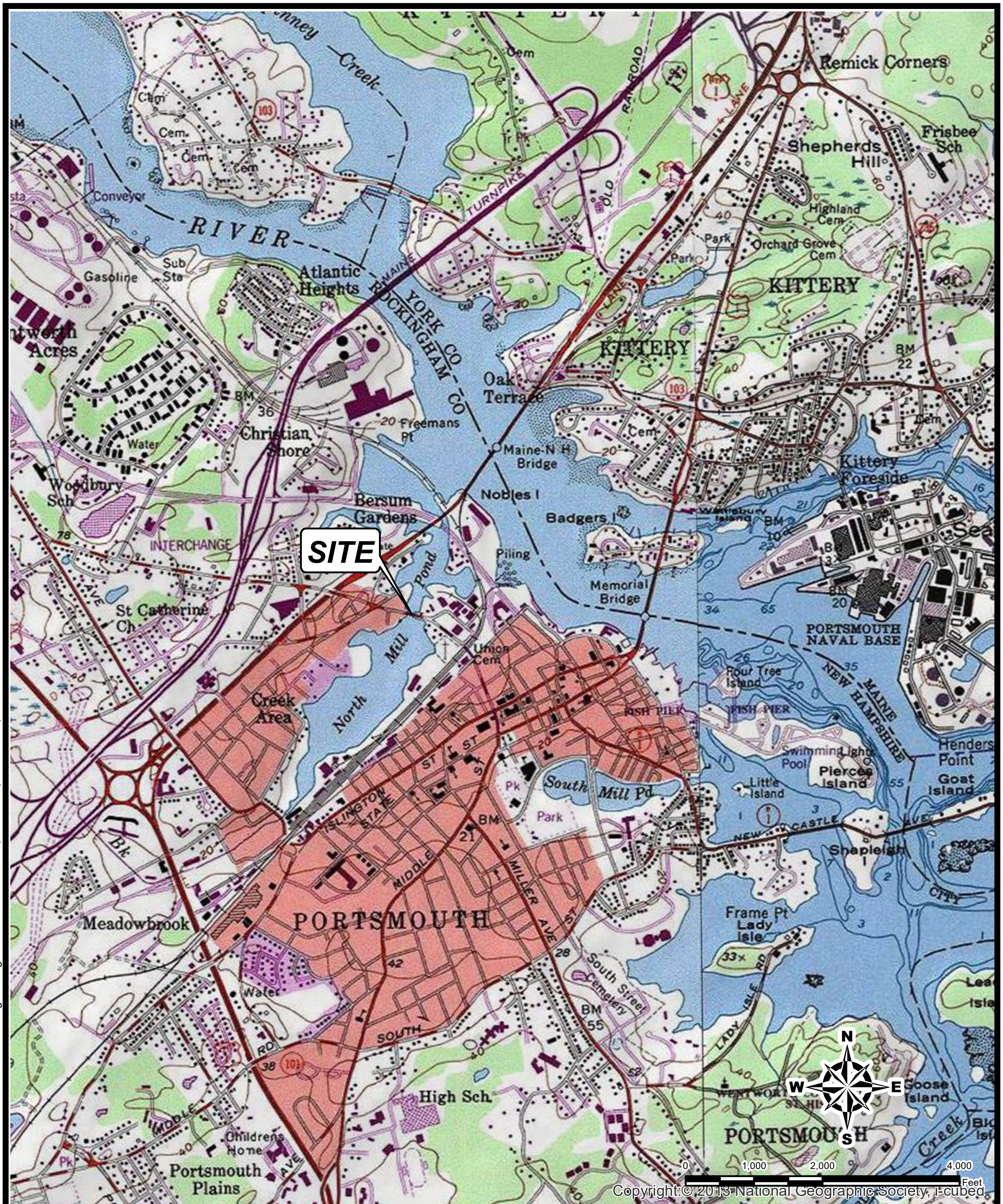
Table

TABLE 1
SUMMARY OF GZA TEST BORINGS
Maplewood Avenue Culvert
Portsmouth, New Hampshire

Boring Number	Boring Depth (ft)	Depth to Groundwater[3] (ft)	Approx. Thickness of Deposit[4] (ft)			Depth to Bedrock (ft)
			Asphalt	Fill	Cobbles, Boulders, and Brick	
B-101	27.0	6.8	0.9	12.8	1.6	15.3
B-102	23.0	8.3	1.0	7.8	12.2	21.0
B-103	34.0	9.0	1.0	18.0	4.5	23.5
B-104	32.5	9.4	0.9	17.0	3.0	20.9
B-105	26.0	NM	1.0	16.5	NE	17.5
B-106	30.0	8.5	1.2	16.8	NE	18.0

NOTES:
1. Refer to Appendix B for test boring logs.
2. "NE" indicates not encountered. "NM" indicates not measured.
3. Groundwater measurements recorded in the table were obtained during the drilling process and should not be considered stabilized.
4. The order that strata were encountered in the test borings may vary from the order shown on this table. Refer to the boring logs in Appendix D for detailed descriptions of the materials encountered at specific locations.

Figures



 P:\04Jobs\0191100s\04.0191113.00\Figures\Figure 1 - Locus Plan.mxd, 9/23/2020, 4:37:57 PM, marvin.revere

NO.	ISSUE / DESCRIPTION	BY	DATE

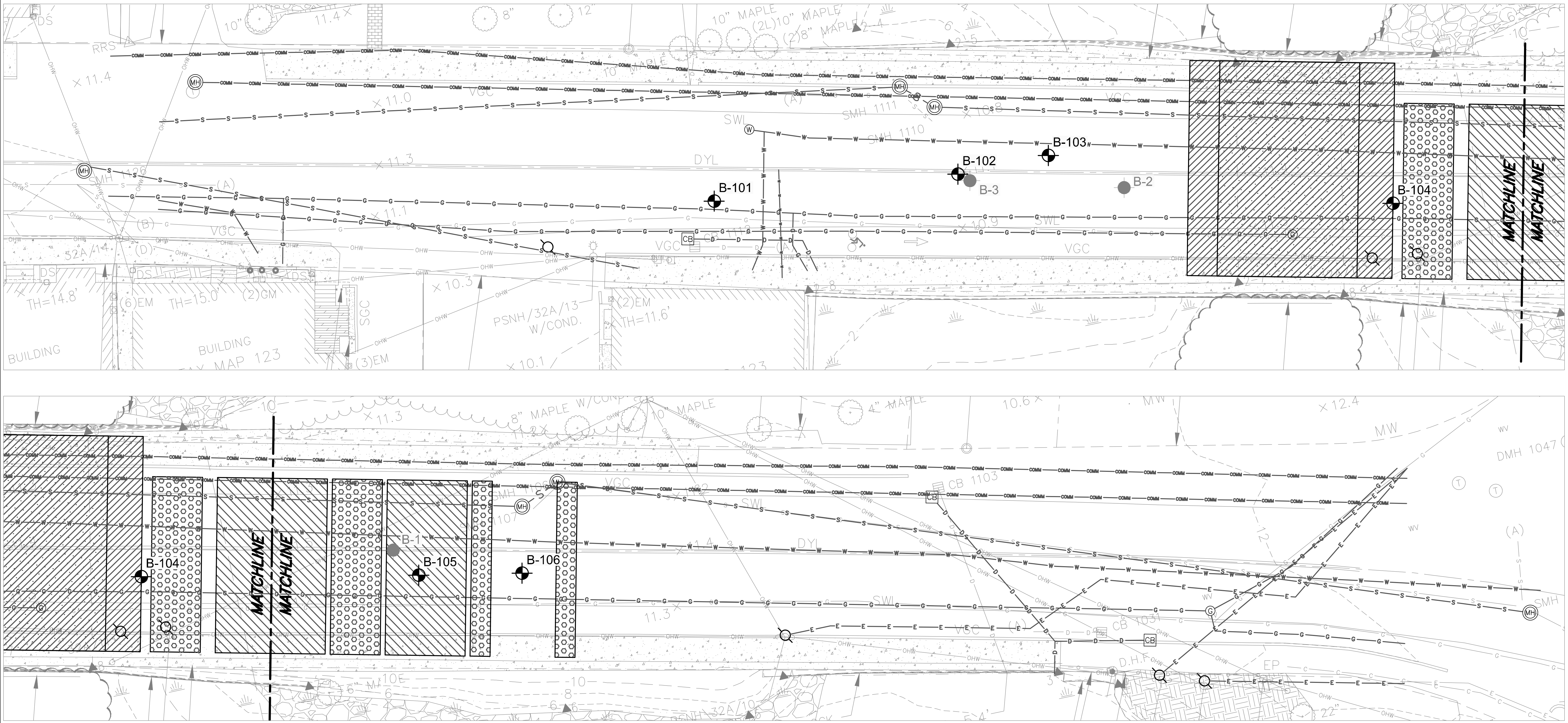
MAPLEWOOD AVENUE CULVERT REPLACEMENT
NHDOT BRIDGE No. 231/103
PORTSMOUTH, NEW HAMPSHIRE

PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
HOYLE, TANNER & ASSOCIATES, INC.

LOCUS PLAN

PROJ MGR: JRB	REVIEWED BY: CLS	CHECKED BY: DGL	FIGURE
DESIGNED BY: MR	DRAWN BY: MR	SCALE: 1 in = 2,000 ft	1
DATE: SEPTEMBER 2020	PROJECT NO. 04.0191113.00	REVISION NO.	



GENERAL NOTES

1. BASE MAP DEVELOPED FROM ELECTRONIC DRAWING FILE "905110ALI.DWG", PREPARED BY HTA AND TRANSMITTED TO GZA ON JULY 31, 2020 AND FILE "FIGURE2-3.DWG", PREPARED BY HAGER-RICHTER AND TRANSMITTED TO GZA ON SEPTEMBER 15, 2020.
2. THE LOCATION OF THE TEST BORINGS WERE APPROXIMATELY DETERMINED BY TAPE MEASUREMENTS FROM EXISTING SITE AND TOPOGRAPHIC FEATURES, BY GZA PERSONNEL.
3. BORINGS PERFORMED BY GZA WERE DRILLED BY NEW ENGLAND BORING CONTRACTORS BETWEEN AUGUST 4 AND 7, 2020 AND LOGGED BY GZA PERSONNEL.
4. BORINGS PREFORMED BY OTHERS WERE DRILLED BY JOHN TURNER CONSULTING OF DOVER, NEW HAMPSHIRE IN OCTOBER 2009.

LEGEND

B-101 BORINGS PERFORMED BY GZA
B-1 BORINGS PERFORMED BY OTHERS
GAS LINE
ELECTRIC LINE
WATER LINE
SEWER LINE
DRAIN LINE
COMMUNICATION LINE
WATER LINE–MARKED BY OTHERS
POSSIBLE FORMER CULVERT/ARCHES
POSSIBLE FORMER PIERS
PRESENT ARCH STRUCTURE
PRESENT ARCH OPENING
GAS VALVE
UTILITY POLE
WATER VALVE
MANHOLE
CATCH BASIN

0 10 20 40
SCALE 1"-20'-0"

N

MAPLEWOOD AVENUE CULVERT REPLACEMENT
NHDOT BRIDGE No. 231/103

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN

PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
HOYLE, TANNER & ASSOCIATES, INC.

PROJ MGR: JRB	REVIEWED BY: CLS	CHECKED BY: DGL	FIGURE
DESIGNED BY: JRB	DRAWN BY: MR	SCALE: 1" = 20'-0"	2
DATE: SEPTEMBER, 2020	PROJECT NO. 04.0191113.00	REVISION NO. 0	

Appendix A – Limitations

GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the agreement, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in GZA's Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.

3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

Subsurface Conditions

4. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs.

5. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

6. Water level readings have been made in test holes (as described in the Report) at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.

7. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

8. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

9. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Cost Estimates

10. Unless otherwise stated, our cost estimates are only for comparative and general planning purposes. These estimates may involve approximate quantity evaluations. Note that these quantity estimates are not intended to be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over either when the work will take place or the labor and material costs required to plan and execute the anticipated work, our cost estimates were made by relying on our experience, the experience of others, and other sources of readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

Additional Services

11. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

Appendix B – Previous Boring Logs

BORING LOG

JOHN TURNER CONSULTING, INC.
19 DOVER STREET
DOVER, NH 03820
PHONE: 603-749-1841
FAX: 603-516-6851

CLIENT:	Waterfront Engineers LLC	BORING #:	B1
PROJECT:	Maplewood Ave, Portsmouth, NH	LOCATION:	77' East of Beginning of Arch
		SURFACE ELEVATION:	
PROJECT NO:	09-GEO-047	DATE:	27-Oct-09

TYPE OF BORING:	SSA/ Switched to casing at 15'	GROUNDWATER OBSERVATIONS		
DRILLING CO:	Great Works Test Boring	DATE:	DEPTH:	TIME:
DRILLER:	Willie Aiken	27-Oct-09	10'	10:00am
JTC REP.:	Kyle Urso			

FT	NO.	SAMPLE DEPTH (FT.)	REC. (IN.)	SOIL & ROCK CLASSIFICATION-DESCRIPTION BURMEISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)	STRATUM CHANGE (FT.)	BLOWS PER 6 INCHES	PEN (N)
0				Asphalt	7"	27-50/4.5"	50+
	S-1	1-3	9"	Dark Brown, Dry, Silt, some sand, trace gravel			
				Dark Brown, Moist, Silt, some sand, trace gravel		3-4-3-4	7
	S-2	3-5	13"				
5	S-3	5-7	10"	Dark Brown, Moist, Silt, some sand, trace gravel		3-2-3-6	5
	S-4	7-9	14"	Dark Brown, Moist, Silt, some sand, trace gravel		4-4-2-3	6
10	S-5	10-12	7"	Dark Brown, Wet, Silt, some sand, trace gravel		7-2-4-4	6
				Drilled through coarse sand, with gravel	13'		
15	S-6	15-16.5'	6"	Brownish Gray, Wet, fine-coarse Sand, some gravel		11-35-12-50/3"	47
				Gray, Wet, Silt and fine Sand, trace gravel	16'		
				Auger Refusal @ 16.5'			

REMARKS:

Standard Penetration Tests (SPT) – 140# hammer falling 30" (ASTM D1586)
Blows are per 6 inches with a 24" long by 2" O.D. by 1 3/8" I.D. split spoon sampler unless otherwise noted
S = split-spoon sample; C = rock core sample; U = undisturbed

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated in the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.
Proportions used: trace (0-10%), little (10-20%), some (20-35%), and (35-50%)

BORING LOG

JOHN TURNER CONSULTING, INC.
19 DOVER STREET
DOVER, NH 03820
PHONE: 603-749-1841
FAX: 603-516-6851

CLIENT:	Waterfront Engineers LLC	BORING #:	B2 Page 1 of 2
PROJECT:	Maplewood Ave, Portsmouth, NH	LOCATION:	28' West of Culvert opening
		SURFACE ELEVATION:	
PROJECT NO:	09-GEO-047	DATE:	27-Oct-09
TYPE OF BORING:	SSA/ Switched to casing at 10'	GROUNDWATER OBSERVATIONS	
DRILLING CO:	Great Works Test Boring	DATE: / DEPTH: / TIME:	
DRILLER:	Willie Aiken	27-Oct-09 / None	Upon Completion
JTC REP.:	Kyle Urso		

FT.	NO.	SAMPLE DEPTH (FT.)	REC. (IN.)	SOIL & ROCK CLASSIFICATION-DESCRIPTION BURMEISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)	STRATUM CHANGE (FT.)	BLOWS PER 6 INCHES	PEN (N)
0	S-1	0-2	15"	Asphalt	7"	20-41-27-18	68
				Dark Brown, Moist, Silt, some sand, trace gravel			
	S-2	2-4	2"	Rock Restricted Recovery-trace silt and fractured rock		12-16-8-9	24
5	S-3	5-7	9"	Dark Brown, Moist, Silt, some sand, trace gravel		4-3-30-50/2"	33
	S-4	7-9	3"	Boulder at 7, could not drill through, moved 3' North			
				Dark Brown, Moist, Silt, some sand, trace gravel		6-4-7-9	11
10	S-5	10-12	11"	Dark Brown, Moist, Silt, some sand, trace gravel		3-2-1-3	3
				Gray silt, trace gravel			
				(Trace wood in tube, may have drilled through old wood tie			
				or buried stump-strong smell of organics)			
15	S-6	15-17	0"	No Recovery		1-2-1-1	3
	S-7	17-19	6"	Wet, Gray, Clay-strong organic smell	17	6-2-3-2	5
				Boring Incomplete-Time Restrictions			
				Continue 10/30			

REMARKS: We had no Officer to control traffic so drilling had to terminate for the day. We will continue drilling from where we left on Friday October 30, 2009

Standard Penetration Tests (SPT) = 140# hammer falling 30" (ASTM D1586)
Blows are per 6 inches with a 24" long by 2" O.D. by 1 3/8" I.D. split spoon sampler unless otherwise noted
S = split-spoon sample; C = rock core sample; U = undisturbed

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated in the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.
Proportions used: trace (0-10%), little (10-20%), some (20-35%), and (35-50%)

BORING LOG

JOHN TURNER CONSULTING, INC. PHONE: 603-749-1841
19 DOVER STREET FAX: 603-516-6851
DOVER, NH 03820

CLIENT:	Waterfront Engineers LLC	BORING #:	B2 Page 2 of 2
PROJECT:	Maplewood Ave, Portsmouth, NH	LOCATION:	31' West of Culvert opening
		SURFACE ELEVATION:	
PROJECT NO:	09-GEO-047	DATE:	30-Oct-09

		GROUNDWATER OBSERVATIONS		
TYPE OF BORING:	SSA/Switched to Casing @ 22'	DATE:	DEPTH:	TIME:
DRILLING CO:	Great Works Test Boring	30-Oct-09	20'	
DRILLER:	Willie Aiken			
JTC REP.:	Kyle Urso			

FT	NO.	SAMPLE DEPTH (FT.)	REC. (IN.)	SOIL & ROCK CLASSIFICATION-DESCRIPTION BURMEISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)	STRATUM CHANGE (FT.)	BLOWS PER 6 INCHES	PEN (N)
20	S-8	20-22	20"	Gray, Wet, Clay-strong smell of organics		3-3-9-12	12
	S-9	22-24	4"	Gray, Wet, Sand & Gravel with some clay		3-6-10-10	16
				Weathered Rock	24.5'		
25				Drilled through 4' of weathered Rock with tri-cone bit			
				Boring Terminated @ 28.5' (weathered rock)			

REMARKS:

Standard Penetration Tests (SPT) = 140# hammer falling 30" (ASTM D1586)
Blows are per 6 inches with a 24" long by 2" O.D. by 1 3/8" I.D. split spoon sampler unless otherwise noted
S = split-spoon sample; C = rock core sample; U = undisturbed

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated in the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.
Proportions used: trace (0-10%), little (10-20%), some (20-35%), and (35-50%)

BORING LOG

JOHN TURNER CONSULTING, INC.
19 DOVER STREET
DOVER, NH 03820
PHONE: 603-749-1841
FAX: 603-516-6851

CLIENT:	Waterfront Engineers LLC	BORING #:	B3
PROJECT:	Maplewood Ave, Portsmouth, NH	LOCATION:	60' West of Culvert opening
		SURFACE ELEVATION:	
PROJECT NO:	09-GEO-047	DATE:	30-Oct-09

		GROUNDWATER OBSERVATIONS		
TYPE OF BORING:	SSA/Switched to Casing @ 15'	DATE:	DEPTH:	TIME:
DRILLING CO:	Great Works Test Boring	30-Oct-09	6'	10:15am
DRILLER:	Willie Aiken			
JTC REP.:	Kyle Urso			

FT	NO.	SAMPLE DEPTH (FT.)	REC. (IN.)	SOIL & ROCK CLASSIFICATION-DESCRIPTION BURMEISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)	STRATUM CHANGE (FT.)	BLOWS PER 6 INCHES	PEN (N)
0				Asphalt	7"		
	S-1	1-3	5"	Dark Brown, Moist silt and sand, some gravel		50/5"	50+
				Drilled through Cobbles			
		3-5	12"	Dark Brown, Moist Silt and Sand, some gravel, trace weathered rock		9-16-12-5	28
5	S-2	5-7	8"	Dark Brown, Moist, Silt and Sand, some gravel		3-4-4-5	8
	S-3	7-9	18"	Dark Brown, Wet Silt and Sand, some gravel	8'	5-5-5-6	10
				Brownish Orange, Wet, fine-coarse Sand			
10	S-4	10-12	15"	Brownish Orange, Wet, fine-coarse Sand		10-14-10-11	24
		12-14	3"	Brownish Orange Wet, Fine Sand and Gravel, little silt		12-7-5-4	12
					14.5'		
15	S-5	15-17	14"	Gray, Wet, Clay with some sand and gravel-strong smell of organics		1-1-2-5	3
	S-6	17-19	7"	Gray, Wet, Clay with and and gravel-strong smell of organics	18'	12-50/5"	50+
				Weathered Rock			
				Drilled through 3' of weathered rock with tri-cone bit			
				Boring Terminated @ 21.0' (weathered rock)			

REMARKS:

Standard Penetration Tests (SPT) = 140# hammer falling 30" (ASTM D1586)
Blows are per 6 inches with a 24" long by 2" O.D. by 1 3/8" I.D. split spoon sampler unless otherwise noted
S = split-spoon sample; C = rock core sample; U = undisturbed

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated in the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made. Proportions used: trace (0-10%), little (10-20%), some (20-35%), and (35-50%)

Appendix C - Results of Geophysical Survey

HAGER-RICHTER
GEOSCIENCE, INC.

GEOPHYSICAL SURVEY
MAPLEWOOD AVENUE BRIDGE
PORTSMOUTH, NEW HAMPSHIRE

Prepared for:

GZA GeoEnvironmental, Inc.
5 Commerce Park North, Suite 201
Bedford, New Hampshire 03110-6984

Prepared by:

Hager-Richter Geoscience, Inc.
8 Industrial Way - D10
Salem, New Hampshire 03079

File 19J82
August 2020

August 6, 2020
File 19J82

Jennifer R. Baron, P.E.
Project Manager
GZA GeoEnvironmental, Inc.
5 Commerce Park North, Suite 201
Bedford, New Hampshire 03110

Dir: 603.232.8758
Cell: 207.232.5832
Email: jennifer.baron@gza.com

RE: Geophysical Survey
Maplewood Avenue Bridge
Portsmouth, New Hampshire

Dear Ms. Baron:

In this report, we summarize the results of a geophysical survey conducted by Hager-Richter Geoscience, Inc. (HRGS) at the above referenced site in Portsmouth, New Hampshire for GZA GeoEnvironmental, Inc. (GZA) in July 2020. The scope of the survey and area of interest were specified by GZA.

INTRODUCTION

The site is the Maplewood Avenue Bridge that crosses North Mill Pond in Portsmouth, New Hampshire. The general location of the site is shown in Figure 1. As part of a geotechnical investigation of the bridge and approaches, GZA requested a geophysical survey to: 1) detect possible subsurface utilities prior to the installation of borings; 2) detect possible culverts/arches and piers that may have been buried or filled when the present bridge was built; and 3) determine the shear wave velocity of the subsurface. According to information provided by GZA, four old culvert/arches were present in the structure that was replaced in 1896. The arch structures may have been left in place when they were filled.

The area of interest (AOI) specified by GZA measured approximately 500 feet by 45 feet and includes the sidewalks and active roadway of Maplewood Avenue in the vicinity of the bridge over North Mill Pond. The ground surface in the AOI was asphalt pavement with concrete sidewalks. The approximate limits of the AOI are shown in Figure 2.

The locations of detected utilities and possible structures were shown on a preliminary map of findings which was transmitted to GZA on July 24, 2020 by email.

OBJECTIVES

The objectives of the geophysical survey were 1) to detect, and if detected, to locate subsurface utilities in the accessible portions of a specified area of interest; 2) to detect, and if detected, to locate possible culverts/arches and piers that may have been buried or filled when the present bridge was built; and 3) to determine the shear wave velocity of the subsurface along the bridge approaches.

THE SURVEY

Michael Howley, P.G., Bryan Carnahan, and Sean Reid of HRGS conducted the geophysical survey on July 14 and 15, 2020. The project was coordinated with Ms. Jennifer Baron, P.E., of GZA. Mr. Joshua Szmyt, also of GZA, was present for the duration of the field work and specified the limits of the AOI for the survey.

The geophysical survey was conducted using multiple geophysical methods to accomplish the objectives of the survey. The time domain electromagnetic induction metal detection (EM61), ground penetrating radar (GPR), and precision utility location (PUL) methods were used to map and locate subsurface utilities and other structures of interest. The multi-channel analysis of surface waves (MASW) method was used to develop shear wave velocity profiles, and the passive shear wave velocity (pVs) method to determine 1-D vertical profiles of shear wave velocity as a function of depth for the midpoint of the seismic lines. Figure 2 is a Site Plan provided by GZA that shows the area of the EM61, GPR, and PUL survey, along with the locations of the MASW and pVs lines.

Subsurface Utilities and Structures Survey.

The EM61 data were acquired at approximately 8-inch intervals along lines spaced 5 feet apart across the accessible portions of the specified area of interest. The EM61 survey detects buried metal. However, the EM method cannot provide information on the type of objects causing an EM anomaly.

GPR data were acquired along traverses oriented in two mutually perpendicular directions, with lines spaced 5 feet apart oriented parallel to the roadway, and lines spaced 10 feet apart oriented perpendicular to the roadway. The GPR method is capable of detecting both metal and nonmetal objects.

The PUL method was used to search for subsurface utilities in the AOI by passively searching for signals from active electric lines and by actively tracing signals applied by direct connections to accessible utility structures such as conduits, valves, and other exposed pipes or conduits.

A local survey grid was established in the AOI for the acquisition of the geophysical data. The locations of utilities detected at the time of the survey were marked on site and their locations were recorded for inclusion on the site plan. The geophysical data were reviewed in the office and additional utility segments and other structures were identified, and their locations are shown on the plan included in this report.

Shear Wave Velocity Surveys.

MASW and pVs data were acquired along four (4) lines totaling 940 linear feet. The MASW and pVs surveys were both conducted using 48 4.5-Hz geophones and a geophone spacing of 5 feet. The energy source for the MASW survey was a 12-lb sledgehammer striking the pavement. The MASW method produces a shear-wave velocity profile along a portion of the survey lines. The pVs method, also called the Refraction Microtremor (ReMi™) method, uses ambient noise rather than an active noise source. The pVs method yields a single vertical velocity profile at the mid points of the test lines, shown in Figure 2. The seismic source for the pVs test was ambient noise and random hammer striking while acquiring the data to enhance the high frequency content of the seismic signal. The locations of the MASW and pVs lines were tied to permanent site features identified on a site plan provided by GZA and are shown in Figure 2.

METHODS AND EQUIPMENT

EM61. The EM survey was conducted using a Geonics EM61-MK2 time domain electromagnetic induction metal detector. The EM61-MK2 instrument was designed specifically for detecting buried metal objects such as utilities, underground storage tanks (USTs), and drums. An air-cored transmitter coil generates a pulsed primary magnetic field in the earth, thereby inducing eddy currents in nearby metal objects. The eddy current produces a secondary magnetic field that is sensed by two receiver coils, one coincident with the transmitter and one positioned 40 cm above the main coil. By measuring the secondary magnetic field after the current in the ground has dissipated but before the current in metal objects has dissipated, the instrument responds only to the secondary magnetic field produced by metal objects. Four channels of secondary response are measured in mV and are recorded on a digital data logger. The system is generally operated by pushing the coils configured as a wagon with an odometer mounted on the axle to trigger the data logger automatically at approximately 8-inch intervals.

GPR. The GPR survey was conducted using a GSSI SIR 4000 digital subsurface imaging radar system. The system includes a survey wheel that triggers the recording of the data at fixed intervals, thereby ensuring the accuracy of the features detected along the survey lines. The system was used with both a dual-frequency (DF) 800 MHz and 300 MHz antenna and a 350 MHz Hyperstacking (HS) antenna. Data were recorded using 43 and 86 ns[1] time windows for the DF antenna and a 95 ns time window for the HS antenna.

GPR uses a high-frequency electromagnetic pulse (referred to herein as "radar signal") transmitted from a radar antenna to probe the subsurface. The transmitted radar signals are reflected from subsurface interfaces of materials with contrasting electrical properties. Travel times of the

[1] ns, abbreviation for nanosecond, 1/1,000,000,000 second. Light and the GPR signal require about 1 ns to travel 1 ft in air. The GPR signal requires about 3.5 ns to travel 1 ft in unsaturated sandy soil.

radar signal can be converted to approximate depth below the surface by correlation with targets of known depths and by a curve matching routine. We monitor the acquisition of GPR data in the field and record the GPR data digitally for subsequent processing. Interpretation of the records is based on the nature and intensity of the reflected signals and on the resulting patterns.
Data from the GPR survey were processed using RADAN 7.6 GPR processing software from Geophysical Survey Systems, Inc. We reviewed profile images of the GPR data. Interpretation of the records is based on the nature and intensity of the reflected signals and on the resulting patterns.

PUL. The PUL survey was conducted using a Radiodetection RD 8000 series PUL instrument. The RD 8000 series consists of separate transmitter and receiver. The system can be used in "passive" and "active" modes to locate buried pipes by detecting electromagnetic signals carried by the pipes. In the "passive" mode, only the receiver unit is used to detect signals carried by the pipe from nearby power lines, live signals transmitted along underground power cables, or very low frequency radio signals resulting from long wave radio transmissions that flow along buried conductors. In the "active" mode of operation, the transmitter is used to induce a signal on a target pipe, and the receiver is used to trace the signal along the length of the pipe. Our system uses a 10W transmitter.

MASW. The multichannel analysis of surface waves (MASW) method is a seismic method that determines a shear-wave velocity (Vs) profile (i.e., Vs versus depth and horizontal distance) by analyzing a particular type of seismic wave on a multichannel record. The MASW method uses Rayleigh waves, which are elastic waves that travel in the subsurface near the earth's surface. The amplitude of such waves decreases with depth and the phase velocity of the waves is a function of frequency. The method uses multichannel recording and processing concepts widely used in reflection surveying by the oil and gas industry.

The MASW method requires multichannel records with at least 12 traces to produce reliable results. We use 48 channels (two 24-channel Geometrics Geode digital seismographs), coupled to 48 geophones to acquire 24-trace records. The data acquired for geophones numbered 1 - 24 are processed as discussed below to determine the shear wave velocity as a function of depth for discrete layers, and the velocity of each layer Vs(x,n) is assigned to the midpoint of the line between Stations 1 and 24, i.e. x = 55 ft if the geophone spacing is 5 feet. The data acquired for geophones numbered 2 through 25 yield the vertical velocity profile at the midpoint of the line between stations 2 and 25, i.e. x = 60 ft if the geophone spacing is 5 feet. By processing the data for geophones m through m+24 and assigning the vertical profiles to the midpoints, the velocity of each layer is generated as a function of horizontal distance. The end point for the velocity determined with a 48-geophone spread using data acquired with 24 geophones is located at x = 175 ft from the start of the line if a 5-foot geophone spacing is used.

The MASW survey is conducted using an active source, and the method using an active source is sometimes called an active MASW survey to distinguish it from a passive MASW survey in which ambient noise is used as the source. Levels of ambient noise are monitored in real time during data acquisition. Ambient noise is not utilized by the survey but is avoided by waiting for times when nearby traffic (the main source of ambient noise) is not adversely affecting the quality of the data. Only active source data were used for the subject survey and no passive source data were acquired. It is also important to use a low natural frequency geophone for most MASW surveys.

The surface waves used in MASW, considered noise in refraction and reflection surveys, are enhanced during data acquisition and processing for the MASW method. The seismic data are analyzed using Surfseis 6.2, a commercially licensed software package developed by the Kansas Geological Survey. Briefly, SurfSeis provides a dispersion curve from which the interpreter selects the fundamental mode in detail, and the software then inverts the dispersion curve in terms of a model of shear wave velocity (Vs) as a function depth at the midpoint of the geophone spread. Results can be presented as 2-D graphical plots of the shear wave velocity as a function of depth and distance along the line using contouring software such as Surfer or in tabular form showing shear wave velocity as a function of depth at a given station.

As discussed above, data are acquired for 24 channels at a time and the resulting 1-D shear wave distribution as a function of depth is assigned the horizontal position at the center of the 24-channel spread. The 1-D distributions are then combined to provide shear wave velocity distribution across the survey line and are presented as 2-D color plots. The variations in color correspond to apparent variations in subsurface shear wave velocity. Low shear wave velocities correlate with softer soils and higher shear wave velocities correlate with harder, more dense soil or bedrock.

pVs. As indicated above, the passive shear wave seismic (pVs) method, also called the Refraction Microtremor method, or ReMi™ was used to determine the shear wave velocity as a function of depth. The passive shear wave seismic (pVs) method is a geophysical method to determine a vertical shear-wave velocity profile at a single location by analyzing a particular type of seismic wave recorded on a multichannel record. The name pVs is derived from p for passive and Vs for velocity of shear waves. The pVs method, also called the Refraction Microtremor method, or ReMi™, uses Rayleigh waves, a particular kind of wave first described by Lord Rayleigh in 1885. Such waves are dispersive (meaning that the velocity is a function of the wavelength), and the amplitude of such waves decreases with depth. The Rayleigh wave velocity depends primarily on the shear wave velocities and layering of the subsurface material.

Rayleigh waves are a significant part of the ambient subsurface noise at most, if not all, sites. There are many sources of such noise, including, but not limited to, wind, pedestrian and vehicular traffic, surface and subway trains, and construction activities. Although such noise can be troublesome for most seismic methods, it is the source of signals for the pVs method, and the higher the noise level, the better the results for this method.

Low frequency (4.5 Hz) geophones are installed 5 ft apart along a straight line and connected to a seismograph. The ambient noise is recorded for 30 seconds two or three times and examined to be sure that noise of sufficiently low frequency is present. If the noise is sufficient, then 15 to 20 such records are acquired. If the noise spectra do not reach sufficiently low frequencies, then one walks or runs along the test line during data acquisition to add low frequency noise to the ambient noise.

The surface waves used in the pVs method, considered noise in seismic refraction and reflection surveys, are enhanced during data acquisition and processing for the pVs method. The seismic data are analyzed using SeisOpt® ReMi™, a commercially licensed software package developed by Optim, Inc. located at the University of Nevada at Reno. Results are normally presented as 1-D plots or in tabular form showing shear wave velocity as a function of depth at the center of the seismic line.

It should be noted that the method produces a single velocity profile (Vs as a function of depth Z) at one location (namely, the center of the line) for each line. The software also calculates the average shear wave velocity using the following equation (taken from the International Building Code):

$$V_{avg} = \left(\sum_{i=1}^{N} d_i\right) / \sum_{i=1}^{N} d_i / V_i \qquad \text{Eq. 1}$$

where V_{avg} is average shear wave velocity
d_i is thickness of the ith layer
V_i is the shear wave velocity of the ith layer
N is the number of layers

The Seismic Site Class, based solely on average shear wave velocity, is defined by the IBC as follows:

Site Class	Soil Profile Name	Soil Shear Wave Velocity (ft/s)
A	Hard rock	$V_s > 5000$
B	Rock	$2500 < V_s \leq 5000$
C	Very dense soil and soft rock	$1200 < V_s \leq 2500$
D	Stiff soil profile	$600 \leq V_s \leq 1200$
E	Soft soil profile	$V_s < 600$

Although the IBC provides other methods to determine the Site Class, such as standard penetration resistance (N-values) and soil undrained shear strength, this report provides site specific data for shear wave velocity only. Furthermore, there is no consideration of other factors that may affect a site such as liquefaction. **The final determination of seismic site class should be made by the project engineer.**

LIMITATIONS OF THE METHODS

HRGS MAKES NO GUARANTEE THAT ALL TARGETS WERE DETECTED IN THIS SURVEY. HRGS IS NOT RESPONSIBLE FOR DETECTING TARGETS THAT CANNOT BE DETECTED BY THE METHODS EMPLOYED OR BECAUSE OF SITE CONDITIONS. GPR SIGNAL PENETRATION MIGHT NOT BE SUFFICIENT TO DETECT ALL TARGETS.

Field mark-outs. Utilities detected by the PUL method at the time of the survey are marked in the field. Adverse weather and site conditions (rain, uneven surfaces, high traffic, etc.) can hamper in-field interpretation. Mark-outs made on wet pavement, sand or gravel surfaces, or in active construction zones may not last. HRGS is not responsible for maintaining utility mark-outs after leaving the work area.

EM61. The EM61 cannot detect non-metallic objects. The data from an EM61 survey are adversely affected by surface metal. The EM61 has a depth sensitivity limited to about 12 feet. The instrument is relatively cumbersome and works best where the transmit and receive coils can be hand pushed in a small wagon.

Detection and identification should be clearly differentiated. Detection is the recognition of the presence of a metal object, and the electromagnetic method is excellent for such purposes. Identification, on the other hand, is determination of the nature of the causative body (i.e., what is the body -- a cache of drums, UST, automobile, white goods, etc.?). Although the EM data cannot be used to identify all buried metal objects, they provide excellent guides to the identification of some objects. For example, buried metal utilities produce anomalies with lengths many times their widths.

GPR. There are limitations of the GPR technique as used to detect and/or locate targets such as those of the objectives of this survey. Limitations include: (1) surface conditions, (2) electrical conductivity of the ground, (3) contrast of the electrical properties of the target and the surrounding soil, and (4) spacing of the traverses. Of these restrictions, only the last is controllable by us.

The condition of the ground surface can affect the quality of the GPR data and the depth of penetration of the GPR signal. Sites covered with high grass, bushes, landscape structures, debris, obstacles, soil mounds, etc. limit the survey access and the coupling of the GPR antenna with the ground. In many cases, the GPR signal will not penetrate below concrete pavement, especially inside buildings, and a target may not be detectable. The GPR method also commonly does not provide useful data under canopies found at some facilities.

The electrical conductivity of the ground determines the attenuation of the GPR signal and thereby limits the maximum depth of exploration. For example, the GPR signal does not penetrate clay-rich soils, and targets buried in clay might not be detected.

A definite contrast in the electrical conductivities of the surrounding ground and the target material is required to obtain a reflection of the GPR signal. If the contrast is too small, possibly due to construction details or deeply corroded metal in the target, then the reflection may be too weak to recognize, and the target can be missed.

Spacing of the traverses is limited by access at many sites, but where flexibility of traverse spacing is possible, the spacing is adjusted to the size of the target. The GPR operator controls the spacing between lines, and the design of the survey is based on the dimensions of the smallest feature of interest. Targets with dimensions smaller than the spacing between GPR survey lines can be missed.

PUL. The PUL equipment cannot detect non-metallic utilities, such as pipes constructed of vitrified clay, transite, plastic, PVC, and unreinforced concrete, when used in passive mode alone. Such pipes can be detected if a wire tracer is installed with access to such tracer for transmission of a signal or where access (such as floor drains and clean-outs) permits insertion of a device on which a signal can be transmitted. In some, but not all cases, the subsurface utility designation equipment cannot detect metal utilities reliably under reinforced concrete because the signal couples onto the metal reinforcing in the concrete. Similarly, the method commonly cannot be used adjacent to grounded metal structures such as chain link fences and metal guardrails. In congested areas, where several utilities are bundled or located within a short distance of each other, the signal transmitted on one utility can couple onto adjacent utilities, and the accuracy of the location indicated by the instrument decreases.

MASW and pVs. As with all physical measurements, there is experimental error in the velocities that are determined using the MASW and pVs methods. The uncertainty in velocity of shear waves is estimated to be approximately 10-15%. For the pVs method, the accuracy of V_{avg} is stated by Optim, Inc. to be 5-15%.

The seismic survey lines must be straight, or nearly so, and cannot pass over excavated areas. Obviously, they also cannot normally extend through existing buildings or concrete walls.

The depth of investigation is a function of the noise spectrum, and long wavelengths (low frequencies) are required to determine velocity at large depths. Noise levels can be improved by a person running along the seismic spread during data acquisition.

RESULTS

General. The geophysical survey was conducted using the EM61, GPR, and PUL methods across the accessible portions of the area of interest (AOI) specified by GZA to detect subsurface utilities and other structures of interest. The multi-channel analysis of surface waves (MASW) and passive shear wave velocity (pVs) methods were used to develop shear wave velocity profiles and 1-D vertical profiles of shear wave velocity as a function of depth for the midpoint of the seismic lines, respectively. Figure 2 is a Site Plan provided by GZA that shows the area of the EM61, GPR, and PUL survey, along with the locations of the MASW lines and the pVs lines. Figure 3 shows the locations of the GPR lines along with the integrated interpretation of the EM61, GPR and PUL surveys. Figure 4 shows the MASW profiles as color contour profile plots of shear wave velocity.

EM61/GPR/PUL Survey Integrated Interpretation. The EM61, GPR, and PUL survey was conducted in an AOI specified by GZA measuring 500 feet by 45 feet. EM61 data were acquired at approximately 8-inch intervals along lines spaced 5 feet apart oriented parallel to the roadway. GPR data were acquired along traverses oriented in two mutually perpendicular directions, with lines spaced 5 feet apart oriented parallel to the roadway, and lines spaced 10 feet apart oriented perpendicular to the roadway with the DF antenna, and along lines spaced 5 feet apart oriented parallel to the roadway with the HS antenna. The PUL method was used to search for subsurface utilities in the AOI by passively searching for signals from active electric lines and by actively tracing signals applied by direct connections to accessible utility structures such as conduits, valves, and other exposed pipes or conduits.

Apparent GPR signal penetration was good along the edges of the roadway and poor to fair near the middle of the roadway. For the DF antenna, two-way traveltime reflections were received for 20 to 30 ns of the 86 ns time window acquired for the 300 MHz antenna and for 15 to 20 ns of the 43 ns time window acquired for the 800 MHz antenna. Based upon site-specific velocity matching calibrations, the GPR signal penetration in the area of interest with the DF antenna is estimated to have been about 3.5 to 5 feet below ground surface. For the HS antenna, two-way traveltime reflections were received for 40 to 60 ns of the 93 ns time window acquired. Based upon site-specific velocity matching calibrations, the GPR signal penetration in the area of interest with the HS antenna is estimated to have been about 6 to 8 feet below ground surface.

Electric lines, gas lines, water lines, and communication lines were detected using the PUL method at the time of the survey. Their locations were marked in the field at the time of the survey and are shown in Figure 3. Sewer lines and drain lines were detected in an office review of the data and are also shown in Figure 3.

GPR reflections attributed to four possible former piers and two possible former culverts/arches are evident in GPR records acquired with the HS antenna. The possible former piers and possible former culverts/arches are evident at depths of 2 to 3 and 4 to 5 feet below the asphalt pavement, respectively. The locations and approximate extents of the possible former piers and former culverts/arches are shown in Figure 3.

Shear Wave Velocity Surveys.

MASW. The results of the MASW survey are shown in profile format as color contour plots of shear wave velocity in Figure 4. The MASW method determines the spatial variation of shear wave velocity along the transects. In general, lower shear wave velocities indicate softer soils while higher shear wave velocities indicate more dense materials. Please note that due the acquisition parameters for the MASW arrays, as explained in the Methods and Equipment section, results cannot be determined for the beginning and end portions of each MASW Line, as indicated in Figure 3. Processed data for MASW Line 4 could be extended longer than the other three lines due to better quality data between distance marks 260 and 300 feet along the profile, as shone in Figure 4. The depth of the MASW profiles is determined by the data quality and varies between lines, as seen in Figure 4.

The color contour profiles of shear wave velocity shown in Figure 4 exhibit somewhat similar patterns for the four MASW lines, with an upper layer of low to moderate shear wave velocities (white, blue, and green) and a bottom layer of high shear wave velocity (green/yellow to red). Regions of relatively low shear wave velocity (<1200 feet/second) within about 10 to 15 feet of the surface are interpreted to be zones of fill and overburden soils (silt, sand, gravel, or till). A small area of relatively high shear wave velocity at the ground surface at the east end of Line 4 is interpreted to be due to the presence of a retaining wall as indicated in Figure 4.

The possible soil/bedrock interface is indicated in Figure 4 by a dashed line below which greater shear wave velocities (>1,200 feet/second) were detected along the MASW lines. Variations of the shear wave velocity below the interpreted bedrock surface are also evident and may be related to weathering or fracture zones.

pVs. The passive shear wave seismic (pVs), also called the Refraction Microtremor or ReMi™, testing was conducted along the same four (4) test lines as the MASW survey, designated as MASW/ReMi Lines 1 through 4. The seismic test line locations and center points for the velocity profiles are shown in Figure 2. Boring logs for test borings were not yet available for correlating stratigraphic units with shear wave velocities.

The results of the pVs testing are reported in Table 1. Note that bedrock depths reported in Table 1 correlate well with the interpreted bedrock depth using the MASW method. For modeling purposes, the subsurface stratigraphy was broken into three discrete units. The velocity units do not necessarily correlate with specific lithologic units, the number of layers and the thickness that provides the best statistical fit to the respective dispersion curve was used for each line independently. The root mean square error for the fit of the dispersion curve versus the measured data using the model velocities was 6.6%, 5.8%, 5.4%, and 6.8% for ReMi Lines 1 through 4, respectively. The velocities determined for the lithological units vary somewhat between lines, likely due to the discontinuous stratigraphic layers or lithological variations.

No attempt was made to "force" a specific model to the data. The velocities for the units to the maximum depth investigated, and the average values of the velocity of shear waves, Vs100, determined by Equation 1 for the depth interval of 0 – 100 ft are also reported in Table 1.

CONCLUSIONS

Based upon the utility location survey conducted by HRGS at Maplewood Avenue Bridge that crosses North Mill Pond in Portsmouth, New Hampshire for GZA GeoEnvironmental, Inc. (GZA) in July 2020, we conclude:

- Electric lines, communication lines, water lines, gas lines, sewer lines and drain lines were detected in the area of interest.
- Four possible former piers and two possible former culverts/arches were detected within the area of interest.
- Shear wave velocity profiles were developed along portions of the four seismic testing lines.
- The average values of the velocity of shear waves, Vs100, was determined for four points along the seismic testing lines.

LIMITATIONS ON USE OF THIS REPORT

This letter report was prepared for the exclusive use of GZA GeoEnvironmental, Inc. (Client). No other party shall be entitled to rely on this Report, or any information, documents, records, data, interpretations, advice, or opinions given to Client by Hager-Richter Geoscience, Inc. (HRGS) in the performance of its work. The Report relates solely to the specific project for which HRGS has been retained and shall not be used or relied upon by Client or any third party for any variation or extension of this project, any other project or any other purpose without the express written permission of HRGS. Any unpermitted use by Client or any third party shall be at Client's or such third party's own risk and without any liability to HRGS.

HRGS has used reasonable care, skill, competence and judgment in the performance of' its services for this project consistent with professional standards for those providing similar services at the same time, in the same locale, and under like circumstances. Unless otherwise stated, the work performed by HRGS should be understood to be exploratory and interpretational in character and any results, findings or recommendations contained in this Report or resulting from the work proposed may include decisions which are judgmental in nature and not necessarily based solely on pure science or engineering. It should be noted that our conclusions might be modified if subsurface conditions were better delineated with additional subsurface exploration including, but not limited to, test pits, soil borings with collection of soil and water samples, and laboratory testing.

Except as expressly provided in this limitations section, HRGS makes no other representation or warranty of any kind whatsoever, oral or written, expressed or implied; and all implied warranties of merchantability and fitness for a particular purpose, are hereby disclaimed. If you have any questions or comments on this letter report, please contact us at your convenience. It has been a pleasure to work with GZA GeoEnvironmental, Inc. on this project. We look forward to working with you again in the future.

Sincerely,
HAGER-RICHTER GEOSCIENCE, INC.

Michael Howley

Michael Howley, P.G.
Geophysicist

Jeffrey Reid, P.G.
Owner / Principal Geophysicist

Attachments: Table 1, Figures 1 – 4

TABLE 1 - pVs TESTING RESULTS

Geologic Unit*	pVs Test Line 1	
	Depth Interval (ft)	Vs** (ft/s)
Asphalt Pavement, Road-Base, and Fill	0 – 4.5	395
Possible Sand or Till	4.5 – 12	845
Probable Bedrock	12 - 100	4,382
$V_{S\ 100}$ (ft/s)	2,478	
RMS Error (%)	6.6	
IBC Site Class	C	

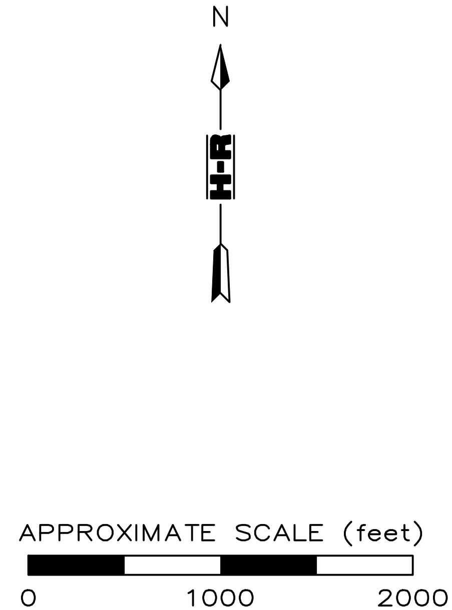
Geologic Unit* Based on Boring B-2	pVs Test Line 2	
	Depth Interval (ft)	Vs** (ft/s)
Asphalt Pavement, Road-Base, and Fill	0 – 4	477
Possible Sand or Till	4 – 14.8	737
Probable Bedrock	14.8 - 100	3,771
$V_{S\ 100}$ (ft/s)	2,193	
RMS Error (%)	5.8	
IBC Site Class	C	

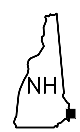
Geologic Unit* Based on Boring B-2	pVs Test Line 3	
	Depth Interval (ft)	Vs** (ft/s)
Asphalt Pavement, Road-Base, and Fill	0 – 4	501
Possible Sand or Till	4 – 14.8	674
Probable Bedrock	14.8 - 100	4,100
$V_{S\ 100}$ (ft/s)	2,236	
RMS Error (%)	5.4	
IBC Site Class	C	

TABLE 1 Continued

Geologic Unit* Based on Boring B-2	pVs Test Line 4	
	Depth Interval (ft)	Vs** (ft/s)
Asphalt Pavement, Road-Base, and Fill	0 – 4	376
Possible Sand or Till	4 – 10	716
Probable Bedrock	10 - 100	3,431
$V_{S\ 100}$ (ft/s)	2,210	
RMS Error (%)	6.8	
IBC Site Class***	C	

* Geologic Unit is inferred based on shear wave velocity.
** Shear wave velocity profile is determined for the mid-point of the test line
***. IBC 2000, 1615.1.1





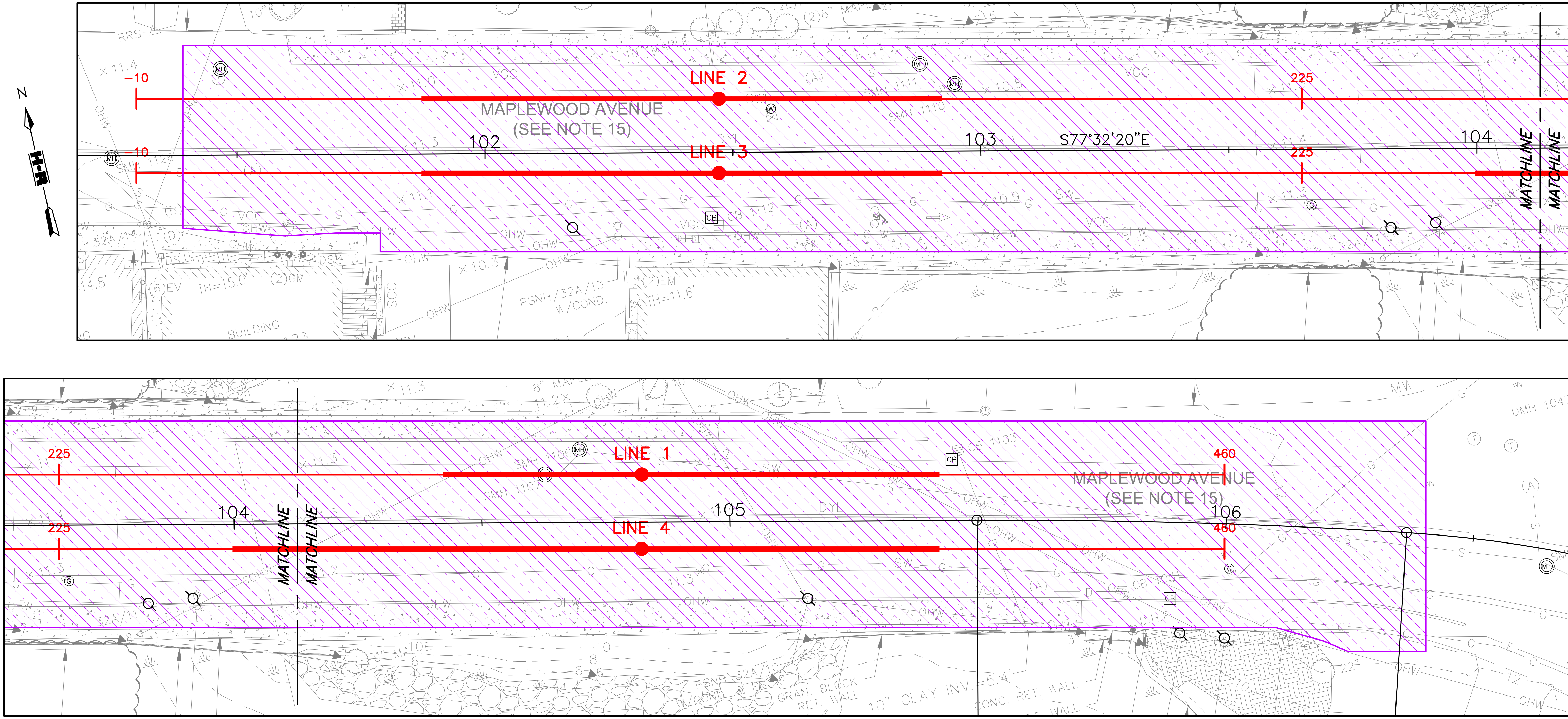
NOTE:

Modified from Google Earth Pro aerial photograph.

Figure 1
General Site Location
Maplewood Avenue Bridge
Portsmouth, New Hampshire

File 19J82	August, 2020

HAGER–RICHTER
Salem, NH | Fords, NJ



SCALE (feet)

0 20 40

LEGEND

	APPROXIMATE LIMITS OF GPR SURVEY AREA
	MASW/REMI SURVEY LINE WITH INDICATION OF THE EFFECTIVE DATA COVERAGE IN BOLD
	GAS VALVE
	UTILITY POLE
	WATER VALVE
	MANHOLE
	CATCH BASIN

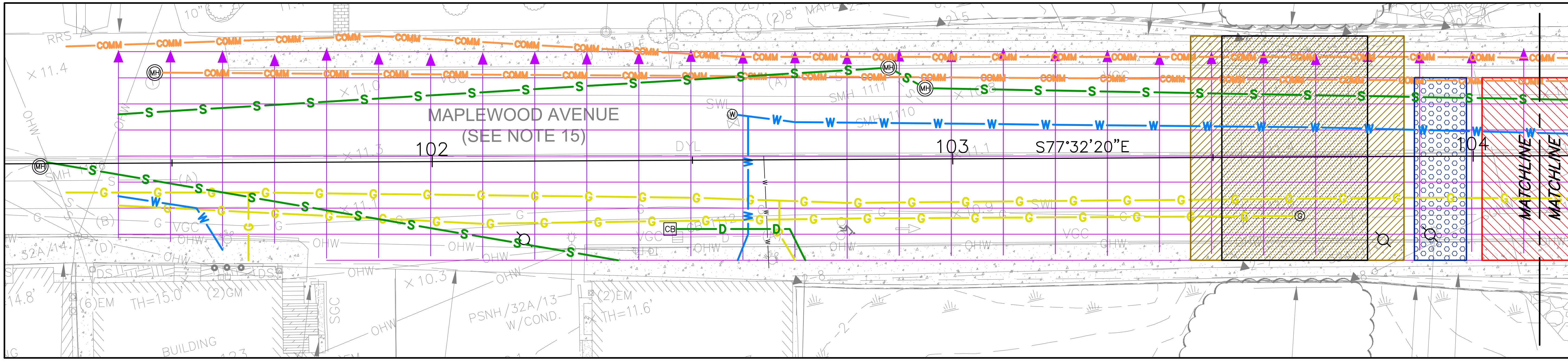
NOTE:

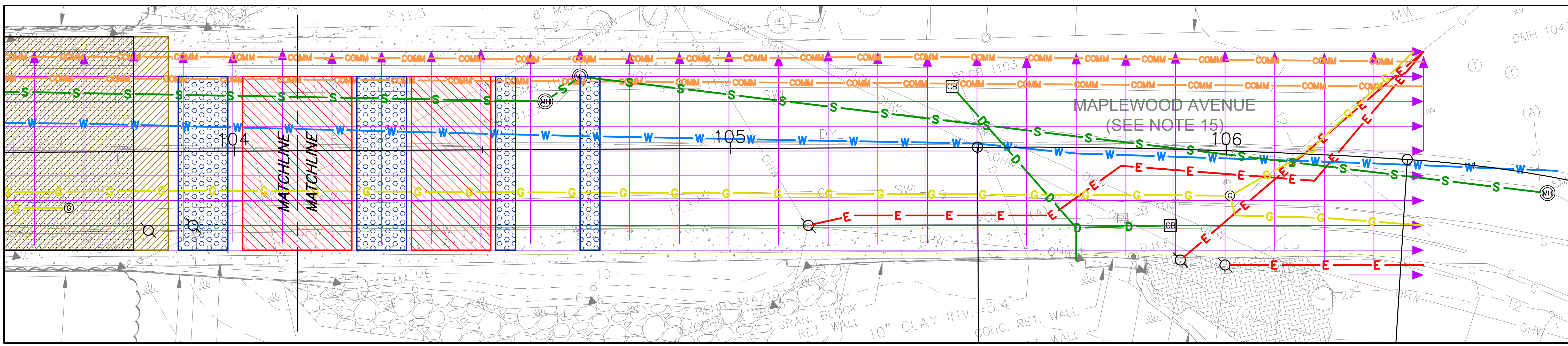
Modified from site plan provided by GZA, identified as 2016 ACAD-6032A 06-19-20.dwg.

Figure 2
Site Plan
Maplewood Avenue Bridge
Portsmouth, New Hampshire

File 19J82 | August, 2020

HAGER-RICHTER
Salem, NH | Fords, NJ





LEGEND

SCALE (feet)

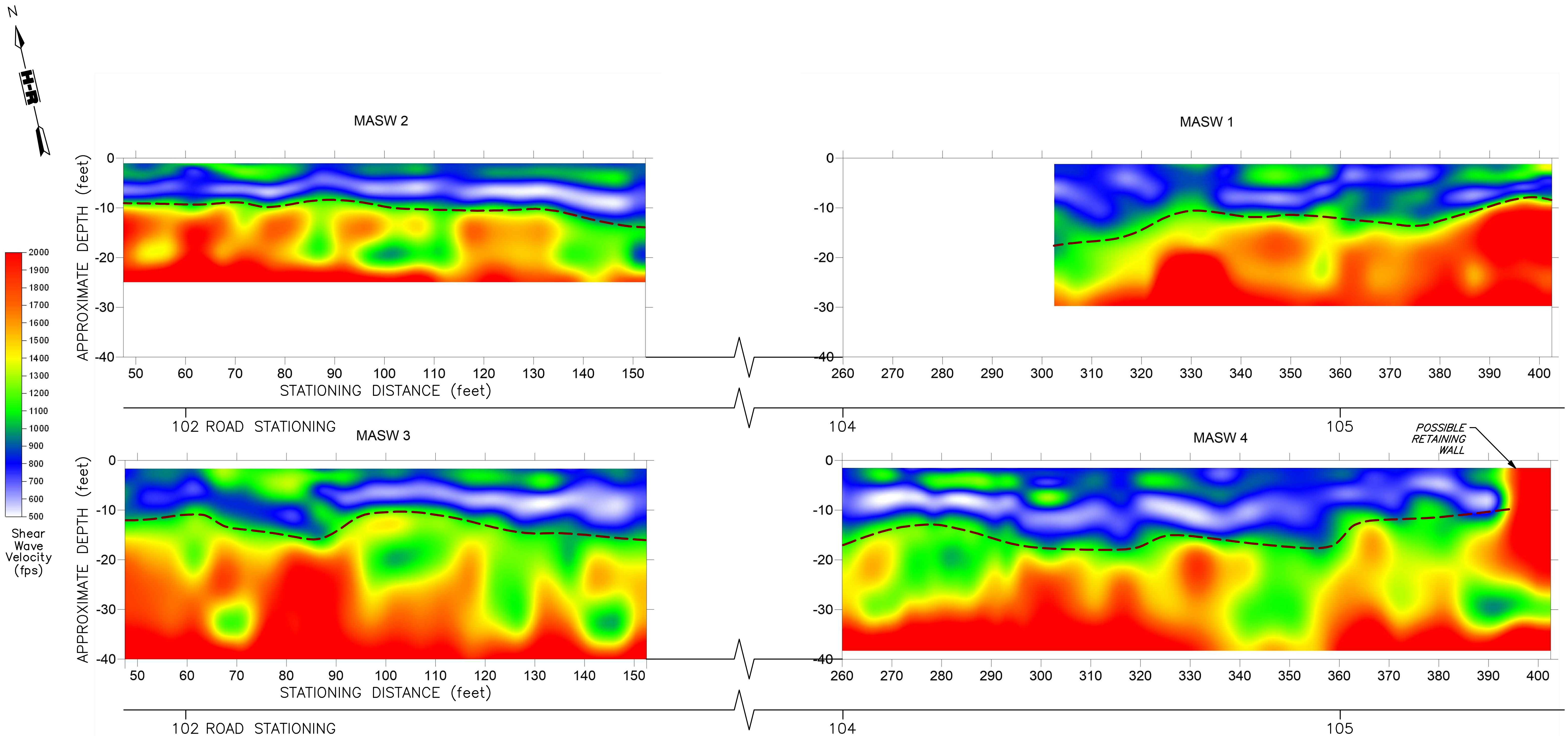
0 20 40

- GPR TRAVERSE
- GAS LINE
- ELECTRIC LINE
- WATER LINE
- SEWER LINE
- DRAIN LINE
- COMMUNICATION LINE
- WATER LINE – MARKED BY OTHERS
- POSSIBLE FORMER CULVERT/ARCHES
- POSSIBLE FORMER PIERS
- PRESENT ARCH STRUCTURE
- PRESENT ARCH OPENING
- GAS VALVE
- UTILITY POLE
- WATER VALVE
- MANHOLE
- CATCH BASIN

NOTE:

Modified from site plan provided by GZA, identified as 2016 ACAD-6032A 06-19-20.dwg.

Figure 3 GPR Survey Maplewood Avenue Bridge Portsmouth, New Hampshire	
File 19J82	August, 2020
HAGER-RICHTER Salem, NH | Fords, NJ	



NOTES:

1. MASW/Refraction data acquired using a 48–channel digital seismograph (Geometrics Geode) coupled to a 48 4.5–Hz geophones.
2. MASW data were analyzed using SurfSeis 6 software by Kansas Geological Survey.

SCALE (feet)

0 20 40

LEGEND

INTERPRETED BEDROCK SURFACE

Figure 4
MASW Profiles
Maplewood Avenue Bridge
Portsmouth, New Hampshire

File 19J82	August, 2020

HAGER–RICHTER
Salem, NH | Fords, NJ

Appendix D – Test Boring Logs

TEST BORING LOG

GZA GeoEnvironmental, Inc.
Engineers and Scientists

Hoyle, Tanner and Associates, Inc.
Maplewood Avenue Culvert
Portsmouth, New Hampshire

EXPLORATION NO.: B-101
SHEET: 1 of 1
PROJECT NO: 04.0191113.00
REVIEWED BY: DGL

Logged By: J. Szmyt
Drilling Co.: New England Boring Contractors
Foreman: B. Raiche

Type of Rig: ATV Track
Rig Model: Mobile B-29
Drilling Method: D&W

Boring Location: See Plan
Ground Surface Elev. (ft.):
Final Boring Depth (ft.): 27
Date Start - Finish: 8/4/2020 - 8/6/2020

H. Datum:
V. Datum:

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
8/6/2020	1136	15 min.	6.8	--

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Depth (ft.)	Stratum Description	Elev. (ft.)
									1		0.6	ASPHALT	
									2		1	SAND	
											1.3	ASPHALT	
								Light brown/orange, fine to medium SAND and Gravel, trace Silt, moist.			4.7	FILL	
5									3		6.3	COBBLES/BOULDERS AND BRICK	
		S-1	8.0-10.0	24	2	4 3 4 4	7	S-1: Loose, brown, fine to medium SAND and Gravel, trace Silt, trace Wood, wet.					
10												FILL	
15		S-2	15.0-15.3	4	2	100/4"	R	S-2: Gray and light brown, fine to medium SAND and Gravel, little Silt, wet.	4		15.3		
	(2:21) (2:32) (2:24)	C-1	17.0-22.0	60	52	RQD= 38%		C-1: Hard, fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to moderately close, low to high angle, planar and undulating, rough, tight to open, discolored.	5				
20	(2:23) (2:21)											PHYLLITE (BEDROCK)	
	(1:53) (2:15) (2:57)	C-2	22.0-27.0	60	59	RQD= 37%		C-2: Hard, fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to moderately close, low to high angle, planar and undulating, rough, tight to open, fresh to discolored.	6				
25	(3:24) (3:54)										27		
								End of exploration at 27 feet.	7				
30													

REMARKS

1 - Vacuum excavated to approximately 8.0 feet below ground surface with the VacMasters 1000 vacuum truck.
2 - Soil descriptions observed from sidewalls of vacuum excavation.
3 - Cobbles/boulders and bricks encountered from approximately 4.7 feet to 6.3 feet below ground surface.
4 - Split spoon refusal at 15.3 feet below ground surface on probable bedrock. Advanced roller bit to approximately 17.0 feet below ground surface before coring.
5 - Bedrock testing completed on C-1 (17.0-17.7 ft.) sample, UCS = 22,273 psi, unit weight = 170.4 pcf.
6 - Lost water return at approximately 23.5 feet below ground surface during core run C-2.
7 - Borehole backfilled with cuttings and sand. Approximately 0.5 feet of concrete was placed below approximately 0.5 feet of asphalt batch.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-101

TEST BORING LOG

GZA GeoEnvironmental, Inc. *Engineers and Scientists*	Hoyle, Tanner and Associates, Inc. Maplewood Avenue Culvert Portsmouth, New Hampshire	EXPLORATION NO.: B-102 SHEET: 1 of 1 PROJECT NO: 04.0191113.00 REVIEWED BY: DGL

Logged By: J. Szmyt
Drilling Co.: New England Boring Contractors
Foreman: B. Raiche

Type of Rig: ATV Track
Rig Model: Mobile B-29
Drilling Method: D&W

Boring Location: See Plan
Ground Surface Elev. (ft.):
Final Boring Depth (ft.): 23
Date Start - Finish: 8/4/2020 - 8/5/2020

H. Datum:
V. Datum:

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: None

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
8/5/2020	1425	30 min.	8.3	9

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Depth (ft.)	Stratum Description	Elev. (ft.)
									1		0.7	ASPHALT	
									2		1	SAND	
											1.3	ASPHALT	
5								Light brown/orange, fine to medium SAND and Gravel, little Silt, moist.				FILL	
									3				
		S-1	8.0-8.8	10	3	11 100 /4"		S-1: Brown, GRAVEL, some fine to coarse SAND, trace Silt, wet.			8.8		
10									4				
15												COBBLE/BOULDERS	
20											21		
									5		23	PROBABLE BEDROCK	
								End of exploration at 23 feet.	6				
25													
30													

REMARKS

1 - Vacuum excavated to approximately 7.0 feet below ground surface with the VacMasters 1000 vacuum truck.
2 - Soil descriptions observed from sidewalls of vacuum excavation.
3 - Refusal on probable cobble or boulder at approximately 8.8 feet below ground surface.
4 - Very hard drilling from approximately 8.8 feet below ground surface with no water return. Cobbles and boulders from approximately 8.8 feet below ground surface to 21 feet below ground surface
5 - Advanced roller bit to approximately 21 feet below ground surface where probable bedrock was encountered. Advance roller bit to approximately 23 feet below ground surface into probable bedrock. Unable to advance casing through cobbles and boulders.
6 - Borehole backfilled with cuttings and sand. Approximately 0.5 feet of concrete was placed below approximately 0.5 feet of asphalt patch.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-102

TEST BORING LOG

GZA GeoEnvironmental, Inc. *Engineers and Scientists*	Hoyle, Tanner and Associates, Inc. Maplewood Avenue Culvert Portsmouth, New Hampshire	**EXPLORATION NO.:** B-103 **SHEET:** 1 of 2 **PROJECT NO:** 04.0191113.00 **REVIEWED BY:** DGL

Logged By: J. Szmyt
Drilling Co.: New England Boring Contractors
Foreman: B. Raiche

Type of Rig: ATV Track
Rig Model: Mobile B-29
Drilling Method: D&W

Boring Location: See Plan
Ground Surface Elev. (ft.):
Final Boring Depth (ft.): 34
Date Start - Finish: 8/4/2020 - 8/5/2020

H. Datum:
V. Datum:

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
8/5/2020	1119	15 min.	9.0	24.0

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Depth (ft.)	Stratum Description	Elev. (ft.)
									1		0.7	ASPHALT	
									2		1	SAND	
											1.3	ASPHALT	
								Light brown/orange, fine to medium SAND and Gravel, little Silt, moist.			3	FILL	
5									3		7.5	COBBLES/BOULDERS AND BRICKS	
10		S-1	10.0-12.0	24	8	6 3 6 5	9	S-1: Loose, brown/gray, GRAVEL, some fine to coarse SAND, some Silt, trace Brick, wet.					
15		S-2	15.0-17.0	24	4	7 5 3 5	8	S-2: Loose, gray, GRAVEL, some fine to coarse SAND, little Silt, wet.				FILL	
20		S-3	20.0-22.0	24	17	WOH/12" 5 6	5	S-3: Loose, gray, fine to medium SAND and Silt, trace Gravel, trace Wood, wet.			23.5		
25	(2:55)	C-1	24.0-26.5	30	24	RQD= 13%		C-1: Hard, slightly weathered, fine-grained, gray, PHYLLITE.	4			PHYLLITE (BEDROCK)	

REMARKS

1 - Vacuum excavated to approximately 9.5 feet below ground surface with the VacMasters 1000 vacuum truck.
2 - Soil descriptions observed from sidewalls of vacuum excavation.
3 - Cobbles/boulders and brick encountered from approximately 3.0 feet to 7.5 feet below ground surface.
4 - Top of bedrock at approximately 23.5 feet below ground surface. Advanced roller bit to approximately 24.0 feet below ground surface before coring.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-103

TEST BORING LOG

GZA GeoEnvironmental, Inc.
Engineers and Scientists

Hoyle, Tanner and Associates, Inc.
Maplewood Avenue Culvert
Portsmouth, New Hampshire

PROJECT NO: 04.0191113.00
REVIEWED BY: DGL

Logged By: J. Szmyt
Drilling Co.: New England Boring Contractors
Foreman: B. Raiche

Type of Rig: ATV Track
Rig Model: Mobile B-29
Drilling Method: D&W

Boring Location: See Plan
Ground Surface Elev. (ft.):
Final Boring Depth (ft.): 34
Date Start - Finish: 8/4/2020 - 8/5/2020

H. Datum:
V. Datum:

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
8/5/2020	1119	15 min.	9.0	24.0

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Depth (ft.)	Stratum Description	Elev. (ft.)
	(2:33) (3:16)							Joints are extremely close to close, low to high angle, planar and stepped, rough, discolored, partially open to open, discolored.					
	(1:56) (3:18) (4:30)	C-2	26.5-29.0	30	17	RQD= 0%		C-2: Hard, slightly weathered, fine-grained, gray, PHYLLITE. Joints are extremely close, moderately dipping to high angle, planar, rough, partially open to open, discolored.	5			PHYLLITE (BEDROCK)	
30	(3:16) (4:40) (2:58) (2:00)	C-3	29.0-32.5	42	32	RQD= 19%		C-3: Hard, slightly weathered, fine-grained, gray, PHYLLITE. Joints are extremely close to close, low angle to moderately dipping, planar and stepped, partially open to open, discolored.					
	(1:13) (2:11)	C-4	32.5-34.0	18	18	RQD= 0%		C-4: Hard, slightly weathered to fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to close, low angle to moderately dipping, planar and stepped, rough, partially open to open, discolored.	6		34		
35								End of exploration at 34 feet.	7				
40													
45													
50													

REMARKS

5 - Core barrel jammed at approximately 26.5 feet below ground surface.
6 - Core barrel jammed at approximately 32.5 feet below ground surface.
7 - Borehole backfilled with cuttings and sand. Approximately 0.5 feet of concrete was placed below approximately 0.5 feet of asphalt patch.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-103

TEST BORING LOG

GZA GeoEnvironmental, Inc. *Engineers and Scientists*	**Hoyle, Tanner and Associates, Inc.** **Maplewood Avenue Culvert** **Portsmouth, New Hampshire**	**EXPLORATION NO.: B-104** **SHEET: 1 of 2** **PROJECT NO: 04.0191113.00** **REVIEWED BY: DGL**

Logged By: J. Szmyt **Drilling Co.:** New England Boring Contractors **Foreman:** B. Raiche	**Type of Rig:** ATV Track **Rig Model:** Mobile B-29 **Drilling Method:** D&W	**Boring Location:** See Plan **Ground Surface Elev. (ft.):** **Final Boring Depth (ft.):** 32.5 **Date Start - Finish:** 8/5/2020 - 8/6/2020	**H. Datum:** **V. Datum:**

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
8/6/2020	1502	15 min.	9.4	7

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Depth (ft.)	Stratum Description	Elev. (ft.)
									1		0.6	ASPHALT	
									2		0.9	SAND	
											1.2	ASPHALT	
								Brown/orange, fine to medium SAND, some Gravel, trace Silt, moist.					
5												FILL	
		S-1	8.0-10.0	24	12	4 7 6 20	13	S-1: Medium dense, brown, GRAVEL and fine to coarse SAND, little Silt, trace Brick, wet.	3				
10											12		
									4			COBBLES/BOULDERS	
15											15		
		S-2	15.0-17.0	24	3	1 19 11 8	30	S-2: Dense, brown, GRAVEL and fine to coarse SAND, trace Silt, trace Brick, wet.				FILL	
20		S-3	20.0-20.9	11	5	11 50 /5"		S-3: Gray, fine to medium SAND and Gravel, little Silt, wet.			20.9		
									5				
	(2:40) (3.49) (2:17/2")	C-1	22.5-24.7	26	26	RQD= 15%		C-1: Hard, fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to close, horizontal to moderately dipping, planar and undulating, tight to partially open, discolored and fresh.	6			PHYLLITE (BEDROCK)	
25	(1:49/10")												

REMARKS

1 - Vacuum excavated to approximately 8.0 feet below ground surface with the VacMasters 1000 vacuum truck.
2 - Soil descriptions observed from sidewalls of vacuum excavation.
3 - Rock in tip of split spoon for Sample S-1.
4 - Cobbles/boulders from approximately 12 to 15 feet below ground surface.
5 - Split spoon refusal at approximately 20.9 feet below ground surface on probable bedrock.
6 - Advanced roller bit to approximately 22.5 feet below ground surface before coring.
7 - Core barrel jammed at approximately 24.7 feet below ground surface.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-104

TEST BORING LOG

GZA GeoEnvironmental, Inc.
Engineers and Scientists

Hoyle, Tanner and Associates, Inc.
Maplewood Avenue Culvert
Portsmouth, New Hampshire

EXPLORATION NO.: B-104
SHEET: 2 of 2
PROJECT NO: 04.0191113.00
REVIEWED BY: DGL

Logged By: J. Szmyt
Drilling Co.: New England Boring Contractors
Foreman: B. Raiche

Type of Rig: ATV Track
Rig Model: Mobile B-29
Drilling Method: D&W

Boring Location: See Plan
Ground Surface Elev. (ft.):
Final Boring Depth (ft.): 32.5
Date Start - Finish: 8/5/2020 - 8/6/2020

H. Datum:
V. Datum:

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)				
Date	Time	Stab. Time	Water	Casing
8/6/2020	1502	15 min.	9.4	7

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Depth (ft.)	Stratum Description	Elev. (ft.)
	(3:44) (1:56)	C-2	24.7-27.5	34	31	RQD= 15%		C-2: Hard, fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to close, low angle to moderately dipping, planar and stepped, rough, tight to partially open, and fresh.	7				
30	(2:32) (3:02) (2:44/10")	C-3	27.5-30.3	34	32	RQD= 0%		C-3: Hard, fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to close, horizontal to high angle, undulating and stepped, rough, partially open to open, fresh and discolored.				PHYLLITE (BEDROCK)	
	(1:46/2") (4:10) (4:08)	C-4	30.3-32.5	26	24	RQD= 0%		C-4: Hard, fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to close, horizontal to high angle, undulating, partially open to open, fresh and discolored.	8		32.5		
								End of exploration at 32.5 feet.	9				
35													
40													
45													
50													

REMARKS

8 - Core barrel jammed at approximately 30.3 feet below ground surface.
9 - Borehole backfilled with cuttings and sand. Approximately 0.5 feet of concrete was placed below approximately 0.5 feet of asphalt patch.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-104

TEST BORING LOG

GZA GeoEnvironmental, Inc.
Engineers and Scientists

Hoyle, Tanner and Associates, Inc.
Maplewood Avenue Culvert
Portsmouth, New Hampshire

EXPLORATION NO.: B-105
SHEET: 1 of 2
PROJECT NO: 04.0191113.00
REVIEWED BY: DGL

Logged By: J. Szmyt
Drilling Co.: New England Boring Contractors
Foreman: B. Raiche

Type of Rig: ATV Track
Rig Model: Mobile B-29
Drilling Method: D&W

Boring Location: See Plan
Ground Surface Elev. (ft.):
Final Boring Depth (ft.): 26
Date Start - Finish: 8/5/2020 - 8/7/2020

H. Datum:
V. Datum:

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
Not Measured				

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Depth (ft.)	Stratum Description	Elev. (ft.)
									1 2		0.7	ASPHALT	
											1	SAND	
											1.3	ASPHALT	
								Light brown/orange, fine to medium SAND, little Gravel, little Silt, moist.					
5													
		S-1	8.0-10.0	24	9	2 2 1 1	3	S-1: Very loose, brown, fine to coarse SAND and Silt, trace Gravel, moist.				FILL	
10													
15		S-2	15.0-17.0	24	3	20 21 26 14	47	S-2: Dense, gray, GRAVEL, little fine to coarse SAND, trace Silt, trace Brick, moist.	3				
									4		17.5	WEATHERED ROCK	
20											20		
	(0:53) (3:34) (3.59) (0.46/2")	C-1	21.0-24.2	38	29	RQD= 0%		C-1: Hard, fresh, fine-grained, gray and white, PHYLLITE. Joints are extremely close to close, low to high angle, planar and stepped, rough, tight to partially open, discolored.				PHYLLITE (BEDROCK)	
25	(4:35)	C-2	24.2-25.3	13	13	RQD=		C-2: Hard, fresh, fine-grained, gray and white, PHYLLITE.	5				

REMARKS

1 - Vacuum excavated to approximately 8.0 feet below ground surface with the VacMasters 1000 vacuum truck.
2 - Soil descriptions observed from sidewalls of vacuum excavation.
3 - Lost water return at approximately 16 feet after sample S-2.
4 - Encountered weathered bedrock at approximately 17.5 feet below ground surface and advanced roller bit to approximately 21 feet below ground surface before coring.
5 - Core barrel jammed at approximately 24.2 feet below ground surface.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-105

TEST BORING LOG

GZA GeoEnvironmental, Inc.
Engineers and Scientists

Hoyle, Tanner and Associates, Inc.
Maplewood Avenue Culvert
Portsmouth, New Hampshire

EXPLORATION NO.: B-105
SHEET: 2 of 2
PROJECT NO: 04.0191113.00
REVIEWED BY: DGL

Logged By: J. Szmyt
Drilling Co.: New England Boring Contractors
Foreman: B. Raiche

Type of Rig: ATV Track
Rig Model: Mobile B-29
Drilling Method: D&W

Boring Location: See Plan
Ground Surface Elev. (ft.):
Final Boring Depth (ft.): 26
Date Start - Finish: 8/5/2020 - 8/7/2020

H. Datum:
V. Datum:

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
Not Measured				

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Depth (ft.)	Stratum Description	Elev. (ft.)
	(1:45/4") (3:27/8")	C-3	25.3-26.0	8	6	0% RQD= 0%		Joints are extremely close to very close, horizontal to high angle, undulating, rough, partially open, fresh. C-3: Hard, fresh, fine-grained, gray and white, PHYLLITE. Joints are extremely close to very close, low to high angle, undulating, rough, partially open, fresh.	6 7 8		26	PHYLLITE (BEDROCK)	
								End of exploration at 26 feet.					
30													
35													
40													
45													
50													

REMARKS

6 - Core barrel jammed at approximately 25.3 feet below ground surface.
7 - Core barrel jammed at approximately 26.0 feet below ground surface.
8 - Borehole backfilled with cuttings and sand. Approximately 0.5 feet of concrete was placed below approximately 0.5 feet of asphalt patch.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-105

TEST BORING LOG

GZA GeoEnvironmental, Inc.
Engineers and Scientists

Hoyle, Tanner and Associates, Inc.
Maplewood Avenue Culvert
Portsmouth, New Hampshire

EXPLORATION NO.: B-106
SHEET: 1 of 2
PROJECT NO: 04.0191113.00
REVIEWED BY: DGL

Logged By: J. Szmyt
Drilling Co.: New England Boring Contractors
Foreman: B. Raiche

Type of Rig: ATV Track
Rig Model: Mobile B-29
Drilling Method: D&W

Boring Location: See Plan
Ground Surface Elev. (ft.):
Final Boring Depth (ft.): 30
Date Start - Finish: 8/5/2020 - 8/7/2020

H. Datum:
V. Datum:

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
8/7/2020	1026	15 min.	8.5	18

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Depth (ft.)	Stratum Description	Elev. (ft.)
									1 2		0.8 1.3 1.7	ASPHALT SAND ASPHALT	
								Light brown/orange, fine to medium SAND, some Gravel, trace Silt, moist.					
5													
		S-1	8.0-10.0	24	3	4 4 1 2	5	S-1: Loose, brown, fine to medium SAND, little Silt, little Gravel, trace Wood, wet.	3				
10												FILL	
									4				
15		S-2	15.0-17.0	24	5	23 19 5 6	24	S-2: Medium dense, brown, fine to coarse SAND and Gravel, trace Silt, trace Brick, wet.					
									5		18		
20	(2:17) (3:03) (3:45) (2:26/4")	C-1	20.0-22.8	34	22	RQD= 0%		C-1: Hard, fresh, fine-grained, gray and white, PHYLLITE. Joints are extremely close to close, low angle to moderately dipping, undulating and planar, rough, tight to partially open, fresh.				PHYLLITE (BEDROCK)	
	(22/8") (2:54)	C-2	22.8-25.0	26	18	RQD= 20%		C-2: Hard, fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to close, low angle to moderately dipping, undulating and stepped, rough, partially open to open, fresh to	6				
25													

REMARKS

1 - Vacuum excavated to approximately 8.0 feet below ground surface with the VacMasters 1000 vacuum truck.
2 - Soil descriptions observed from sidewalls of vacuum excavation.
3 - Wood in tip of split spoon for sample S-1.
4 - Very hard drilling and rods chattering at approximately 13.0 feet below ground surface.
5 - Probable bedrock at approximately 18.0 feet below ground surface and advanced roller bit to approximately 20 feet below ground surface before coring.
6 - Core barrel jammed at approximately 22.8 feet below ground surface.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-106

TEST BORING LOG

GZA GeoEnvironmental, Inc.
Engineers and Scientists

Hoyle, Tanner and Associates, Inc.
Maplewood Avenue Culvert
Portsmouth, New Hampshire

EXPLORATION NO.: B-106
SHEET: 2 of 2
PROJECT NO: 04.0191113.00
REVIEWED BY: DGL

Logged By: J. Szmyt **Drilling Co.:** New England Boring Contractors **Foreman:** B. Raiche	**Type of Rig:** ATV Track **Rig Model:** Mobile B-29 **Drilling Method:** D&W	**Boring Location:** See Plan **Ground Surface Elev. (ft.):** **Final Boring Depth (ft.):** 30 **Date Start - Finish:** 8/5/2020 - 8/7/2020	**H. Datum:** **V. Datum:**

Hammer Type: Automatic Hammer
Hammer Weight (lb.): 140
Hammer Fall (in.): 30
Auger or Casing O.D./I.D Dia (in.): 4

Sampler Type: SS
Sampler O.D. (in.): 2.0
Sampler Length (in.): 24
Rock Core Size: NX

Groundwater Depth (ft.)

Date	Time	Stab. Time	Water	Casing
8/7/2020	1026	15 min.	8.5	18

Depth (ft)	Casing Blows/ (Core Rate)	Sample No.	Sample Depth (ft.)	Pen (in)	Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	Field Test Data	Stratum Depth (ft.)	Stratum Description	Elev. (ft.)
	(?)	C-3	25.0-26.5	18	17	RQD= 0%		discolored.					
	(1:26)							C-3: Hard, fresh, fine-grained, gray, PHYLLITE. Joints are extremely close to close, moderately dipping to vertical, undulating and planar, rough, partially open to open, fresh.	7			PHYLLITE (BEDROCK)	
	(2:33/6")												
	(1:27/6")	C-4	26.5-30.0	42	40	RQD= 51%		C-4: Hard, fresh, fine-grained, gray and white, PHYLLITE. Joints are extremely close to moderately close, low to high angle, planar, tight to partially open, fresh and discolored.	9				
	(2:40)												
	(2:46)												
30								End of exploration at 30 feet.	8		30		
35													
40													
45													
50													

REMARKS

7 - Core barrel jammed at approximately 26.5 feet below ground surface.
9 - Bedrock testing completed on C-4 (27.6-28.3 ft.) sample, UCS = 7,714 psi, unit weight = 169.1 pcf.
8 - Borehole backfilled with cuttings and sand. Approximately 0.5 feet of concrete was placed below approximately 0.5 feet of asphalt patch.

See Log Key for explanation of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: **B-106**

Appendix E - Laboratory Test Results

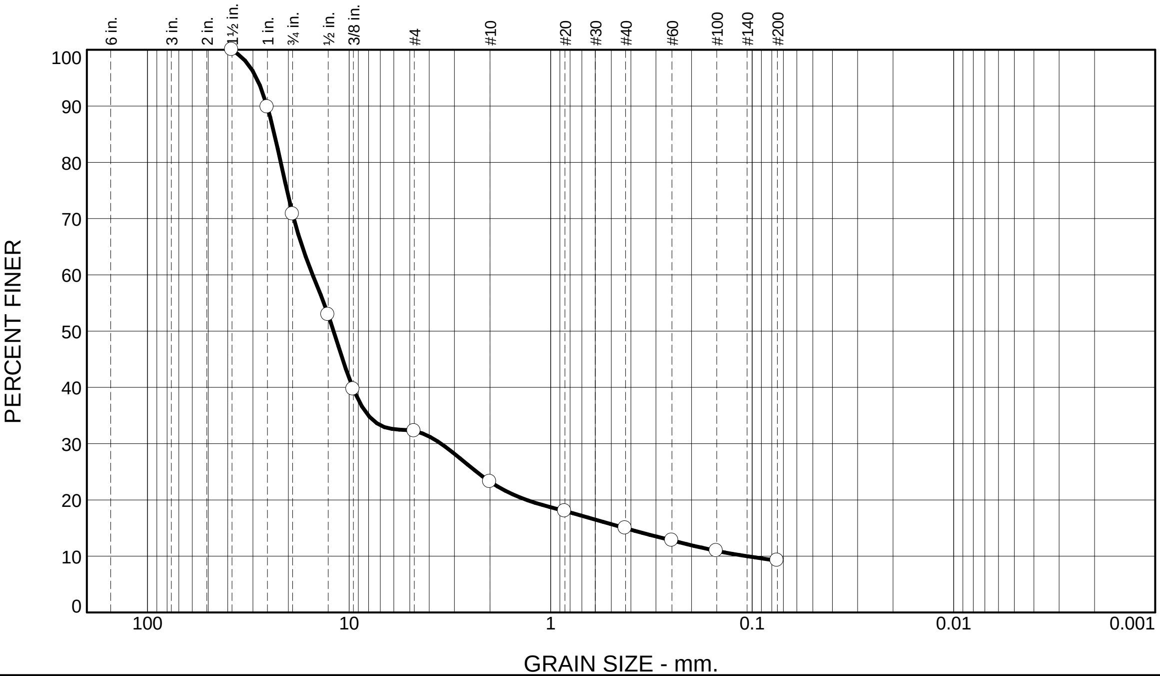
THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA GeoEnvironmental Bedford, NH PM: Jay Hodkinson Assigned By: Andrew Martin Collected By: Joshua Szmyt	Project Information: **Maplewood Ave Culvert Replacement** **Portsmouth, NH** GZA Project Number: 04.0191113.00 Summary Page: 1 of 1 Report Date: 08.19.2020

LABORATORY TESTING DATA SHEET, Report No.: 7420-H-145

Boring No.	Sample No.	Depth (Ft)	Laboratory No.	Identification Tests									Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s		Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
				D2216	D4318		D6913			D2974	D854				D1557						
B-102	S-1	8-8.8	20-S-2406				67.8	23.0	9.2												Brown GRAVEL, some f-c Sand, trace Silt
B-103	S-1	10-12	20-S-2407				39.6	33.8	26.6												Brown GRAVEl, some f-c Sand, some Silt
B-103	S-3	20-22	20-S-2408				4.0	52.1	43.9												Gray f-m SAND and SILT, trace Gravel
B-104	S-1	8-10	20-S-2409				43.6	40.7	15.7												Brown GRAVEl and f-c SAND, little Silt
B-104	S-2	15-17	20-S-2410				53.0	43.2	3.8												Brown GRAVEL and f-c SAND, trace Silt
B-105	S-1	8-10	20-S-2411				6.3	50.6	43.1												Light Brown f-c SAND and SILT, trace Gravel
B-106	S-2	15-17	20-S-2412				46.7	49.2	4.1												Brown f-c SAND and GRAVEL, trace Silt

Date Received: *08.17.2020* Reviewed By: St Bo Date Reviewed: *08.20.2020*

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	29.2	38.6	9.0	8.2	5.8	9.2	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	89.8		
0.75"	70.8		
0.5"	52.9		
0.375"	39.7		
#4	32.2		
#10	23.2		
#20	18.0		
#40	15.0		
#60	12.8		
#100	10.9		
#200	9.2		

* (no specification provided)

Material Description

Brown GRAVEL, some f-c Sand, trace Silt

Atterberg Limits (ASTM D 4318)

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

Classification

USCS (D 2487)= GP-GM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 25.4965 D_{85}= 23.4676 D_{60}= 15.1542
D_{50}= 11.9293 D_{30}= 3.4997 D_{15}= 0.4286
D_{10}= 0.1060 C_u= 142.95 C_c= 7.62

Remarks

Date Received: 08.17.2020 **Date Tested:** 08.19.2020

Tested By: AV / JM / LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

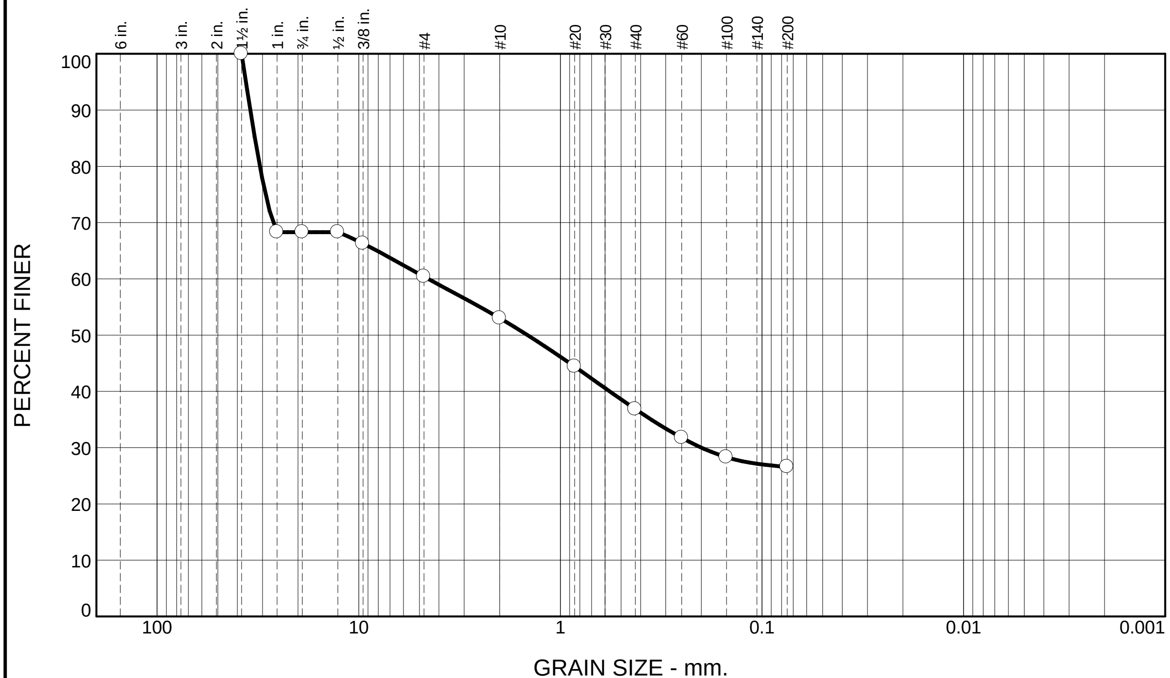
Source of Sample: Borings **Depth:** 8-8.8' **Date Sampled:**
Sample Number: B-102 / S-1

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Maplewood Ave Culvert Replacement
Portsmouth, NH
Project No: 04.0191113.00 **Figure** 20-S-2406

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	31.7	7.9	7.4	16.1	10.3	26.6	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	68.3		
3/4"	68.3		
1/2"	68.3		
3/8"	66.3		
#4	60.4		
#10	53.0		
#20	44.4		
#40	36.9		
#60	31.8		
#100	28.3		
#200	26.6		

* (no specification provided)

Material Description

Brown GRAVEl, some f-c Sand, some Silt

Atterberg Limits (ASTM D 4318)

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

Classification

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

Coefficients

D_{90}= 34.4949 D_{85}= 32.7274 D_{60}= 4.5289
D_{50}= 1.4541 D_{30}= 0.1995 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 08.17.2020 **Date Tested:** 08.19.2020

Tested By: AV / JM / LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

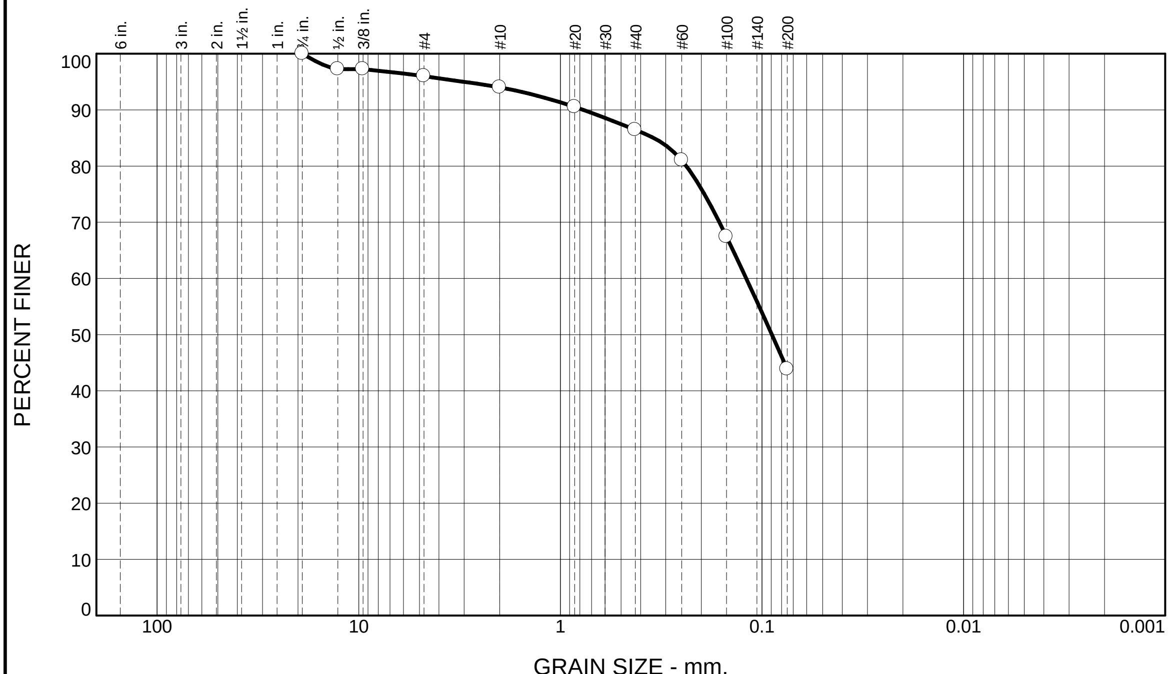
Source of Sample: Borings **Depth:** 10-12' **Date Sampled:**
Sample Number: B-103 / S-1

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Maplewood Ave Culvert Replacement
Portsmouth, NH
Project No: 04.0191113.00 **Figure** 20-S-2407

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.0	2.0	7.5	42.6	43.9	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	97.3		
0.375"	97.3		
#4	96.0		
#10	94.0		
#20	90.5		
#40	86.5		
#60	81.0		
#100	67.4		
#200	43.9		

* (no specification provided)

Material Description

Gray f-m SAND and SILT, trace Gravel

Atterberg Limits (ASTM D 4318)

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

Classification

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

Coefficients

D_{90}= 0.7693 D_{85}= 0.3442 D_{60}= 0.1196
D_{50}= 0.0894 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Sample visually classified as non-plastic.

Date Received: 08.17.2020 **Date Tested:** 08.19.2020

Tested By: AV / JM / LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

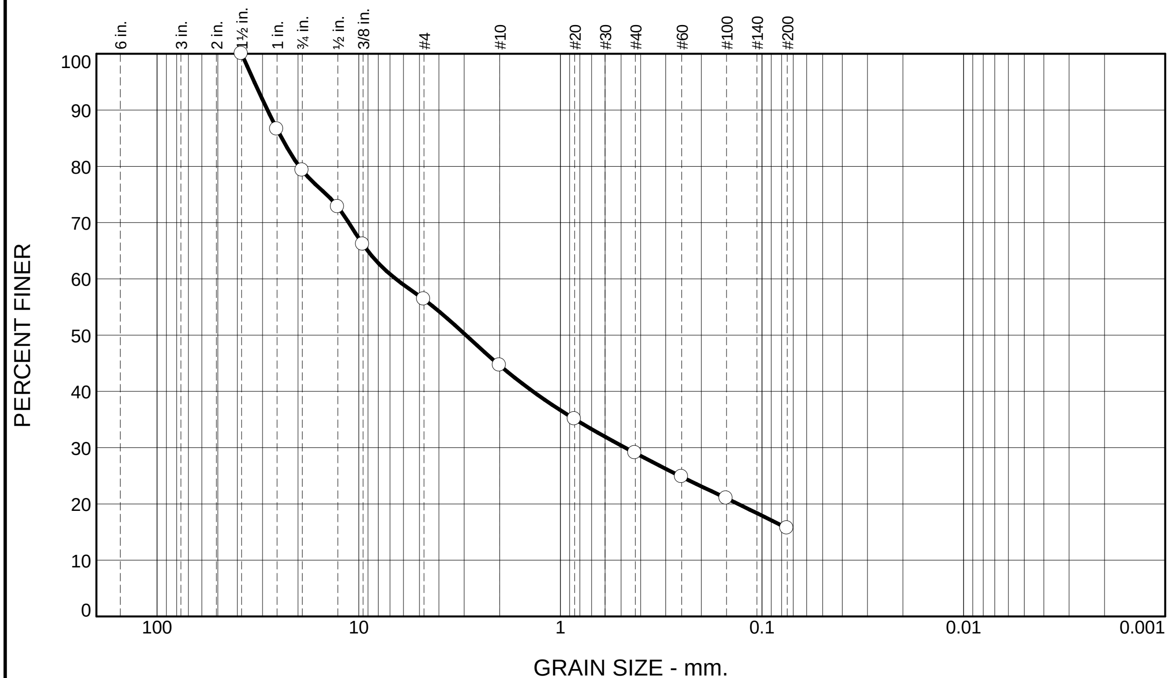
Source of Sample: Borings **Depth:** 20-22' **Date Sampled:**
Sample Number: B-103 / S-3

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Maplewood Ave Culvert Replacement
Portsmouth, NH
Project No: 04.0191113.00 **Figure** 20-S-2408

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	20.7	22.9	11.8	15.6	13.3	15.7	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	86.6		
3/4"	79.3		
1/2"	72.8		
3/8"	66.1		
#4	56.4		
#10	44.6		
#20	35.1		
#40	29.0		
#60	24.8		
#100	21.0		
#200	15.7		

* (no specification provided)

Material Description

Brown GRAVEl and f-c SAND, little Silt

Atterberg Limits (ASTM D 4318)

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

Classification

USCS (D 2487)= GM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 28.3328 D_{85}= 24.0619 D_{60}= 6.5498
D_{50}= 2.9405 D_{30}= 0.4770 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 08.17.2020 **Date Tested:** 08.19.2020

Tested By: AV / JM / LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

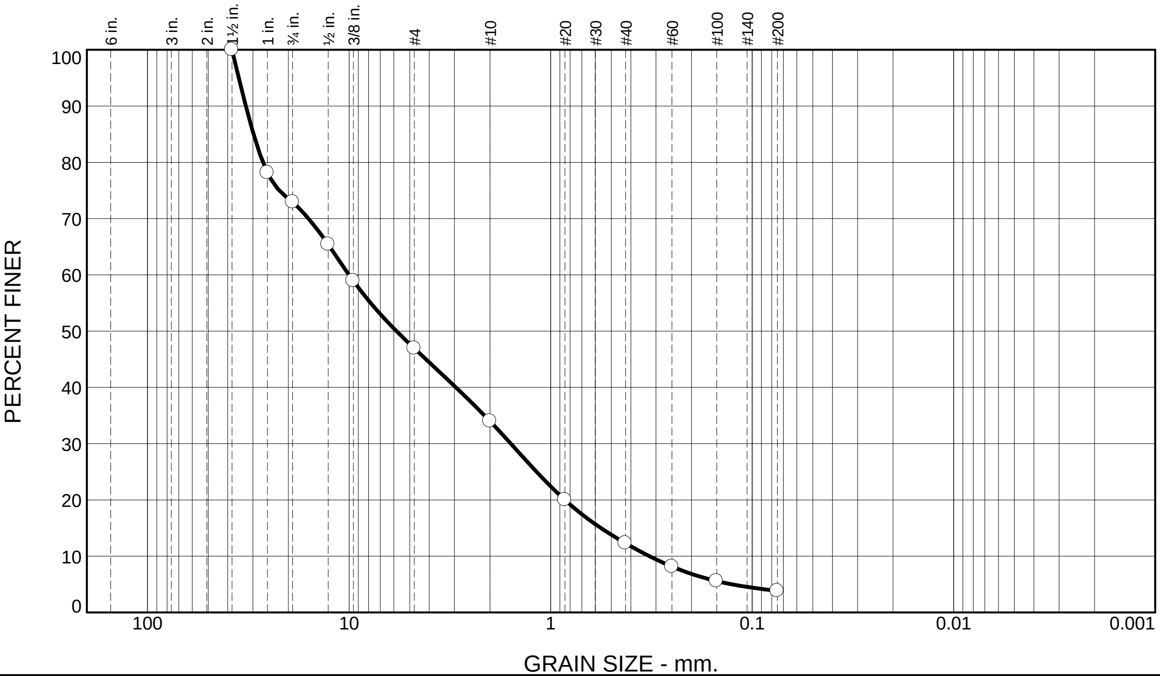
Source of Sample: Borings **Depth:** 8-10' **Date Sampled:**
Sample Number: B-104 / S-1

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Maplewood Ave Culvert Replacement
Portsmouth, NH
Project No: 04.0191113.00 **Figure** 20-S-2409

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	27.1	25.9	13.0	21.7	8.5	3.8	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	78.1		
3/4"	72.9		
1/2"	65.4		
3/8"	58.9		
#4	47.0		
#10	34.0		
#20	20.0		
#40	12.3		
#60	8.2		
#100	5.6		
#200	3.8		

* (no specification provided)

Material Description

Brown GRAVEL and f-c SAND, trace Silt

Atterberg Limits (ASTM D 4318)

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

Classification

USCS (D 2487)= GP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 32.5231 D_{85}= 29.7711 D_{60}= 10.0047
D_{50}= 5.8125 D_{30}= 1.5760 D_{15}= 0.5624
D_{10}= 0.3235 C_u= 30.93 C_c= 0.77

Remarks

Date Received: 08.17.2020 **Date Tested:** 08.19.2020

Tested By: AV / JM / LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

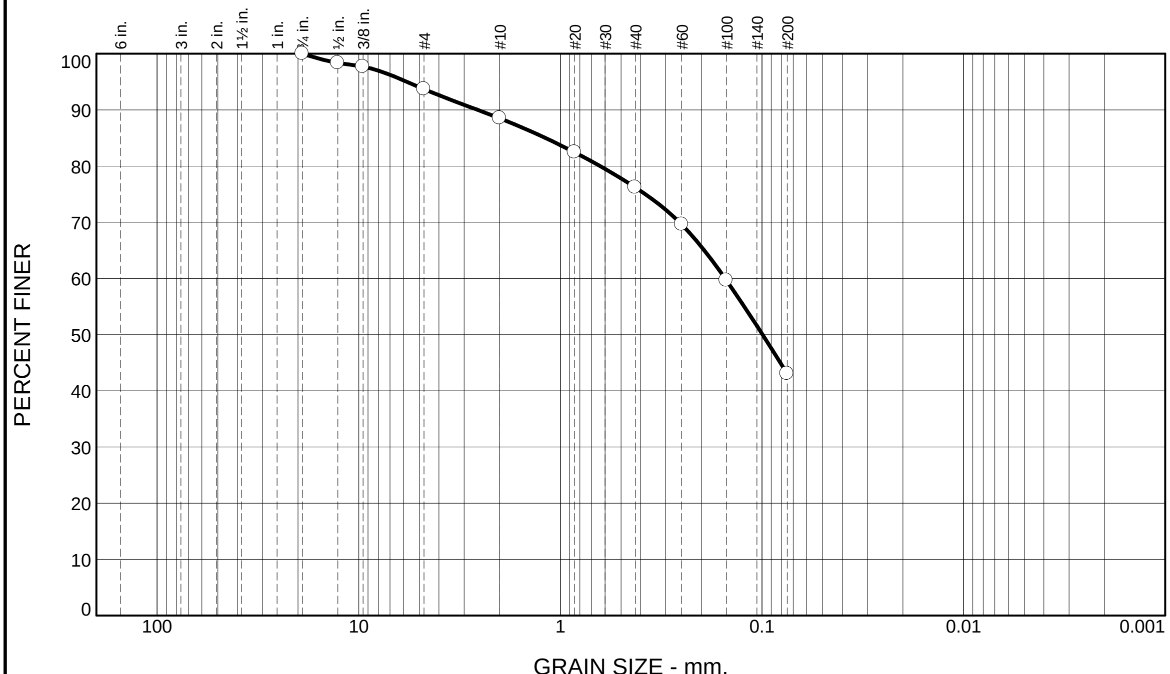
Source of Sample: Borings **Depth:** 15-17' **Date Sampled:**
Sample Number: B-104 / S-2

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Maplewood Ave Culvert Replacement
Portsmouth, NH
Project No: 04.0191113.00 **Figure** 20-S-2410

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.3	5.2	12.3	33.1	43.1	

Test Results (D6913 & ASTM D 1140)			
Opening Size	**Percent Finer**	**Spec.* (Percent)**	**Pass? (X=Fail)**
0.75"	100.0		
0.5"	98.4		
0.375"	97.7		
#4	93.7		
#10	88.5		
#20	82.5		
#40	76.2		
#60	69.6		
#100	59.6		
#200	43.1		

* (no specification provided)

Material Description

Light Brown f-c SAND and SILT, trace Gravel

Atterberg Limits (ASTM D 4318)

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

Classification

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

Coefficients

D_{90}= 2.5668 D_{85}= 1.1848 D_{60}= 0.1524
D_{50}= 0.0994 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Sample visually classified as non-plastic.

Date Received: 08.17.2020 **Date Tested:** 08.19.2020

Tested By: AV / JM / LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

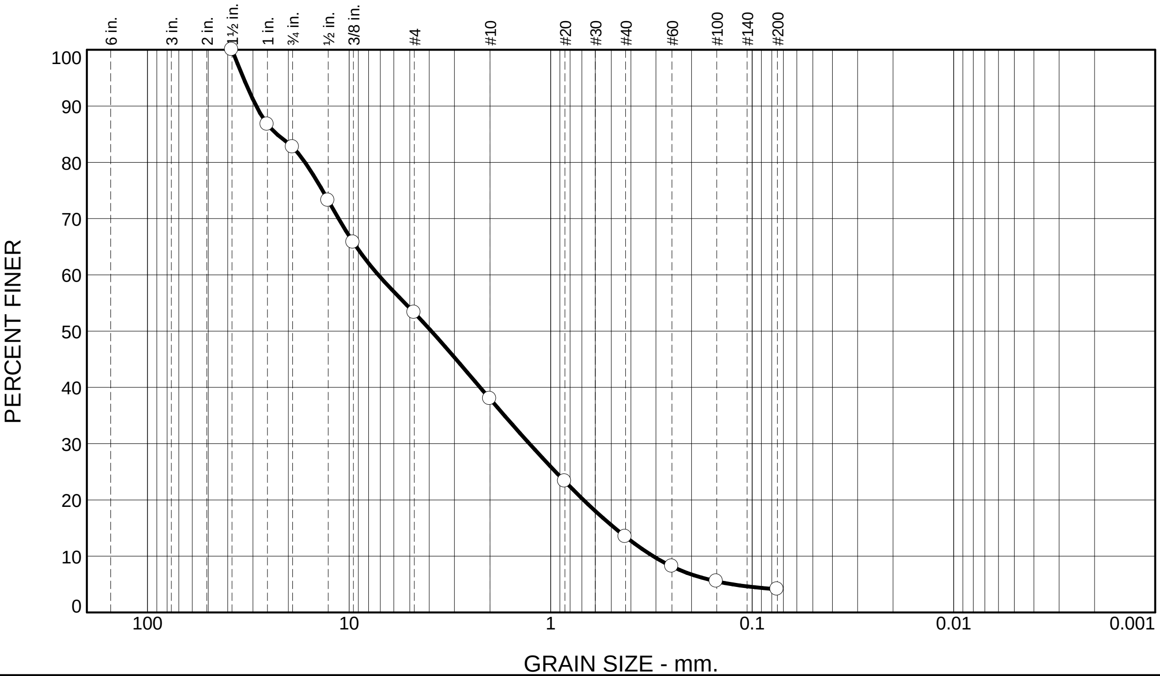
Source of Sample: Borings **Depth:** 8-10' **Date Sampled:**
Sample Number: B-105 / S-1

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Maplewood Ave Culvert Replacement
Portsmouth, NH
Project No: 04.0191113.00 **Figure** 20-S-2411

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	17.3	29.4	15.3	24.5	9.4	4.1	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	86.7		
3/4"	82.7		
1/2"	73.2		
3/8"	65.8		
#4	53.3		
#10	38.0		
#20	23.3		
#40	13.5		
#60	8.2		
#100	5.5		
#200	4.1		

* (no specification provided)

Material Description

Brown f-c SAND and GRAVEL, trace Silt

Atterberg Limits (ASTM D 4318)

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

Classification

USCS (D 2487)= SP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 28.8648 D_{85}= 22.7942 D_{60}= 7.1612
D_{50}= 3.9061 D_{30}= 1.2740 D_{15}= 0.4807
D_{10}= 0.3097 C_u= 23.12 C_c= 0.73

Remarks

Date Received: 08.17.2020 **Date Tested:** 08.19.2020

Tested By: AV / JM / LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: Borings **Depth:** 15-17'
Sample Number: B-106 / S-2

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Maplewood Ave Culvert Replacement
Portsmouth, NH
Project No: 04.0191113.00 **Figure** 20-S-2412

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA Geoenvironmental Bedford, NH PM: Jen Baron Assigned By: Andrew Martin Collected By: Joshua Szmyt	Project Information: **Maplewood Ave Culvert Replacement** **Portsmouth, NH** GZA Project Number: 04.0191113.00 Summary Page: 1 of 1 Report Date: 08.19.2020

LABORATORY TESTING DATA SHEET, Report No.: 7420-H-151

				Specimen Data						Compressive Strength Tests								
Boring No.	Sample No.	Depth (ft)	Laboratory No.	Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	$\sigma\tau$ PSI	Is_{50} PSI	(8) s_c PSI	Rock Formation or Description or Remarks
B-101	C-1	17.0-17.7	20-S-2419		1.992	4.445	170.4				22273							Phyllite
Break was fresh.																		
B-106	C-4	27.6-28.3	20-S-2420		1.995	4.472	169.1				7714							Phyllite
Break was along existing fault.																		

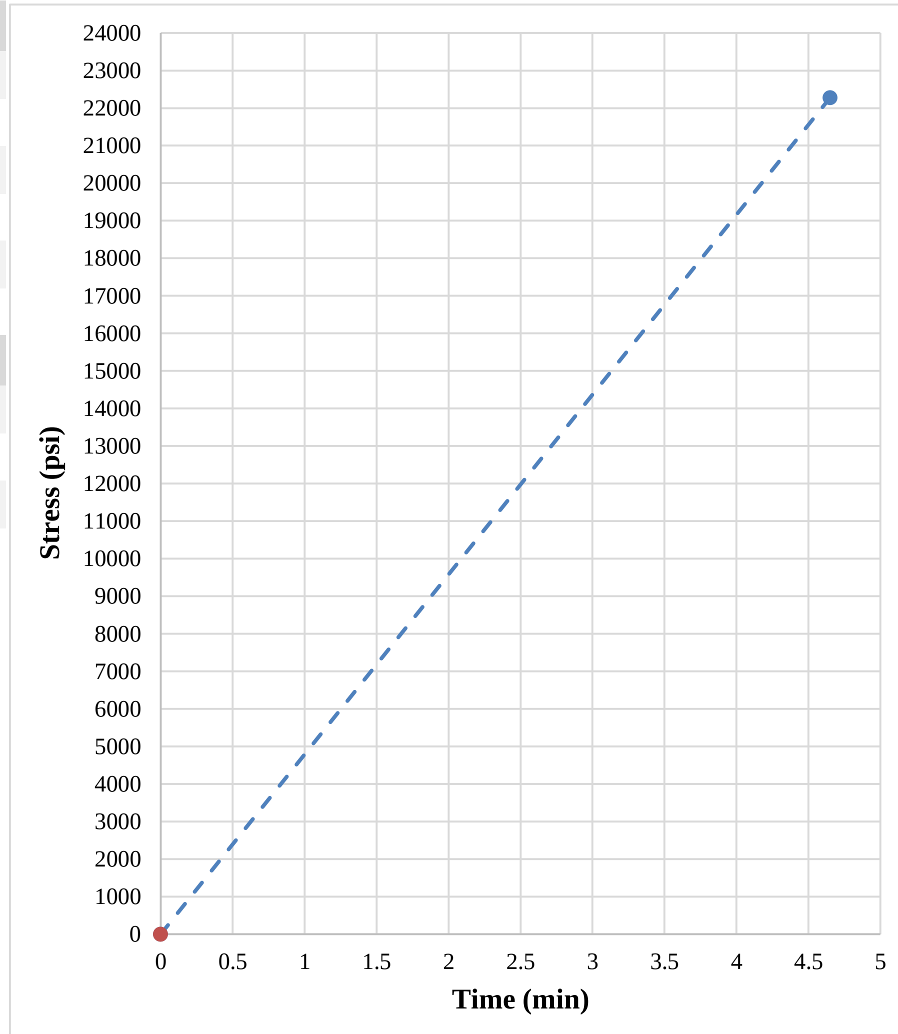
	Notes		Notes	
(1) Volume Determined By Measuring Dimensions (2) Determined by Measuring Dimensions and Weight of Saturated Sample		(3) PLD=Point Load (diametrical), PLA= Point Load (Axial) ST= Splitting Tensile U= Unconfined Compressive Strength (4) Taken at Peak Deviator Stress		(5) Strain at Peak Deviator Stress (6) Represents Secant Modulus at 50% of Total Failure Stress (7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress (8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (Is x 24)

Date Received: *08.17.2020*   Reviewed By: ______   Date Reviewed: *08.20.2020*

THIELSCH ENGINEERING	195 Frances Avenue Cranston, Rhode Island 02910 Phone: (401) 467-6454 Fax: (401) 467-2398 www.thielsch.com *Let's Build a Solid Foundation*	**Client Information:** GZA GeoEnvionmental Bedford, NH PM: Jen Baron Assigned by: Andrew Martin Collected by: Joshua Szmyt	**Project Information:** Maplewood Ave Culvert Replacement Portsmouth, NH Project Number: 04.0191113.00 Technician: JM Report Date: 08.19.2020

ASTM D7012 Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Sample Information		Compressive Test Information	
Boring ID:	B-101	**Unit Weight (pcf):**	170.4
Sample No.:	C-1	**Failure Stress (psi):**	22,273
Depth (ft):	17.0-17.7	**Failure Mode:**	Fresh
Tested Depth (ft):	17.2-17.6	**Time to Failure (min):**	4.65
Rock Type:	Slate		
Features:	Unweathered		
Test Specimen Information		**Elastic Moduli Test Information**	
Diameter, D (in):	1.992	**Poisson's Ratio @ 50%:**	NA
Length, L (in):	4.445	**Strain %:**	NA
L:D Ratio:	2.23	**E sec PSI @ 50%:**	NA

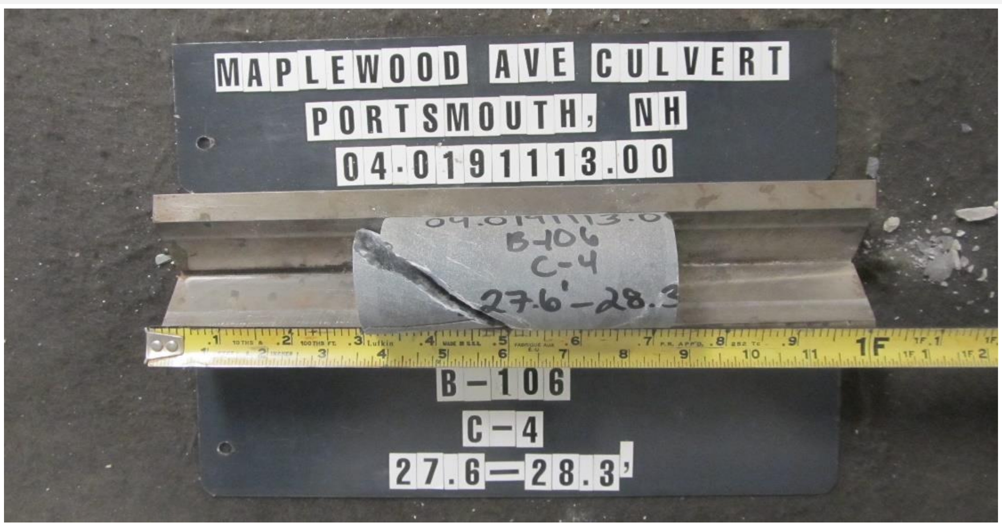


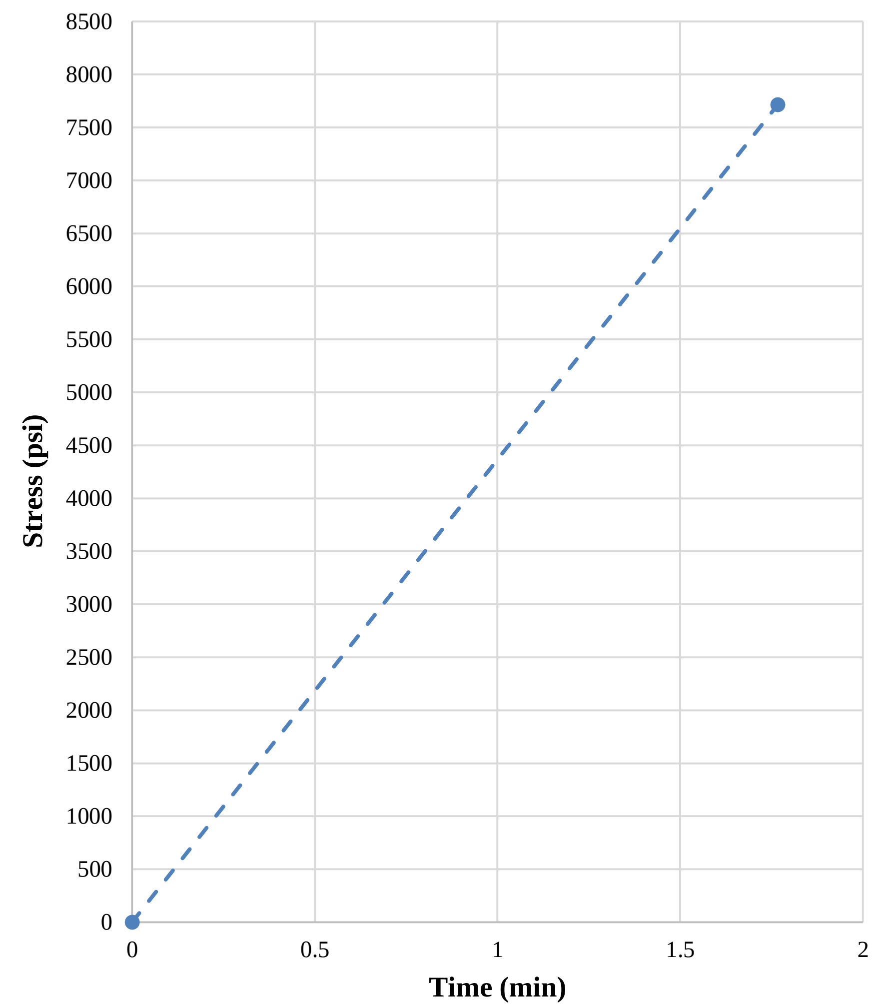
Testing Notes: Break was fresh.

THIELSCH ENGINEERING	195 Frances Avenue Cranston, Rhode Island 02910 Phone: (401) 467-6454 Fax: (401) 467-2398 www.thielsch.com *Let's Build a Solid Foundation*	**Client Information:** GZA GeoEnvionmental Bedford, NH PM: Jen Baron Assigned by: Andrew Martin Collected by: Joshua Szmyt	**Project Information:** Maplewood Ave Culvert Replacement Portsmouth, NH Project Number: 04.0191113.00 Technician: JM Report Date: 08.19.2020

ASTM D7012 Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Sample Information		Compressive Test Information	
Boring ID:	B-106	**Unit Weight (pcf):**	169.1
Sample No.:	C-4	**Failure Stress (psi):**	7,714
Depth (ft):	27.6-28.3	**Failure Mode:**	Along fault
Tested Depth (ft):	27.7-28.1	**Time to Failure (min):**	1.77
Rock Type:	Slate		
Features:	Exisiting faults		
Test Specimen Information		**Elastic Moduli Test Information**	
Diameter, D (in):	1.995	**Poisson's Ratio @ 50%:**	NA
Length, L (in):	4.472	**Strain %:**	NA
L:D Ratio:	2.24	**E sec PSI @ 50%:**	NA





Testing Notes: Break was along existing fault.