Appendix A:

Geotechnical Report

CONSULTING GROUP

Report on Geotechnical Investigation

Proposed Pedestrian Bridge Replacement Mill Pond Park 301 S. Leroy Street Road Fenton, Michigan

Latitude 42.794294˚ N
Longitude 83.704763˚ W

Prepared for:

City of Fenton
301 S. Leroy Street
Fenton, Michigan 48430

G2 Project No. 230607
February 8, 2024

CONSULTING
GROUP

February 8, 2024

Mr. Dan Brisson
Director - Department of Public Works
City of Fenton
301 S. Leroy Street
Fenton, Michigan 48430

RE: Report of Geotechnical Investigation
Proposed Pedestrian Bridge Replacement
Mill Pond Park
301 S. Leroy Street
Fenton, Michigan 48430
G2 Project No. 230607

Dear Mr. Brisson:

In accordance with your request, we have completed a geotechnical investigation for the proposed pedestrian bridge replacement project at Mill Pond Park located within the City of Fenton, Michigan. This report presents the results of our observations and analyses and includes recommendations and construction considerations relative to the proposed construction.

We appreciate the opportunity to be of service to the City of Fenton and look forward to discussing our findings. In the meantime, if you have any questions regarding this report or any other matter pertaining to the project, please call us.

Sincerely,

G2 Consulting Group, LLC

Jeffrey M. Hayball

Jeffrey M. Hayball, P.E.
Project Manager

Noel J. Hargrave-Thomas, P.E.
Principal

JMH/NJHT/ljv

Enclosures

EXCECUTIVE SUMMARY

We understand the proposed project includes replacing an existing pedestrian bridge spanning over the Shiawassee River at Mill Pond Park located within the City of Fenton, Michigan. The proposed bridge will consist of a Conspan 0-949 pre-fabricated structure with a top walk elevation of 895 feet and proposed bottom of foundation elevation of 883 feet, which will span 49 feet over the Shiawassee River. The ground surface elevation near the southwest abutment is approximately 894 feet and the ground surface elevation near the northeast abutment is approximately 892 feet.

Approximately 12 inches of silty clay topsoil are present at the ground surface of boring B-1. Granular fill soils, consisting of very loose to medium compact sand and gravelly sand, underlie the topsoil within boring B-1 and are present at the ground surface of boring B-2, and extend to depths of 6-1/2 and 9 feet, respectively. Very loose peat underlies the granular fill within boring B-1 and extends to an approximate depth of 11 feet. Native cohesive soils, consisting of very stiff to hard silty clay and sandy clay, are present below the peat within boring B-1 and gravelly sand fill within boring B-2, and extends to an approximate depth of 18 feet. Native granular soils, consisting of medium compact to very compact sand, gravelly sand, clayey sand, and silt, underlie the native cohesive soils within the borings and generally extend to the explored depth of 50 feet. However, native hard sandy clay is present within boring B-2 between depths of 23 and 36 feet. Groundwater was observed within borings B-1 and B-2 during drilling operations at an approximate depth of 4-1/2 feet, corresponding to elevations of 889-1/2 feet and 886-1/2 feet, respectively. Upon completion of drilling operations, groundwater was measured at depths ranging from 3-1/2 to 4 feet, corresponding to elevations ranging from 887-1/2 to 890 feet.

All vegetation, any surficial or buried organic soils, and any unsuitable existing fill soils should be removed in their entirety from within the areas to receive embankment fills or abutment backfills. The existing granular fill soils have organic matter contents ranging from 1.5 to 11.1 percent. The peat present within boring B-1 has an organic matter content of 6.5 percent. The peat present within boring B-1 and granular fill soils present within the upper 6-1/2 to 9 feet of the soil borings are not suitable for support of the embankment fill or the proposed bridge foundations. Therefore, the granular fill soils and peat must be completely removed and replaced with engineered fill.

Due to the presence of the Shiawassee River, temporary cofferdams will likely be required to reduce water infiltration into foundation excavations for the proposed abutments and to facilitate construction in dry conditions. The cofferdam(s) may consist of interlocking steel sheet piling. The preferred type of sheet piling consists of hot-rolled steel sheets, since the tight interlocks will not leak as much as cold-rolled steel sheet piling interlocks; however, the type of sheet piling may be left to the discretion of the contractor with the understanding that additional internal dewatering will be required for cold-rolled sheeting. Since the retained soils will not be drained, cofferdams should be designed to include a hydrostatic load as described in this report.

We understand the foundation bearing elevation at the proposed bridge is 883 feet. Given the existing soil conditions present at the anticipated foundation depths, we recommend the proposed concrete bridge abutments be supported on conventional spread and strip footings that extend through the existing fill and peat soils and bear within the underlying native very stiff silty clay. Foundations should be designed for a net allowable bearing capacity of 4,000 psf bearing in the very stiff silty clay. We recommend a G2 engineer be on site during construction to observe the excavations, measure the bearing depths, and verify the adequacy of the bearing soils.

Do not consider this summary separate from the entire text of this report, with all the conclusions and qualifications mentioned herein. Details of our analysis and recommendations are discussed in the following sections and in the Appendix of this report.

PROJECT DESCRIPTION

We understand the proposed project includes replacing an existing pedestrian bridge spanning over the Shiawassee River at Mill Pond Park located within the City of Fenton, Michigan. The proposed bridge will consist of a Conspan 0-949 pre-fabricated structure with a top walk elevation of 895 feet and proposed bottom of foundation elevation of 883 feet, which will span 49 feet over the Shiawassee River. The ground surface elevation near the southwest abutment is approximately 894 feet and the ground surface elevation near the northeast abutment is approximately 892 feet.

SCOPE OF SERVICES

The field operations, laboratory testing, and engineering report preparation were performed under the direction and supervision of a licensed professional engineer. Our services were performed according to generally accepted standards and procedures in the practice of geotechnical engineering in this area. Our scope of services for this project consists of the following specific items:

1. We drilled a total of two (2) soil borings to a depth of 50 feet each. Soil boring B-1 was drilled approximately 40 feet west of the southwest abutment and soil boring B-2 was performed near the northeast abutment.
2. We performed laboratory testing on samples obtained from the soil borings. Laboratory testing included visual engineering classification, natural moisture content, organic matter content (loss-on-ignition), and unconfined compressive strength determinations.
3. We prepared this engineering report. Our report includes recommendations for new bridge foundation design and construction and other construction considerations related to the proposed pedestrian bridge replacement.

FIELD OPERATIONS

G2 Consulting Group, LLC (G2) selected the number, depth, and location of the soil borings. The soil borings were located in the field by a G2 representative by measuring from existing site features and landmarks using conventional taping methods. The approximate soil boring locations are shown on the Soil Boring Location Plan, Plate No. 1. Ground surface elevations were interpolated from the plan and profile sheet prepared by OHM Advisors.

The soil borings were drilled using an All-Terrain Vehicle (ATV) mounted rotary drilling rig. Continuous flight 2-1/4 inch inside diameter hollow-stem augers were used to advance the boreholes to the explored depth. Soil samples were obtained at intervals of 2-1/2 feet within the upper 10 feet and at intervals of 5 feet thereafter. The samples were obtained by the Standard Penetration Test method (ASTM D 1586), which involves driving a 2-inch diameter split-spoon sampler into the soil with a 140-pound weight falling 30 inches. The sampler is generally driven three successive 6-inch increments with the number of blows for each increment recorded. The number of blows required to advance the sampler the last 12 inches is termed the Standard Penetration Resistance (N). The blow counts for each 6-inch increment and the resulting N-value are presented on the soil boring logs.

The soil samples were placed in sealed containers and brought to our laboratory for testing and classification. During field operations, the driller maintained logs of the subsurface conditions, including changes in stratigraphy and observed groundwater levels. The final boring logs are based on the field boring logs supplemented by laboratory soil classification and test results. The boreholes were backfilled with auger cuttings upon completion of drilling operations.

LABORATORY TESTING

Representative soil samples were subjected to laboratory testing to determine soil parameters pertinent to design and construction of the proposed pedestrian bridge. An experienced geotechnical engineer classified the samples in general conformance with the Unified Soil Classification System.

Laboratory testing included natural moisture content, organic matter content (loss-on-ignition), and unconfined compressive strength determinations. The organic matter content (loss-on-ignition, LOI) of representative samples was determined in accordance with ASTM Test Method D 2974, “Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils”. The unconfined compressive strengths were determined by using a spring loaded hand penetrometer. The hand penetrometer estimates the unconfined compressive strength to a maximum of 4-1/2 tons per square foot (tsf) by measuring the resistance of the soil sample to the penetration of a calibrated spring loaded cylinder.

The results of the moisture content, organic matter content, and unconfined compressive strength laboratory tests are indicated on the soil boring logs at the depths the samples were obtained. We will hold the soil samples for 60 days from the date of this report. If you would like the samples, please let us know.

EXISTING SITE CONDITIONS

The existing pedestrian bridge consists of 2 steel beams with a concrete deck that is 4-1/2 feet wide and spans the Shiawassee River. No information regarding the type or depth of the existing bridge abutment foundations were available upon completion of this report. The existing bridge runs northeast to southwest. Ground surface elevations on the southwest side of the bridge is approximately 894 feet and the ground surface elevation on the northeast side is approximately 891 feet. A gazebo is present near the southwest abutment. The remaining portions of Mill Pond Park is generally grass covered with some mature trees. Commercial properties are present to the north and south of Mill Pond Park. Leroy Street bounds the park to the west.

EXISTING SOIL CONDITIONS

Approximately 12 inches of silty clay topsoil are present at the ground surface of boring B-1. Granular fill soils, consisting of sand and gravelly sand, underlie the topsoil within boring B-1 and are present at the ground surface of boring B-2, and extend to depths of 6-1/2 and 9 feet, respectively. Peat underlies the granular fill within boring B-1 and extends to an approximate depth of 11 feet. Native cohesive soils, consisting of silty clay and sandy clay, are present below the peat within boring B-1 and gravelly sand fill within boring B-2, and extends to an approximate depth of 18 feet. Native granular soils, consisting of sand, gravelly sand, clayey sand, and silt, underlie the native cohesive soils within the borings and generally extend to the explored depth of 50 feet. However, native sandy clay is present within boring B-2 between depths of 23 and 36 feet.

The granular fill soils are very loose to medium compact with Standard Penetration Test (SPT) N-values ranging from 2 to 12 blows per foot (bpf) and organic matter contents ranging from 1.5 to 11.1 percent. The peat is very loose in compactness with a SPT N-value of 2 bpf, moisture contents of 45 and 60 percent, and an organic matter content of 6.5 percent. The native cohesive soils are very stiff to hard in consistency with natural moisture contents ranging from 8 to 16 percent and unconfined compressive strengths ranging from 4,000 to 9,000 pounds per square foot (psf). The native granular soils are medium compact to very compact with SPT N-values ranging from 17 bpf to 50 blows per 5 inches driven.

The stratification depths shown on the soil boring logs represent the soil conditions at the boring locations. Variations may occur between borings. Additionally, the stratigraphic lines represent the approximate boundaries between soil types. The transition may be more gradual than what is shown. We have prepared the boring logs on the basis of laboratory classification and testing as well as field logs of the soils encountered.

The Soil Boring Location Plans, Plate No. 1, and Soil Boring Logs, Figure Nos. 1 and 2, are presented in the Appendix. The soil profiles described above are generalized descriptions of the conditions encountered at the boring locations. General Notes defining the nomenclature used on the boring logs and elsewhere in this report are presented on Figure No. 3.

GROUNDWATER CONDITIONS

Groundwater observations were made during and upon completion of drilling operations. Groundwater was observed within borings B-1 and B-2 during drilling operations at an approximate depth of 4-1/2 feet, corresponding to elevations of 889-1/2 feet and 886-1/2 feet, respectively. Upon completion of drilling operations, groundwater was measured at depths ranging from 3-1/2 to 4 feet, corresponding to elevations ranging from 887-1/2 to 890 feet. Fluctuations in perched and long term groundwater levels should be anticipated due to seasonal variations and following periods of prolonged precipitation.

PEDESTRIAN BRIDGE RECOMMENDATIONS

General

It is our understanding the existing pedestrian bridge over the Shiawassee River will be replaced with a pre-fabricated structure, spanning 49 feet, supported on a 6 feet wide footing, bearing at elevation 883 feet.

Site Preparation and Earth Work Recommendations

We anticipate earthwork operations will consist of excavating the existing abutment backfill, excavating for the new bridge foundations, backfilling the new abutments, and preparing the bridge approach road subgrades. We recommend all earthwork operations be performed in accordance with comprehensive specifications and be observed in the field by qualified technical personnel working under the direction of a professional engineer.

All vegetation, any surficial or buried organic soils, and any unsuitable existing fill soils should be removed in their entirety from within the foundation and embankment area. The existing granular fill soils have organic matter contents ranging from 1.5 to 11.1 percent. The peat present within boring B-1 has an organic matter content of 6.5 percent. The peat present beneath within boring B-1 and granular fill soils present within the upper 6-1/2 to 9 feet of the soil borings are not suitable for support of the embankment fill or the proposed bridge foundations. Therefore, the granular fill soils and peat must be completely removed and replaced with engineered fill.

After excavating the existing abutment backfill and peat soils, the exposed subgrade should be visually evaluated for unstable and/or unsuitable soil conditions. Any remaining unstable or unsuitable areas noted should be removed and replaced with engineered fill. We anticipate site dewatering will be required to excavate to the required foundation depths. The dewatering system must be installed and the groundwater level lowered prior to excavation operations.

Engineered fill should be free of organic matter, frozen soil, clods, or other harmful material. We - recommend a soil, which meets the requirements of MDOT Class II granular material, be used as backfill /for the abutments. The fill soils should be placed in uniform horizontal layers that are not more than 9 inches in loose thickness. The engineered fill should be compacted to achieve a density of at least 95 percent of the maximum dry density as determined by the Modified Proctor compaction test (ASTM D 1557). All engineered fill material should be placed and compacted at approximately the optimum moisture content. Frozen soil should not be used as fill, nor should fill be placed on a frozen subgrade.

Temporary Excavation and Slope Recommendations

Groundwater was encountered at an approximate depth of 4 feet within the borings performed within the vicinity of the proposed bridge. We anticipate excavations for the proposed abutment and associated foundations will extend to depths up to 8 to 11 feet below existing grades. Therefore, in order to construct the proposed abutment and associated foundations, we recommend cofferdams be constructed and dewatered for foundation construction. Sheeting should extend through the upper granular soils and peat and into the underlying silty clay soils. We anticipate the cofferdams can be dewatered with pumping from properly constructed sumps. The sheeted cofferdams should be designed to withstand the earth pressure and hydrostatic groundwater pressure.

For cofferdam design with cantilevered shoring or shoring with a single row of bracing, a triangular distribution of lateral earth pressure may be used. It may be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 35 pcf for soils above water level. Soils below the water level should be modeled as a fluid with a density of 85 pcf. If construction traffic or material storage is allowed within 10 feet of the excavation, a uniform lateral pressure of 100 pounds per square foot should be added to the design lateral loads.

The temporary cofferdams will be required to reduce water infiltration into foundation excavations for the proposed abutments and to facilitate construction in dry conditions. The cofferdam(s) may consist of interlocking steel sheet piling. The preferred type of sheet piling consists of hot-rolled steel sheets, since the tight interlocks will not leak as much as cold-rolled steel sheet piling interlocks; however, the type of sheet piling may be left to the discretion of the contractor with the understanding that additional internal dewatering will be required for cold-rolled sheeting. Since the retained soils will not be drained, cofferdams should be designed to include a hydrostatic load as described above for shoring design.

The design of temporary cofferdams or shoring is typically the responsibility of the contractor; however, we recommend that the design of the cofferdam(s) or shoring be developed by a registered professional engineer with substantial experience in geotechnical engineering with respect to these types of structures. If the cofferdam(s) include temporary bracing, the top elevation, tip elevation, and alignment of the steel sheet piling must be designed to prevent interference with new foundation locations. G2 can provide cofferdam design services, if required.

All excavations should be safely sheeted, shored, sloped, or braced in accordance with MI-OSHA requirements. If material is stored or equipment is operated near an excavation, stronger shoring must be used to resist the extra pressure due to the superimposed loads. Care should always be exercised when excavating near existing roadways or utilities to avoid undermining.

Bridge Foundation Recommendations

We understand the elevation at the proposed bridge is 883 feet. Given the existing soil conditions present at the anticipated foundation depths, we recommend the proposed concrete bridge abutments be supported on conventional spread and strip footings bearing on the native very stiff silty clay present below the fill and peat soils. Foundations should be designed for a net allowable bearing capacity of 4,000 psf bearing in the very stiff silty clay. We recommend a G2 engineer be on site during construction to observe the excavations, measure the bearing depths, and verify the adequacy of the bearing soils.

If the recommendations outlined in this report are adhered to, total and differential settlements for the completed structure should be within 1 inch and 1/2 inch, respectively. We expect settlements of these magnitudes are within tolerable limits for the type of structure proposed.

Lateral Loads on Abutments

Conventional abutments should be designed to resist the combined loads from a triangular distribution of lateral earth pressures, a triangular distribution of hydrostatic groundwater pressures, plus uniform and point surcharges from live loads including vehicle traffic and wind. For design of relatively rigid structures, we recommend the lateral soil load be modeled using an at-rest lateral earth pressure. Based on an at-rest condition (K_o), it may be assumed that the retained soils will exert a lateral pressure equal to that developed by a fluid with a density of 60 pcf ($K_o = 0.5$). It should be assumed that the lateral loads are imposed from the top of the abutment section to the bottom of the footing.

We recommend the use of well-drained, properly compacted fill, consisting of MDOT Class II granular fill material, behind abutments. Abutment drainage should be provided along the base of the heel of abutment footings in accordance with the current approved MDOT Standard Detail.

Backfill

After construction of the proposed bridge foundations, the resulting excavations should be backfilled. The entire excavation should be backfilled with the imported engineered fill meeting the requirements of MDOT Class III granular backfill. The granular backfill should be placed in loose layers not to exceed 9 inches in thickness and should be mechanically compacted to at least 95 percent of the maximum dry density as determined by the Modified Proctor Test (ASTM D1557).

Based on visual soil classification, it was determined that no soils within the alignment of the soil borings would warrant the determination of grain-size distribution for comparison to MDOT Class III soil specifications. Granular soils observed in the soil borings generally do not meet gradation requirements of a MDOT Class III granular material and are therefore unsuitable for re-use as utility backfill.

GENERAL COMMENTS

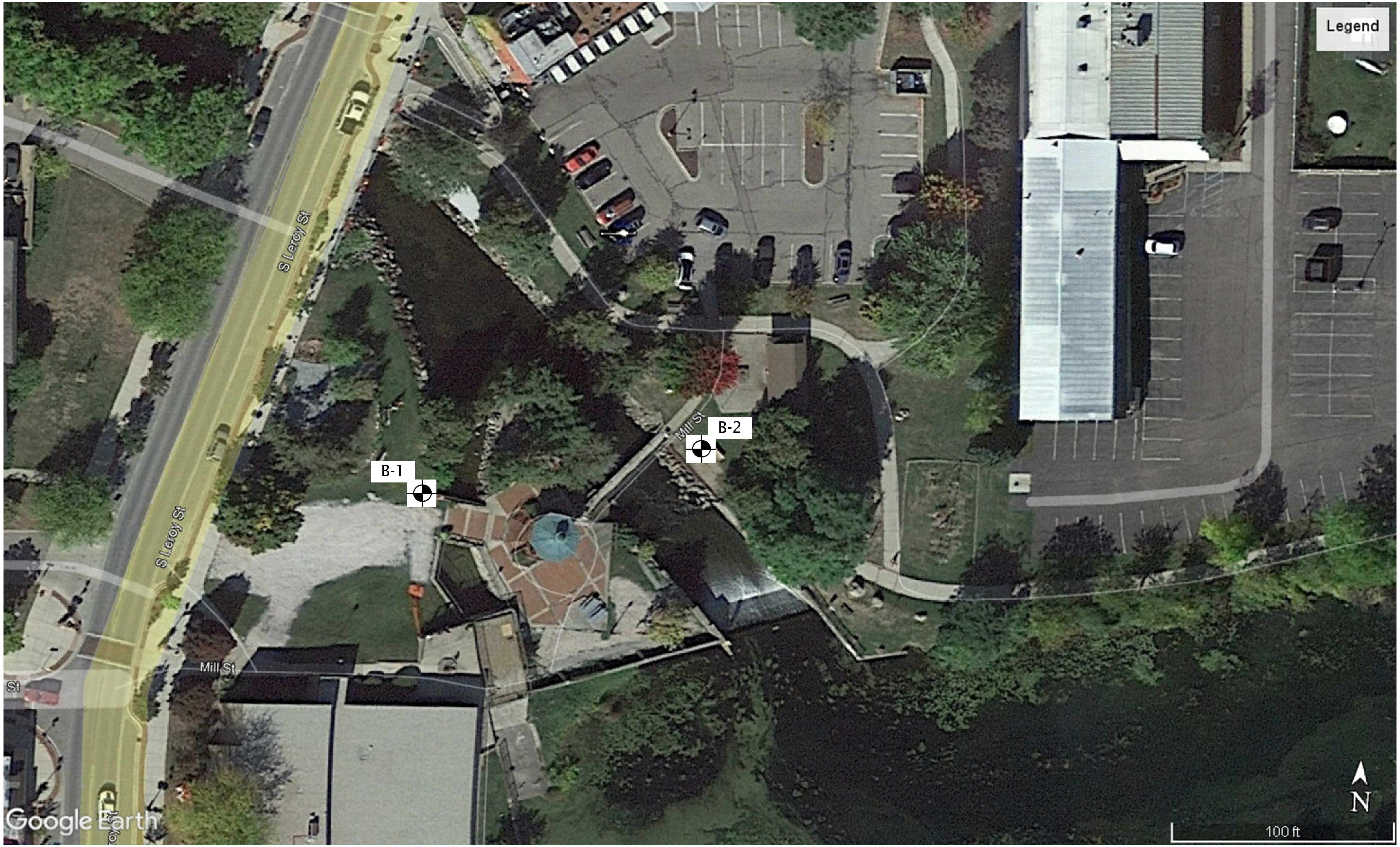
We have formulated the evaluations and recommendations presented in this report relative to site preparation and new foundation construction on the basis of data provided to us relating to the general location for the proposed pedestrian bridge. Any significant change in this data should be brought to our attention for review and evaluation with respect to the prevailing subsurface conditions.

The scope of the present investigation was limited to evaluation of subsurface conditions for the support of the new structure and other related aspects of the development. No chemical, environmental, or hydrogeological testing or analyses were included in the scope of this investigation. If changes occur in the design, location, or concept of the project, the conclusions and recommendations contained in this report are not valid unless G2 Consulting Group, LLC reviews the changes. G2 Consulting Group, LLC will then confirm the recommendations presented herein or make changes in writing.

We have based the analyses and recommendations submitted in this report upon the data from soil borings performed at the approximate locations shown on the Soil Boring Location Plans, Plate No. 1. This report does not reflect variations that may occur between the actual boring locations. The nature and extent of any such variations may not become clear until the time of construction. If significant variations then become evident, it may be necessary for us to re-evaluate our report recommendations.

Soil conditions at the site could vary from those generalized on the basis of soil borings made at specific locations. It is, therefore, recommended that G2 Consulting Group, LLC be retained to provide soil engineering services during the site preparation, foundation, and pavement construction phases of the proposed project. This is to observe compliance with the design concepts, specifications, and recommendations. Also, this allows design changes to be made in the event that subsurface conditions differ from those anticipated prior to the start of construction.

APPENDIX



Legend

Soil Borings performed by Strata Drilling, Inc. on October 27, 2023

Soil Boring Location Plan

Mill Pond Park Pedestrian Bridge
301 S. Leroy Street
Fenton, Michigan 48430

2G CONSULTING GROUP

Project No. 230607	
Drawn by: JMH	
Date: 2/5/24	Plate No. 1
Scale: NTS	

Project Name: Mill Pond Park Pedestrian Bridge

Project Location: 301 S. Leroy Street
Fenton, Michigan

G2 Project No. 230607

Latitude: 42.794294° Longitude: -83.704763°

Soil Boring No. **B-1**

CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA					
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 894.0 ft ±	DEPTH (ft)	SAMPLE TYPE-NO.	BLOWS/ 6-INCHES	STD. PEN. RESISTANCE (N)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. STR. (PSF)
		Topsoil: Dark Brown Silty Clay (12 inches) 1.0		S-1	4 5 4	9	23.2		
889.0		Fill: Very Loose to Loose Black Gravelly Sand with trace silt, clay, and organic matter (Organic Matter Content = 8.7% - 11.1%) 6.5	5	S-2	1 1 1	2	35.9		
				S-3	1 1 1	2	44.9		
884.0		Peat: Very Loose Black Silt (Organic Matter Content = 6.5%) 11.0	10	S-4	4 1 1	2	59.7		
879.0		Very Stiff Gray Silty Clay with trace sand and gravel 18.0	15	S-5	4 5 8	13	14.3		4000*
874.0		Compact Gray Silty Sand with trace clay and gravel 23.0	20	S-6	16 16 23	39			
869.0		Medium Compact Gray Gravelly Sand with trace silt 27.0	25	S-7	9 8 9	17			
864.0		Very Compact Gray Clayey Sand with trace silt and gravel 33.0	30	S-8	48 50/5"	---			
859.0		Compact to Very Compact Gray Gravelly Sand with trace silt	35	S-9	12 21 29	50			

Total Depth: 50 ft
Drilling Date: October 27, 2023
Inspector:
Contractor: Strata Drilling, Inc.
Driller: B. Sienkiewicz

Drilling Method:
2-1/4 inch inside diameter hollow-stem augers

Water Level Observation:
4-1/2 feet during; 4 feet upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Auger cuttings

Figure No. 1a

Project Name: Mill Pond Park Pedestrian Bridge

Project Location: 301 S. Leroy Street
Fenton, Michigan

G2 Project No. 230607

Latitude: 42.794294° Longitude: -83.704763°

Soil Boring No. **B-1**

G2 CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA					
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 894.0 ft ±	DEPTH (ft)	SAMPLE TYPE-NO.	BLOWS/ 6-INCHES	STD. PEN. RESISTANCE (N)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. STR. (PSF)
854.0		Compact to Very Compact Gray Gravelly Sand with trace silt *(continued)*	40	S-10	8 14 28	42			
849.0			45	S-11	12 28 40	68			
		48.0							
844.0		Very Compact Gray Silt with trace clay, sand, and gravel 50.0	50	S-12	14 31 55	86			
		End of Boring @ 50 ft							
839.0			55						
834.0			60						
829.0			65						
824.0			70						

Total Depth: 50 ft
Drilling Date: October 27, 2023
Inspector:
Contractor: Strata Drilling, Inc.
Driller: B. Sienkiewicz

Drilling Method:
2-1/4 inch inside diameter hollow-stem augers

Water Level Observation:
4-1/2 feet during; 4 feet upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Auger cuttings

Figure No. 1b

Project Name: Mill Pond Park Pedestrian Bridge

Project Location: 301 S. Leroy Street
Fenton, Michigan

G2 Project No. 230607

Latitude: 42.794376° Longitude: -83.704316°

Soil Boring No. **B-2**

G2 CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA					
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 891.0 ft ±	DEPTH (ft)	SAMPLE TYPE-NO.	BLOWS/ 6-INCHES	STD. PEN. RESISTANCE (N)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. STR. (PSF)
		Fill: Loose Brown Sand with trace silt and gravel		S-1	2 2 3	5			
886.0		4.5 Fill: Very Loose Black Gravelly Sand with trace clay, silt, and organic matter (Organic Matter Content = 7.6%)	5	S-2	1 1 3	4	41.2		
		6.0 Fill: Medium Compact Black Gravelly Sand with trace clay, silt, and organic matter (Organic Matter Content = 1.5%)		S-3	2 4 8	12	13.8		
881.0		9.0 Very Stiff Gray Silty Clay with trace sand and gravel	10	S-4	14 8 7	15	10.6		4500*
876.0		13.0 Hard Gray Sandy Clay with trace silt and gravel	15	S-5	11 20 31	51	10.3		9000*
871.0		18.0 Very Compact Gray Clayey Sand with trace silt and gravel	20	S-6	8 18 40	58			
866.0		23.0 Hard Gray Sandy Clay with trace silt and gravel	25	S-7	14 26 48	74	15.7		9000*
861.0			30	S-8	37 50/5"	---	15.5		9000*
856.0			35	S-9	31 50/5"	---	8.4		9000*

Total Depth: 50 ft
Drilling Date: October 27, 2023
Inspector:
Contractor: Strata Drilling, Inc.
Driller: B. Sienkiewicz

Drilling Method:
2-1/4 inch inside diameter hollow-stem augers

Water Level Observation:
4-1/2 feet during; 3-1/2 feet upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Auger cuttings

Figure No. 2a

Project Name: Mill Pond Park Pedestrian Bridge

Project Location: 301 S. Leroy Street
Fenton, Michigan

G2 Project No. 230607

Latitude: 42.794376°   Longitude: -83.704316°

Soil Boring No. **B-2**

G2 CONSULTING GROUP

SUBSURFACE PROFILE / SOIL SAMPLE DATA

GROUND SURFACE ELEVATION: 891.0 ft ±

ELEV. (ft)	GROUND SURFACE ELEVATION: 891.0 ft ±	DEPTH (ft)	SAMPLE TYPE-NO.	BLOWS/ 6-INCHES	STD. PEN. RESISTANCE (N)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. STR. (PSF)
	36.0							
851.0	Compact Gray Gravelly Sand with trace silt	40	S-10	14 19 25	44			
846.0		45	S-11	16 23 27	50			
	48.0							
841.0	Very Compact Gray Sandy Silt with trace clay and gravel (50.0)	50	S-12	118 36 43	79			
	End of Boring @ 50 ft							
836.0		55						
831.0		60						
826.0		65						
821.0		70						

Total Depth: 50 ft
Drilling Date: October 27, 2023
Inspector:
Contractor: Strata Drilling, Inc.
Driller: B. Sienkiewicz

Drilling Method:
2-1/4 inch inside diameter hollow-stem augers

Water Level Observation:
4-1/2 feet during; 3-1/2 feet upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Auger cuttings

Figure No. 2b

C2G CONSULTING GROUP

GENERAL NOTES TERMINOLOGY

Unless otherwise noted, all terms herein refer to the Standard Definitions presented in ASTM 653.

PARTICLE SIZE

Boulders		- greater than 12 inches
Cobbles		- 3 inches to 12 inches
Gravel	- Coarse	- 3/4 inches to 3 inches
	- Fine	- No. 4 to 3/4 inches
Sand	- Coarse	- No. 10 to No. 4
	- Medium	- No. 40 to No. 10
	- Fine	- No. 200 to No. 40
Silt		- 0.005mm to 0.074mm
Clay		- Less than 0.005mm

CLASSIFICATION

The major soil constituent is the principal noun, i.e. clay, silt, sand, gravel. The second major soil constituent and other minor constituents are reported as follows:

Second Major Constituent (percent by weight)	Minor Constituent (percent by weight)
Trace - 1 to 12%	Trace - 1 to 12%
Adjective - 12 to 35%	Little - 12 to 23%
And - over 35%	Some - 23 to 33%

COHESIVE SOILS

If clay content is sufficient so that clay dominates soil properties, clay becomes the principal noun with the other major soil constituent as modifier, i.e. sandy clay. Other minor soil constituents may be included in accordance with the classification breakdown for cohesionless soils, i.e. silty clay, trace sand, little gravel.

Consistency	Unconfined Compressive Strength (psf)	Approximate Range of (N)
Very Soft	Below 500	0 - 2
Soft	500 - 1,000	3 - 4
Medium	1,000 - 2,000	5 - 8
Stiff	2,000 - 4,000	9 - 15
Very Stiff	4,000 - 8,000	16 - 30
Hard	8,000 - 16,000	31 - 50
Very Hard	Over 16,000	Over 50

Consistency of cohesive soils is based upon an evaluation of the observed resistance to deformation under load and not upon the Standard Penetration Resistance (N).

COHESIONLESS SOILS

Density Classification	Relative Density %	Approximate Range of (N)
Very Loose	0 - 15	0 - 4
Loose	16 - 35	5 - 10
Medium Compact	36 - 65	11 - 30
Compact	66 - 85	31 - 50
Very Compact	86 - 100	Over 50

Relative Density of cohesionless soils is based upon the evaluation of the Standard Penetration Resistance (N), modified as required for depth effects, sampling effects, etc.

SAMPLE DESIGNATIONS

AS - Auger Sample – Cuttings directly from auger flight
BS - Bottle or Bag Samples
S - Split Spoon Sample - ASTM D 1586
LS - Liner Sample with liner insert 3 inches in length
ST - Shelby Tube sample - 3 inch diameter unless otherwise noted
PS - Piston Sample - 3 inch diameter unless otherwise noted
RC - Rock Core - NX core unless otherwise noted

STANDARD PENETRATION TEST (ASTM D 1586) - A 2.0 inch outside-diameter, 1-3/8 inch inside-diameter split barrel sampler is driven into undisturbed soil by means of a 140-pound weight falling freely through a vertical distance of 30 inches. The sampler is normally driven three successive 6-inch increments. The total number of blows required for the final 12 inches of penetration is the Standard Penetration Resistance (N).