APPENDIX 4

PRELIMINARY GEOTECHNICAL REPORT

THURBER

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT MUNICIPAL CLASS ENVIORNMENTAL ASSESSMENT FOR MACDONELL AND ALLAN'S STRUCTURES GUELPH REVITALIZATION PROJECT

CITY OF GUELPH, ONTARIO

Report

to

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APPENDICES

1. INTRODUCTION

This report presents the factual findings obtained from a preliminary geotechnical and hydrogeological investigation conducted in support of the Macdonell and Allan's Structures Municipal Class Environmental Assessment (EA) Study, which is part of the Guelph Revitalization Project in the City of Guelph, Ontario.

The Macdonell Street corridor is presently a multi-lane roadway crossing the Speed River. Current plans call for the improvements and modifications to the Macdonell and Allan's Structures and surrounding area in the Macdonell corridor at Speed River to either replace or rehabilitate the existing Macdonell and Allan's Structures and facilitate the City's proposed Downtown Infrastructure Revitalization Program. Thurber Engineering Ltd. (Thurber) carried out the investigation as a sub-consultant to R.V. Anderson Associates Limited (RVA) who are conducting the EA Study for the City of Guelph.

The purpose of the geotechnical investigation was to explore the subsurface conditions within the project limits and based on the data obtained, to provide borehole logs, borehole location plans, a written description of the subsurface conditions, and preliminary geotechnical comments and recommendations in support of the design and construction of the any proposed structure upgrades and road improvements.

The purpose of the hydrogeological investigation was to assess the groundwater conditions at the Site, potential water well and aquifer impacts and mitigation measures, and construction dewatering requirements. It is noted this report will be revised to include hydrogeological recommendations at a later time once further is design inputs are available.

The scope of work did not include the completion of environmental quality testing to assess options for management options for excess excavated soils that may be generated during the proposed construction works. It is understood that such testing will be completed at later stages of the project.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement For Use and Interpretation of Report.

2. BACKGROUND INFORMATION

2.1 Site Description

The study area encompasses Macdonell Street Bridge at Speed River and surrounding intersections. The approximate limits of the Site are shown on the Borehole Plan included in Appendix A.

The existing Macdonell Street bridge is located between Woolwich Street and Elizabeth Street approximately 500 m east of downtown Guelph. The bridge runs in a northeast-southwest direction and carries four lanes of Macdonell Street traffic over Speed River. Based on archived drawings, the existing bridge is a two-span reinforced concrete rigid frame supported on spread footings with wingwalls extending towards the north and south from the ends of abutments.

The roadway at the bridge presently consists of an urban cross section with concrete sidewalks. The posted speed limit is 50 km/h.

There are presently residential subdivisions to the east of the bridge site and condominiums and commercial properties to the west. There is also a historic dam and bridge (Allan's Street bridge) located south of the existing Macdonell Street bridge as well as a Guelph Junction Railway (GJR) track located immediately west of the existing bridge which crosses Speed River to the south. Additionally, there is an existing overhead steel rail bridge structure immediately south of the existing bridge which carries two Canadian Nation Railway (CNR) tracks over Wellington Street, the GJR track, Speed River and Elizabeth Street.

Typical photographs of the Site are provided in Appendix F.

A historic General Arrangement drawing provided by RVA shows the regulated water level in Speed River at Elev. 315.6 m.

2.2 Geology

Based on the information in The Physiography of Southern Ontario[1] by Chapman and Putnam (1984), the site lies within an area referred to as the Guelph Drumlin Field, an area of drumlinized till plain, also mapped as containing eskers. The till is described as stony and the occurrence of surface boulders is noted. Chapman and Putnam give a typical gradation of the till as being 50% sand, 35% silt and 15% clay. Swampy valleys are reported to occur between the drumlins and

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[1] Chapman, L.J. and Putnam, D.F. 1984. The Physiography of Southern Ontario, Ontar]io Geological Survey Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

associated gravel terraces. Large sand and gravel deposits occur in outwash plains, kames eskers, and extensive spillway terraces.

According to *Paleozoic Geology of Southern Ontario*[2], the Site is located in a paleozoic area known as the Guelph Formation. This formation is comprised of buff to cream-colored crystalline dolostone. The Guelph Formation bedrock is noted as soft and easy to quarry due to its high concentration of magnesium. It has a thickness of over 30 m (100 ft) but generally thins towards the north, and outcrops along the Speed River north and south of Guelph, and along the Grand River and Irvine Creek at Elora in high cliffs.

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

The field investigation for this project was carried out on July 20th, July 21st and July 30th, 2021, and comprised a total of nine (9) boreholes (Boreholes 21-01 to 21-08 and 21-05C) advanced to depths ranging from 1.4 to 8.9 m. Borehole details are provided in Table 3.1 and in the Record of Borehole sheets included in Appendix B. The approximate locations of the boreholes are shown on the Borehole Location Plan included in Appendix A. The Records of Borehole sheets are provided in Appendix B.

Table 3.1 – Borehole Details

Borehole No.	Ground Elevation (m)	Borehole Termination Depth (m)	Borehole Termination Elevation (m)
21-01	318.2	2.9	315.3
21-02	318.2	2.1	316.1
21-03	318.1	2.4	315.7
21-04	318.1	8.9	309.2
21-05	317.5	1.4	316.1
21-05C	317.5	3.8	313.7
21-06	318.9	2.5	316.4
21-07	317.8	3.5	314.3
21-08	321.4	6.3	315.1

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[2] Armstrong, D.K. and Dodge, J.E.P., 2007: Paleozoic geology of southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 219.

The ground surface elevations and coordinates of the borehole locations were determined using a Trimble R10 GNSS receiver.

All borehole locations were cleared of utilities prior to commencement of drilling. The boreholes were repositioned in the field as necessary in consideration of surface features, underground utilities, and overhead wires.

The boreholes were advanced using solid stem augers powered by a truck mounted B-57 drill rig supplied and operated by Landshark Drilling of Brantford, Ontario. Soil samples were obtained at selected intervals using a 50 mm outside diameter split-spoon sampler driven in conjunction with the Standard Penetration Test (SPT). The field investigation was supervised on a full-time basis by a member of Thurber's technical staff who marked/staked the boreholes in the field, arranged for the clearance of subsurface utilities, directed the drilling, sampling and in-situ testing operations, logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions were observed in the open boreholes throughout the drilling operations. Monitoring wells were installed in Boreholes 21-01, 21-04, 21-05C and 21-06 to permit monitoring of the groundwater levels at the site. The monitoring wells consisted of 50 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the boreholes. The installation details are summarized in the table below.

Table 3.2 – Monitoring Well Details

Borehole No.	Monitoring Well Tip		Slotted Screen Length (m)
	Depth (m)	Elevation (m)	
21-01	2.9	315.3	1.5
21-04	8.9	309.2	3.0
21-05C	3.8	313.7	1.5
21-06	2.5	316.4	1.5

The boreholes in which no monitoring wells were installed were backfilled in general accordance with Ontario Regulation 903,as amended. Boreholes advanced through the road surface were reinstated and resurfaced with cold patch asphalt.

3.2 Laboratory Testing

The recovered soil samples were subjected to visual identification (VI) and to natural moisture content determination. Selected samples were subjected to grain size distribution analyses (sieve and/or hydrometer) and Atterberg Limits testing. Geotechnical laboratory testing results are summarized on the Record of Borehole sheets included in Appendix B and are presented on the figures included in Appendix C.

Selected soil samples were also submitted for analytical testing to assess the potential for soil corrosion including the potential for sulphate action on concrete. The analyses were carried out by SGS North America Inc., an independent Canadian Association for Laboratory Accreditation (CALA) accredited laboratory. The results of the analytical testing are presented in Appendix D.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

A generalized description of the subsurface conditions encountered in the boreholes is given in the following sections. Detailed descriptions of the soil conditions at the specific locations drilled are presented on the Record of Borehole sheets in Appendix B and take precedence over the generalized description. It should be recognized and expected that soil conditions will vary between and beyond borehole locations.

In general, the subsurface stratigraphy encountered in the boreholes consist of surficial asphalt overlying fill layers underlain by native deposits of silty sand till and clayey silt to silty clay till. These overburden materials are underlain by dolostone bedrock. Further description of the individual strata are presented below.

4.1 Asphalt

Asphalt was encountered at the ground surface in all of the boreholes. The thickness of the asphalt ranged from 75 mm to 250 mm.

4.2 Granular Fill

Granular fill was encountered underlying the asphalt in all boreholes. The granular fill generally consisted of sand and gravel fill containing trace to some silt. Trace peat and occasional brick fragments were noted within the granular fill in Borehole 21-06 and cobbles were noted within this fill in Borehole 21-07. Brown sand and silt with variable amounts of gravel and trace clay was encountered underlying the asphalt in Borehole 21-07.

The thickness of this fill ranged from 0.6 m to 3.9 m and the base of this fill was encountered at depths between 0.8 m and 4.1 m (Elev. 320.6 m and 314.0). The depth of the base of the fill was typically 1.5 m below ground surface.

SPT ‘N’ values obtained in the granular fill ranged from 20 to 89 blows per 0.3 m penetration, indicating a compact to very dense relative density. Measured moisture contents ranged from 2 to 9%.

The results of grain size distribution tests conducted on selected samples of the granular fill are presented on Figure C1 of Appendix C and summarized below:

Soil Particle	Percentage (%)	
	Sand & Gravel	Sand & Silt
Gravel	38 to 46	11
Sand	41 to 54	39
Silt	-	42
Clay	-	8
Silt + Clay	8 to 13	-

4.3 Silty Clay Fill

Brown silty clay fill was encountered underlying the granular fill in Borehole 21-03. The silty clay fill is described as sandy with trace gravel.

The top of the silty clay fill was encountered at a depth of 0.9 m (Elev. 317.2 m) and the fill extended to a depth of 2.4 m (Elev. 315.7 m) where auger refusal was encountered.

SPT ‘N’ values obtained in the silty clay fill ranged from 2 to 3 blows per 0.3 m, indicating a soft consistency. Measured moisture contents ranged from 13 to 19%.

The results of a grain size distribution test carried out on a selected sample of the silty clay fill are shown on Figure C2 in Appendix C and summarized below:

Soil Particle	Percentage (%)
Gravel	1
Sand	23
Silt	55
Clay	21

The results of an Atterberg Limits test carried out on a sample of the silty clay fill are shown on Figure C6 in Appendix C and summarized below:

Soil Property	Percentage (%)
Liquid Limit	27
Plastic Limit	15
Plasticity Index	12

The results of the Atterberg Limit testing indicate that the silty clay fill has low plasticity (CL).

4.4 Gravelly Sand Fill

Brown to grey gravelly sand fill was encountered in underlying the granular fill in Boreholes 21-04, 21-05C, 21-06, and 21-07. The sand fill generally contained varying amounts of silt and gravel and also contained trace to some clay.

The thickness of this fill ranged from 1.1 m to 2.9 m and the base of this fill was encountered at depths between 2.3 m and 7.0 m (Elev. 316.4 m and 311.0 m).

SPT ‘N’ values obtained in the gravelly sand fill were highly variable and ranged from 2 blows per 0.3 m penetration to 100 blows per 0.075 m penetration with most values between 2 and 13 indicating a typical very loose to compact relative density. Measured moisture contents ranged from 6 to 23%.

The results of grain size distribution tests carried out on selected samples of the gravelly sand fill are shown on Figure C3 in Appendix C and summarized below:

Soil Particle	Percentage (%)
Gravel	2 to 32
Sand	43 to 76
Silt	9 to 44
Clay	0 to 11

4.5 Silty Sand Till

Brown silty sand till was encountered underlying the granular fill in Boreholes 21-01 and 21-02. The till is described as gravelly with trace clay. Occasional dolostone fragments were noted in the till in Borehole 21-01.

The thickness of the silty sand till ranged from 0.5 m to 1.7 m and the base of the till extended to depths ranging from 2.0 m to 4.0 m (Elev. 317.4 m to 315.6 m).

SPT ‘N’ values obtained in the silty sand till ranged from 17 blows per 0.3 m penetration to 60 blows per 0.1 m penetration, indicating a compact to very dense relative density. Measured moisture contents ranged from 5 to 9%.

The results of a grain size distribution test carried out on a sample of the silty sand till are shown on Figure C4 in Appendix C and summarized below:

Soil Particle	Percentage (%)
Gravel	25
Sand	48
Silt + Clay	27

Till soils frequently contain cobbles and boulders, and these should be anticipated when excavating during construction.

4.6 Clayey Silt to Silty Clay Till

A deposit of brown to grey clayey silt to silty clay till, sandy to some sand, was encountered in Borehole 21-08 underlying the granular fill at a depth of 0.8 m (Elev. 320.6 m) and below the silty sand till at a depth of 4.0 m (Elev. 317.4 m)

SPT ‘N’ values of 22 to 73 blows per 0.3 m of penetration were recorded in the till, indicating a very stiff to hard consistency. Moisture contents of 7 to 11% were measured.

The result of a grain size analysis conducted on a sample of the silty sandy clay are presented on Figure C5 of Appendix C. The result of the grain size distribution analysis is summarized below:

Soil Particle	Percentage (%)
Gravel	0 to 1
Sand	13 to 19
Silt	48 to 68
Clay	18 to 33

Till soils frequently contain cobbles and boulders, and these should be anticipated when excavating during construction.

4.7 Dolostone Bedrock

Highly to completely weathered dolostone bedrock was encountered underlying the silty sand till in Boreholes 21-01 and 21-02 at depths ranging from 2.0 m to 2.6 m (Elev. 316.2 m to 315.6 m), and below the sand fill in Borehole 21-04 at a depth of 7.0 m (Elev. 311.0 m). The bedrock was not proven by coring.

4.8 Groundwater Levels

Standpipe piezometers were installed in Boreholes 21-01, 21-04, 21-05C, and 21-06 to permit groundwater monitoring at the site.

The groundwater depths and elevations measured in the piezometers installed in the boreholes are summarized in Table 4.1.

Table 4.1 – Summary of Water Level Measurements

Borehole	Date	Water Level (m)		Remark
		Depth	Elevation	
21-01	Aug 11, 2021 Aug 18, 2021	Dry Dry	- -	Piezometer
21-04	Aug 11, 2021 Aug 18, 2021	4.3 5.1	313.8 313.0	Piezometer
21-05C	Aug 11, 2021 Aug 18, 2021	2.3 2.3	315.2 315.2	Piezometer
21-06	Aug 11, 2021 Aug 18, 2021	Dry Dry	- -	Piezometer

In general, the water levels in Boreholes 21-04 and 21-05C near the Speed River are expected to be governed by the prevailing water level in the river. A historic GA drawing dated May 14, 1963, shows the regulated water level in the Speed River at Elev. 315.6 m.

The above groundwater level measurements are short-term observations and seasonal fluctuations of the groundwater levels are to be expected. Further, groundwater levels may be higher after prolonged periods of precipitation and in the spring following snow melt.

5. CORROSIVITY AND SULPHATE TEST RESULTS

Samples of the fill sand and gravel and native gravelly silty sand till from Boreholes 21-04 and 21-05C, respectively, were submitted for analytical testing of corrosivity parameters and sulphate. The laboratory certificates of analysis for the current investigation are presented in Appendix D. The results of the analytical tests are summarized below in Table 5.1.

Table 5.1 – Analytical Test Results

Parameter	Units (Soil)	Units (Water)	Test Results	
			21-04, SS4 (Depth = 4.6 – 5.2 m)	21-05C, SS3 (Depth = 2.3 – 2.9 m)
			Granular Fill	Gravelly Sand Fill
Corrosivity Index	-	N/A	13	14
Redox Potential	mV	mV	266	202
Sulphide	%	µg/L	<0.04	<0.04
pH	-	-	9.08	9.39
Chloride	µg/g	mg/L	2600	630
Sulphate	µg/g	mg/L	62	27
Conductivity	uS/cm	µS/cm	4130	1160
Resistivity	ohm-cm	ohm-cm	242	863

6. ENGINEERING DISCUSSION AND RECOMMENDATIONS

This section of the report provides preliminary geotechnical recommendations for design and construction of the roadway improvements and structure foundations. The recommendations are based on the subsurface soil and groundwater conditions encountered during the preliminary investigation. The soil conditions may vary between and beyond the borehole locations. Additional investigation will be required during the detailed design stage to supplement the subsurface information and confirm the preliminary recommendations.

6.1 Preliminary Pavement Design

6.1.1 Design Analysis

Traffic projections were provided by RVA for Macdonell Street, Woolwich Street, and Wellington Street, and are summarized Table 6.1. It is understood that 2025 is the estimated year for the construction completion of the roads within the study area. Traffic data on Macdonell Street was applied to Rose Street, Elizabeth Street, and adjacent local roads, as traffic information was not provided for these facilities.

Table 6.1 – Project Traffic Volumes (AADT)

Section	AADT (2021)	Forecasted AADT (2025)	Truck Traffic %
Macdonell Street	17,000	17,690	2.0 %
Woolwich Street	14,000	14,568	2.0 %
Wellington Street	18,500	19,251	2.0 %

The above volumes were forecasted with a growth rate of 1.0 % to calculate traffic volumes over a 20-year design period. The traffic data was used to determine the pavement damage caused by the anticipated traffic volumes over the design life of the pavement. Using axle load equivalency factors, different axle loads and axle groups are converted to a standard axle load known as an Equivalent Single Axle Loads (ESALs). The Design ESALs calculation was completed in accordance with the MTO *Procedures for Estimating Traffic Loads for Pavement Designs.*

Assuming an average truck factor of 2.0, the number of ESALs during a 20-year design period was computed to be 2.85 million for Macdonell Street, and 2.34 million for Woolwich Street, and 3.10 million for Wellington Street. Considering the close calculated ESALs for three road sections, the higher ESALs with 3.10 million was used in the traffic analysis.

The pavement design analysis was carried out using the methodology outlined in the 1993 AASHTO "*Guide for the Design of Pavement Structures*", as modified by the Ministry's "*Adaptation and Verification of AASHTO Pavement Design Guide for Ontario Conditions*", and the MTO "*Pavement Design and Rehabilitation Manual*". This analysis was completed to determine the structural requirements for the pavement at the proposed grade separation. The AASHTO procedure for the design of flexible pavement determines a Structural Number that characterizes the structural capacity of the pavement layers for a given set of inputs. The following design inputs were used in the AASHTO design analysis.

- Design Period = 20 years
- Initial serviceability, (Pi) = 4.5
- Terminal serviceability (Pt) = 2.5
- Reliability level ® = 85 percent
- Overall standard of deviation (So) = 0.44
- Mean soil resilient modulus (MR) =30 MPa

The structural and drainage coefficients applied and provided Table 6.2. Detailed results of the pavement design analysis are provided in Appendix G.

Table 6.2 – Structural and Drainage Coefficient

Pavement Layer	Structural Coefficient	Drainage Coefficient
New Hot Mix Asphalt	0.42	1.0
New Granular Base Material	0.14	1.0
New Granular Subbase Material	0.09	1.0

Based on the design input parameters and calculated ESALs, design structural number (SN_{Des}) was calculated. The recommended pavement design thickness, based on the structural requirements, traffic projections, and subgrade conditions, is presented below.

6.1.2 Preliminary Pavement Design Recommendations

Based on the analysis presented above, the following new pavement design can be used to support the anticipated traffic over 20-year design life where grade changes are required, or if existing base and subbase are found to be inadequate based on future study:

Table 6.3 – Preliminary Pavement Design

Component and Asphalt Thickness	Granular A	Granular B Type I
40 mm HL1 100 mm HL8	150	475

If the reinstatement of the asphalt layers and approach slab is required, the asphalt pavement should have a minimum thickness of 140 mm with new HMA to support the anticipated traffic. However, based on existing asphalt thicknesses, the preliminary recommended asphalt reinstatement is provided in Table 6.4.

It should be noted that the preliminary pavement designs for reinstatement purposes were developed based on the analysis presented above, including limited testing of existing granular materials, and assuming adequate drainage of the pavement materials. Assessment of base and subbase material quality would be completed as a final design task.

Table 6.4 – Preliminary Pavement Reinstatement Design

Facilities	**Asphalt Thickness**	**Component and Thickness**
Macdonell Street	200 mm	50 mm HL1 75 mm HL8 75 mm HL8
Woolwich Street	150 mm	50 mm HL1 100 mm HL8
Wellington Street and the remaining local roads	140 mm	40 mm HL1 100 mm HL8

It can be expected that minor grading of the underlying granular base may be required in all reinstatement areas prior to the placement of the new HMA.

The pavement design thicknesses should be reviewed during detailed design.

All Hot Mix Asphalt (HMA) materials should meet the requirements of OPSS.MUNI 310 and OPSS.MUNI 1150 specifications as applicable. All asphalt lifts should be placed and compacted to levels between 92 and 96.5 percent of the Maximum Relative Density (MRD).

Based on the estimated 20-year design ESAL, Traffic Category C for should be used for all asphalt mix designs. The recommended asphalt cement grade for surface mix in Traffic Category C should be PG 58-28 should be used for all the binder mixes. Consideration should be given to further upgrading of the PGAC grade to PG 64-28 if rutting has been experienced in other sections of this roadway due to truck traffic. Aggregates for the asphalt mixes should be in accordance with OPSS.MUNI 1003. Recycled Asphalt Pavement (RAP) material is not permitted in the production of new asphalt mixes.

Granular material is not required but, where new granular base/subbase material is needed, it should consist of OPSS Granular A and OPSS Granular B Type I material. All new granular material should meet the requirements of OPSS 1010 specifications, and be compacted to 100 percent of the Standard Proctor Maximum Dry Density (SPMDD) within 2 percent of Optimum Moisture Content (OMC). All granular material should be compacted in accordance with the requirements of OPSS.MUNI 501, and should be carried the entire width of the roadway platform to maintain appropriate drainage.

Smooth transitions are required in all areas where the new pavement meets the existing asphalt surface. All longitudinal and transverse joints should meet the requirements of OPSS.MUNI 310. All longitudinal joints should be staggered between the asphalt lifts, accomplished by offsetting the paving edge and the upper asphalt course by a minimum of 150 mm. At all transverse tie-ins to existing pavements, the top lift of asphalt should extend a minimum of 5 m in length beyond the transverse joint in the upper binder lift. A tack coat shall be utilized between all asphalt lifts, all vertical faces, and at all tie-ins to existing pavement.

6.2 Preliminary Foundation Design

6.2.1 Macdonell Street Bridge

The existing Macdonell Street Bridge may require rehabilitation and/or replacement as part of the roadway reconstruction project. No details regarding the proposed rehabilitation and/or replacement have been provided as of the date of this report. The preliminary recommendations provided below will need to be reassessed at the detailed design stage following completion of additional boreholes at the site.

The subsurface conditions encountered in Boreholes 21-04 and 21-05C advanced near the likely location of the east and west abutments, respectively, consisted of surficial asphalt and granular fill layers overlying gravelly sand fill above highly to completely weathered dolostone bedrock. The top of the bedrock was encountered at the east abutment in Borehole 21-04 at a depth of 7.0 m (Elev. 311.0 m). Top of bedrock was not encountered at the west abutment in Borehole 21-05C which met auger refusal at a depth of 3.8 m (Elev. 313.7 m). Depth to competent bedrock must be confirmed by rock coring during final design.

The water level measured in a monitoring well installed at the east abutment ranged from 4.3 m to 5.1 m below ground surface (Elev. 313.8 m to 313.0 m). The water level measured at the east abutment was at a depth of 2.3 m (Elev. 315.2 m). In general, the water level at the bridge is expected to be at approximately the same elevation as the river level (Elev. 315.6 m).

6.2.1.1 Spread Footings on Bedrock

Based on the borehole data, spread footings founded on dolostone bedrock are considered a suitable option for supporting the future bridge. For the purposes of preliminary design, spread footings bearing on competent dolostone bedrock may be designed using a Factored Geotechnical Resistance at ULS of 2,000 kPa. Competent bedrock was encountered in BH 21-04 at a depth of approximately 7.5 m below grade (elev. 310.6 m). The SLS condition will not govern for footings founded on competent bedrock. The recommended geotechnical resistances are based on a minimum 2 m wide footing subjected to vertical concentric loading. Where eccentric or inclined loads are applied, the resistance values used in design must be reduced in accordance with the CHBDC Clause 6.10.3 and Clause 6.10.4.

The historic GA shows that west abutment of the existing bridge is founded at a depth of approximately 5.9 m (Elev. 311.8 m) and the east abutment of the bridge is founded at a depth of approximately 8.1 m (Elev. 310. 1 m). The pier of the existing bridge is founded at approximately Elev. 307.5 m. It is recommended that the spread footings for the new replacement structure be founded at or below the same elevation as the existing bridge.

All sediment, cobbles, boulders and loose fragments of rock must be removed from the bearing surface prior to constructing the footings. Foundation bearing surfaces should be inspected by qualified geotechnical personnel. To prevent softening and degradation of the highly to completely weathered limestone, exposed bearing surfaces must be protected by placement of a mud slab within 24 hours of completion.

The depth of frost penetration at this site is approximately 1.4 m in accordance with OPSD 3090.101. All spread footings should be provided with a minimum of 1.4 m of soil cover or

equivalent insulation as protection against frost action.

The lateral resistance developed along the base of cast-in place concrete footings founded on the bedrock may be computed using an ultimate friction coefficient of 0.65.

6.2.1.2 Micropiles

It is our understanding that the use of micropiles is being considered for the proposed pier to avoid deep excavations near the existing dam structure located on the south side of the pier.

Micropiles socketed into the weathered dolostone bedrock are considered a feasible option to provide foundation support.

For micropile design and construction recommendations, reference can be made to the U.S. Federal Highway Administration (FHWA) Reference Manual titled “Micropile Design and Construction”, Publication No. FHWA NHI-05-039 dated December 2005.

The cross section of a typical micropile consists of a steel reinforcing rod, grout body and a steel casing. The steel casing serves dual purposes of increasing the lateral load capacities and prevention of hole cave-in during drilling. The nominal diameter of the grouted zones of micropiles typically range from 100 mm to the order of 300 mm.

6.2.1.2.1 Axial Capacity

Based on the subsurface information encountered in Boreholes 21-04 and 21-05, a grouted micropile should have its bond zone formed within the underlying weathered dolostone. For preliminary design purposes, a factored grout-to-sand bond stress at ULS of 40 kPa and a factored grout-to-dolostone bond stress at ULS of 200 kPa be used. However, it should be noted that the depth to bedrock is currently unknown for the area of the pier.

This geotechnical analysis should be considered for preliminary design and planning purposes only since factors used in the final design may vary depending on the equipment and the installation methods utilized during construction. Micropiles are typically design/build elements of a structure and the final micropile design should be provided by a micropile specialty Contractor and should be compatible with the site conditions and his installation methods and equipment.

The actual capacity of the micropiles must be confirmed by on-site load tests. These tests should include a selected number of verification (performance) tests prior to production installation. A selected percentage of production micropiles should also be proof tested. Recommendations on the minimum scope of testing will be provided at a later date as more design details become available.

The design unconfined compressive strength of the cement grout should not be less than 30 MPa.

Consideration should be given to providing corrosion protection to all production micropiles.

The factored axial geotechnical resistance at ULS, PULS, of a single micropile may be calculated by the following expression:

$$P_{ULS} = \alpha \cdot A_s \cdot L$$

where α = factored ULS grout-to-sand or grout-to-dolostone bond stress, kPa
A_s = surface area per metre of bond length, m^2/m
L = bond length, m

Provided that the centre-to-centre spacing of the micropiles in a group is equal to or greater than three times the diameter of the grouted body, the capacity of a micropile group at this site may be calculated as the sum of capacities of all the individual micropile in the group.

It must be noted that the available subsurface information is insufficient to reliably evaluate the strength and deformation characteristics of the bedrock, including parameters such as unconfined compressive strength, rock quality designation (RQD), and fracture index. For the purposes of preliminary analysis, reference has been made to published data outlining typical ranges of unconfined compressive strength for limestone bedrock. It is recommended that boreholes be advanced at each abutment location (refer to Section 6.3) to obtain the necessary geotechnical information to support the design of the replacement bridge.

Table 6.5 provides preliminary geotechnical resistances for nominal diameter micropiles founded with a grouted bond zone straddling the sand and the weathered dolostone.

Table 6.5 – Preliminary Factored Axial Geotechnical Resistance at ULS for Micropiles in Compression

Micropile Bond Length within sand and weathered dolostone (m)	Micropile Diameter 100 mm (kN)	Micropile Diameter 150 mm (kN)	Micropile Diameter 200 mm (kN)	Micropile Diameter 300 mm (kN)
6(1)	225	325	450	675
7(2)	280	425	575	850

(1) Socket of approximately 3 m into bedrock.
(2) Socket of approximately 4 m into bedrock.

The above are nominal diameters for preliminary design purposes. Various suppliers and manufacturers of micropiles may provide products that have slightly different diameters from those shown above.

6.2.1.2.2 Lateral Capacity

The lateral resistance that can be provided by a micropile is relatively limited largely due to its flexibility. Consideration may be given to resisting a portion of the lateral loads by battering the micropiles. Battering of micropiles in the order of 1H: 3V is not uncommon although, micropiles may be battered over a range of inclinations. In addition, a steel casing installed within the upper portion of a micropile will increase its lateral resistance.

6.2.1.2.3 Micropile Installation

It is important to note that the geotechnical load capacity of a micropile is highly sensitive to the processes carried out during micropile installation including drilling techniques, drill cuttings flushing and grouting. At the north abutment, currently available design information indicates that installation of some micropiles requires coring through the existing unreinforced concrete footing. A steel casing should be installed through the cored zone to serve as a sleeve prior to grouting. A reputable proprietary supplier and installer should be contacted for detail information.

During construction, the Contractor shall observe the conditions vicinity of the micropile construction site and nearby structures on a daily basis for signs of ground heave or subsidence.

6.2.1.2.4 Micropile Verification and Proof Testing

As pointed out above, micropile load tests prior to and during construction are essential for verification of the assumed grout-to-soil and grout-to-rock bond stresses, the design of the pile system and the construction methods proposed prior to installing any production piles. The construction load testing should be considered an extension of the design. Based on current preliminary design requirements, one (1) sacrificial load test to failure should be carried out at a suitable location close to one of the abutments. A minimum of one (1) proof test should be carried out at each abutment.

All micropile testing and installation should be witnessed by qualified geotechnical personnel. The proprietary supplier and installer should be requested to submit the methodology of micropile installation, verification and proof testing setup and procedures for review and approval prior to installation and testing.

6.2.1.3 Temporary Excavation and Groundwater Control

The excavations for spread footing construction are expected to extend through granular fill and gravelly sand fill to reach the dolostone bedrock. Localized excavation of the bedrock may be required where the bedrock surface is encountered above the design underside of the footing elevation. An assessment of the strength of the bedrock should be carried out during the detailed design stage. The selection of the excavation equipment and the means and method of excavation is the responsibility of the Contractor.

All temporary excavations must be carried out in accordance with the current Occupational Health and Safety Act (OHSA) of Ontario and local regulations. In general, the soils are classified as Type 3 soils above the groundwater level, and Type 4 soils if excavation extends below the water level without prior dewatering.

It is anticipated that temporary cofferdams will be required to facilitate spread footing construction given the proximity of the bridge foundations to the Speed River. In addition to facilitating footing construction, the cofferdam will serve as temporary protection system at this site. The cofferdam/temporary protection system should be implemented in accordance with OPSS PROV 539 and designed for Performance Level 2. For preliminary design purposes, the cofferdam/temporary protection system may be designed using the soil parameters provided in the table below.

Table 6.6 – Soil Parameters for Temporary Protection System Design

Soil Parameter	Existing Dense Sand and Gravel Fill	Native Compact to Very Loose Sand
Φ (angle of internal friction)	32˚	30˚
γ (total unit weight)	21.5 kN/m^3	21 kN/m^3
γ_w (Submerged unit weight)	11.5 kN/m^3	11 kN/m^3
K_a	0.30	0.33
K_p	3.3	3.0

Full hydrostatic pressure should be considered assuming a water level at least equal to the design river level. The design and construction of temporary protection system is the responsibility of the

Contractor. The actual pressure distribution acting on the cofferdam/temporary protection system is a function of the construction sequence and the relative flexibility of the wall, and these factors have to be considered when designing the shoring system. All protection systems should be designed by a Professional Engineer experienced in such designs. The Contractor shall retain a Professional Engineer to carry out the design of the cofferdam/temporary protection system.

Unwatering from inside the cofferdam will be required to maintain a dry base during construction. Further comments on construction dewatering for the bridge foundations and an assessment of the need for a PTTW will be provided at a later time once further is design inputs are available.

Excavation and backfilling for the footings must be in accordance with OPSS.MUNI 902.

Care must be taken during the demolition and removal of the existing bridge and footings such that the founding subgrade would not be disturbed prior to constructing the new footings. Where sub-excavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using mass concrete of the same class as that of the footing.

6.2.1.4 Abutment Backfill and Lateral Earth Pressures

Backfill to the bridge abutments should consist of non-frost susceptible, free-draining granular material conforming to OPSS Granular A or Granular B Type II specifications. Compaction should be carried out in accordance with OPSS.MUNI 501. Small vibratory compaction equipment should be used within about 0.5 m of the abutments to minimize compaction induced stresses.

Earth pressures acting on the structure may be assumed to impose a triangular distribution governed by the characteristics of the backfill. For a fully drained condition, the lateral earth pressures on the abutment walls may be calculated using the following expression:

$$p_h = K(\gamma h + q)$$

Where:
p_h = horizontal pressure on the wall at depth h (kPa)
K = earth pressure coefficient (see table below)
γ = unit weight of retained soil (see table below)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

The earth pressure coefficients are dependent on the material used as backfill. Recommended unfactored values for horizontal ground surface behind the wall are shown in Table 6.7.

Table 6.7 – Lateral Earth Pressure Coefficients

Loading Condition	OPSS Granular A or Granular B Type II ϕ = 35°, γ = 22.8 kN/m^3	OPSS Granular B Type I or Type III ϕ = 32°, γ = 21.2 kN/m^3
	Horizontal Surface Behind Wall	**Horizontal Surface Behind Wall**
Active (Unrestrained Wall), K_a	0.27	0.31
At-rest (Restrained Wall) , K_o	0.43	0.47
Passive, K_p	3.7	3.3

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC.

In accordance with Clause 6.12.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

Design of the structures must incorporate measures such as weepholes to permit drainage of the backfill and avoid potential build-up of hydrostatic pressures behind the walls.

6.2.1.5 Seismic Considerations

In accordance with the CHBDC, the selection of the seismic site class is based on the average soil conditions encountered in the upper 30 m of the ground profile. The stratigraphy at this site generally consists of surficial asphalt overlying fill layers underlain by native deposits of silty sand till and clayey silt to silty clay till. These overburden materials are underlain by dolostone bedrock. The depth to bedrock at the structure site is in the order of 6 to 8 m below ground surface. As per Table 4.1 of the CHBDC, for a bridge supported on spread footing founded on rock, the site may be classified as Seismic Site Class B.

Based on the National Building Code of Canada (NBCC 2015), the peak horizontal ground acceleration (PGA), corresponding to a design earthquake having a 2 percent probability of being exceeded in 50 years (i.e. 2,475 year return period) is 0.084 g at the site.

Based on review of the SPT data, seismically-induced liquefaction of foundation soils is not anticipated under the design earthquake.

6.2.1.6 Corrosivity and Sulphate Attack Potential

The results of the corrosivity and sulphate analytical tests indicate the following conditions at the locations tested:

- The potential for sulphate attack on concrete foundations from the surrounding fill and native soils is considered to be negligible due to the low concentration of sulphate and chloride in the samples.
- The potential for soil corrosion on metal is considered to be mild to moderate based on the low resistivity values measured on the samples.
- Appropriate protection measures commensurate with the above are recommended if metal structural elements are used. The effects of road de-icing salts should be also considered.

6.2.2 Municipal Service Installation

In general, excavations for open cut installation of municipal services will extend through the existing pavement structure, fill materials and into the silty sand till and clayey silt to silty clay till. Use of a hydraulic excavator should be suitable for trench excavation within these the overburden soils. Provision should be made for handling and removal of pavement materials, possible obstructions in the fill, and cobbles or boulders in the till.

Localized excavation of the bedrock may be required where the bedrock surface is encountered above the pipe invert elevation. An assessment of the strength of the bedrock should be carried out during the detailed design stage. The selection of the excavation equipment and the means and method of excavation is the responsibility of the Contractor.

Where there is sufficient space available, sloped excavations may be used for municipal service installation. All temporary excavations must be carried out in accordance with the current Occupational Health and Safety Act (OHSA) of Ontario and local regulations. In general, the soils are classified as Type 3 soils above the groundwater level, and Type 4 soils if excavation extends below the water level without prior dewatering.

Where there is insufficient space to accommodate sloped excavations, installation of temporary protection will be required. The temporary protection system should be implemented in accordance with OPSS PROV 539 and designed for Performance Level 2. Depending on the depth of excavation, a trench box may be suitable. The protection system may encounter obstructions in the fill and till materials at this site including cobbles and boulders. The design and construction of temporary protection system is the responsibility of the Contractor. The Contractor's method of installation will need to be able to penetrate or dislodge any encountered obstructions. The temporary protection may be designed using the soil parameters previously provided in Table 6.5.

Excavations for municipal services are generally expected to remain above the groundwater table provided they are away from the Speed River. Dewatering of shallow excavations for excavations away from the river is expected to be feasible using sumps and pumps. Perched water may be encountered in permeable layers above the cohesive fill and till layers. All municipal service installations should be carried out in the dry.

Excavations for municipal services near the Speed River may extend below the water level. The water level is generally expected to be at approximately the same elevation as the river level (Elev. 315.6 m). Additional groundwater inflow into the excavations should be expected where the excavations for the municipal services extend close to the river and near or below the water level. The Contractor must be prepared to employ more elaborate dewatering procedures as necessary to complete the installations in the dry. An assessment of the requirements for construction dewatering will be provided at a later time once further is design inputs are available.

Prior to placement of pipe bedding, the base of the trench should be maintained in a dry condition, free of loose or disturbed material. The pipes must be placed on a uniformly competent subgrade. Pipe bedding materials, compaction and cover should follow OPSD 802.030 to 802.034, and/or City of Guelph specifications.

In areas where a less competent subgrade is encountered, it may be necessary to increase the sewer bedding thickness. Any excessively soft, loose or compressible materials at the pipe subgrade should be subexcavated and replaced with OPSS Granular A material compacted to at least 95 percent of SPMDD.

Trench backfill materials should be placed and compacted as per OPSS.MUNI 401 or City of Guelph specifications. The backfill should consist of OPSS Granular A or B material, or unshrinkable fill.

6.3 Detailed Geotechnical Investigation

The information presented in this report is provided for preliminary design and planning purposes only. Detailed geotechnical investigation will be required to confirm the subsurface conditions and recommendations. This work should incorporate:

- A detailed pavement investigation including additional boreholes within the existing roadway pavement to further define the existing granulars and subgrade conditions and confirm the pavement design recommendations;
- Boreholes within the envelope of all bridge foundation units to confirm the subsurface conditions at the structure location and develop detailed geotechnical recommendations for design and construction of the bridge foundations.
- Bedrock coring in boreholes at the proposed bridge foundations (abutments and piers) to confirm bedrock elevation and depth to competent bedrock. An assessment of the strength (including laboratory testing Point Load Test and Unconfined compressive strength tests) and quality of the bedrock for foundation design purposes.
- Chemical testing to confirm the requirements for reuse or disposal of excavated material in accordance with Ontario Regulations.

STATEMENT FOR USE AND INTERPRETATION OF REPORT

1. STANDARD OF CARE

This Report has been prepared in a manner consistent with that degree of care and skill ordinarily exercised by members of the same profession currently practicing under similar circumstances at the same time and in the same or similar locality and in compliance with all applicable laws.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment, including this Statement For Use and Interpretation of Report, are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT, AS DESCRIBED ABOVE. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TOTHE WHOLE OF THE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives, and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client for the development, design objectives, and/or purposes described to Thurber by the Client. **NO OTHER PARTY MAY USE OR RELY ON THE REPORT OR ANY PORTION THEREOF FOR OTHER THAN THE CLIENT'S BENEFIT IN CONNECTION WITH THE PURPOSES DESCRIBED IN THE REPORT**. Any use which a third party makes of the Report is the sole responsibility of such third party and is always subject to this Statement for Use and Interpretation of Report. Thurber accepts no liability or responsibility for damages suffered by any third party resulting from use of the Report for purposes outside the reasonable contemplation of Thurber at the time it was prepared or in any manner unintended by Thurber.

5. INTERPRETATION OF THE REPORT

a) **Nature and Exactness of Soil and Contaminant Description:** Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors is inherently judgement-based. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other parties making use of such documents or records with or without our express written consent need to be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other parties. Some conditions are subject to change over time and those making use of the Report need to be aware of this possibility and understand that the Report only presents the interpreted conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client must disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) **Reliance on Provided Information:** The evaluation and conclusions contained in the Report have been prepared based on conditions in evidence at the time of site inspections and based on information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report resulting from misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other parties providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) **Design Services:** The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber is recommended to be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design need to be reported to Thurber immediately so that Thurber can address potential conflicts.

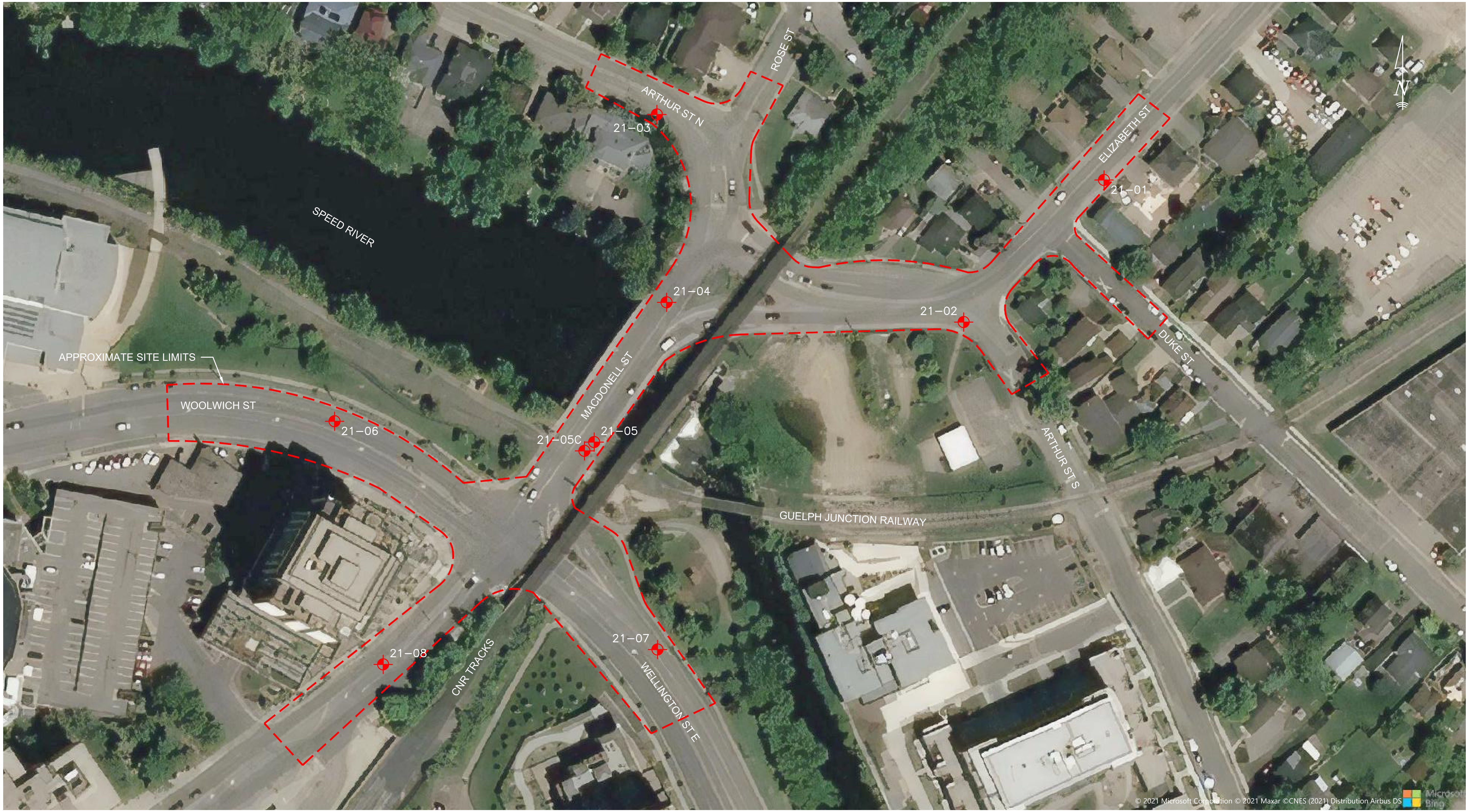
d) **Construction Services:** During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions to confirm and document that the site conditions do not materially differ from those conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or other parties who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes, but is not limited to, decisions made to develop, purchase, or sell land, unless such decisions expressly form part of the stated purpose of the Report as described in Paragraph 3.

Appendix A

Borehole Location Plan



LEGEND

⌖ BOREHOLE LOCATION

25 0 25 50m
SC-LE 1:1250

R.V. Anderson Associates Limited

GUELPH REVITALIZATION PROJECT
GUELPH, ONTARIO

BOREHOLE LOCATION PLAN

JOB# 30842

THURBER ENGINEERING LTD.

ENGINEER : JA	DRAWN : BH	APPROVED : GRL
DATE : May 2023	SCALE : 1:1250	DRAWING No. 30842-1

FILENAME: H:\Drafting\30000\30842\TED-30842-BHPL.dwg
PLOTDATE: Apr 11, 2023 - 8:57 AM

Appendix B

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) ‘N’ VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT “N” VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS Split Spoon Sample
WS Wash Sample
AS Auger (Grab) Sample
TW Thin Wall Shelby Tube Sample
TP Thin Wall Piston Sample
PH Sampler Advanced by Hydraulic Pressure
PM Sampler Advanced by Manual Pressure
WH Sampler Advanced by Self Static Weight
RC Rock Core
SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

▼ Water Level
C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT ‘N’ Value Standard Penetration Test ‘N’ Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to “A” size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

▨	CLAYSTONE
▤	SILTSTONE
░	SANDSTONE
■	COAL
▩	Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

RECORD OF BOREHOLE 21-01

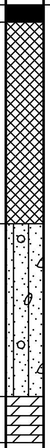
PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 21, 2021
COMPLETED : July 21, 2021

N 4 821 950.7 E 561 264.9

Project No. 30842
SHEET 1 OF 1
DATUM Geodetic

BORING METHOD: Hollow Stem Augers

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	318.23
ASPHALT(113mm)	0.11
SAND and **GRAVEL**, dense brown, moist: (FILL)	
SAND, silty, gravelly, trace clay, occasional lime stone fragments, dense to dense, brown, moist: (TILL)	316.78 1.45
DOLOSTONE, highly weathered	315.64 2.59
END OF BOREHOLE AT 2.9m UPON AUGER REFUSAL.	315.33 2.90

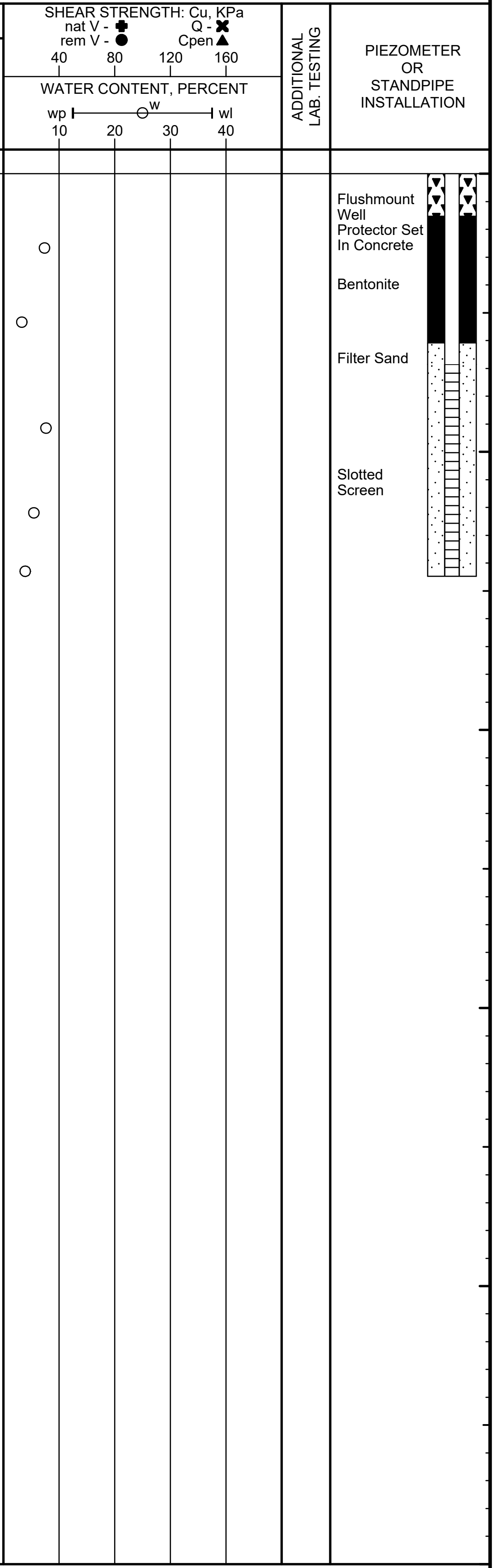


NUMBER	TYPE	BLOWS/0.3m	COMMENTS
1	SS	38	
2	SS	33	
3	SS	33	Grain Size Analysis: Gr 25%/Sa 48%/ Si & Cl 27%
4	SS	110/ 0.225	
5	SS	60/ 0.075	

Monitoring Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEV.(m)
Aug 11/21	Dry	-
Aug 18/21	Dry	-



DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

DEPTH SCALE (metres): 1 2 3 4 5 6 7 8 9

GROUNDWATER ELEVATIONS

∇ WATER LEVEL UPON COMPLETION ▼ WATER LEVEL IN WELL/PIEZOMETER

LOGGED : SM
CHECKED : JA

THURBER

RECORD OF BOREHOLE 21-02

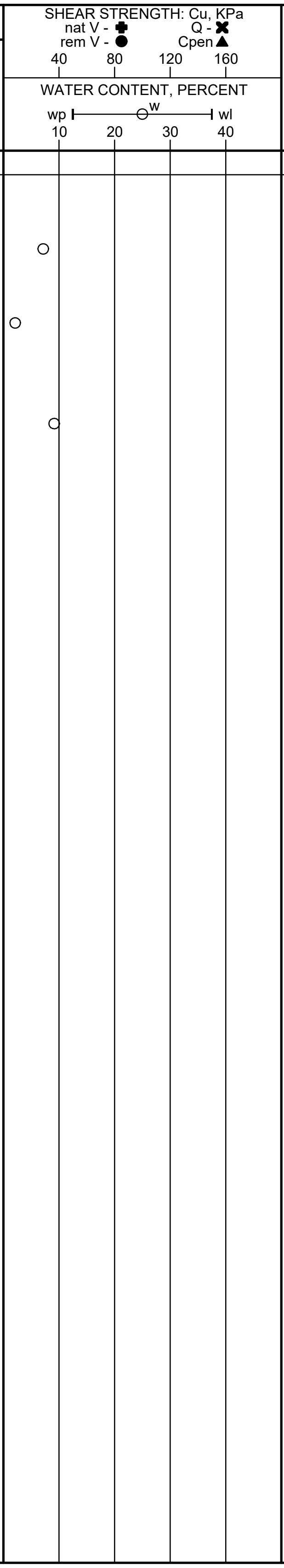
PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 21, 2021
COMPLETED : July 21, 2021

N 4 821 902.7 E 561 217.6

Project No. 30842
SHEET 1 OF 1
DATUM Geodetic

BORING METHOD: Hollow Stem Augers

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	COMMENTS
GROUND SURFACE	318.22				
ASPHALT(100mm)	0.10				
SAND and **GRAVEL**, trace silt, dense to compact, brown, moist: (FILL)		1	SS	48	Grain Size Analysis: Gr 38%/Sa 54%/ Si & Cl 8%
		2	SS	29	
	316.77 1.45				
SAND, silty, gravelly, compact, brown: (TILL)		3	SS	17	
	316.23 1.98				
DOLOSTONE, highly weathered	2.06				
END OF BOREHOLE AT 2.1m UPON AUGER REFUSAL.					



DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

GROUNDWATER ELEVATIONS

▽ WATER LEVEL UPON COMPLETION   ▼ WATER LEVEL IN WELL/PIEZOMETER

LOGGED : SM
CHECKED : JA

RECORD OF BOREHOLE 21-03

PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 30, 2021
COMPLETED : July 30, 2021

N 4 821 972.9 E 561 114.0

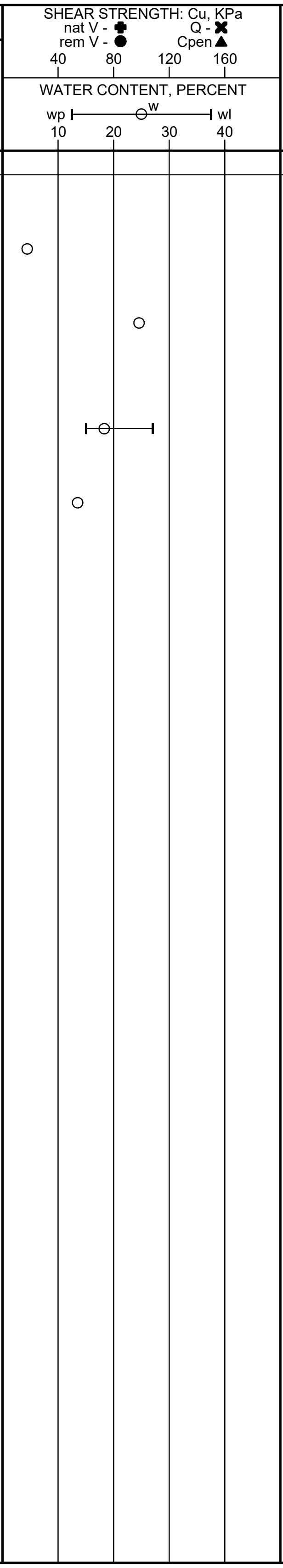
Project No. 30842
SHEET 1 OF 1
DATUM Geodetic

BORING METHOD: Hollow Stem Augers

DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	COMMENTS
GROUND SURFACE	318.07				
ASPHALT(75mm)					
SAND and **GRAVEL**, trace silt, dense, brown, moist: (FILL)	0.10				
		1	SS	22	
CLAY, silty, sandy, trace gravel, soft, brown: (FILL)	317.15 0.91	2	SS	2	
		3	SS	3	Grain Size Analysis: Gr 1%/ Sa 23%/ Si 55%/ Cl 21%
	315.65	4	SS	60/ 0.125	
END OF BOREHOLE AT 2.4m UPON AUGER REFUSAL.	2.41				

DEPTH SCALE (metres): 1, 2, 3, 4, 5, 6, 7, 8, 9

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

GROUNDWATER ELEVATIONS

▽ WATER LEVEL UPON COMPLETION
▼ WATER LEVEL IN WELL/PIEZOMETER

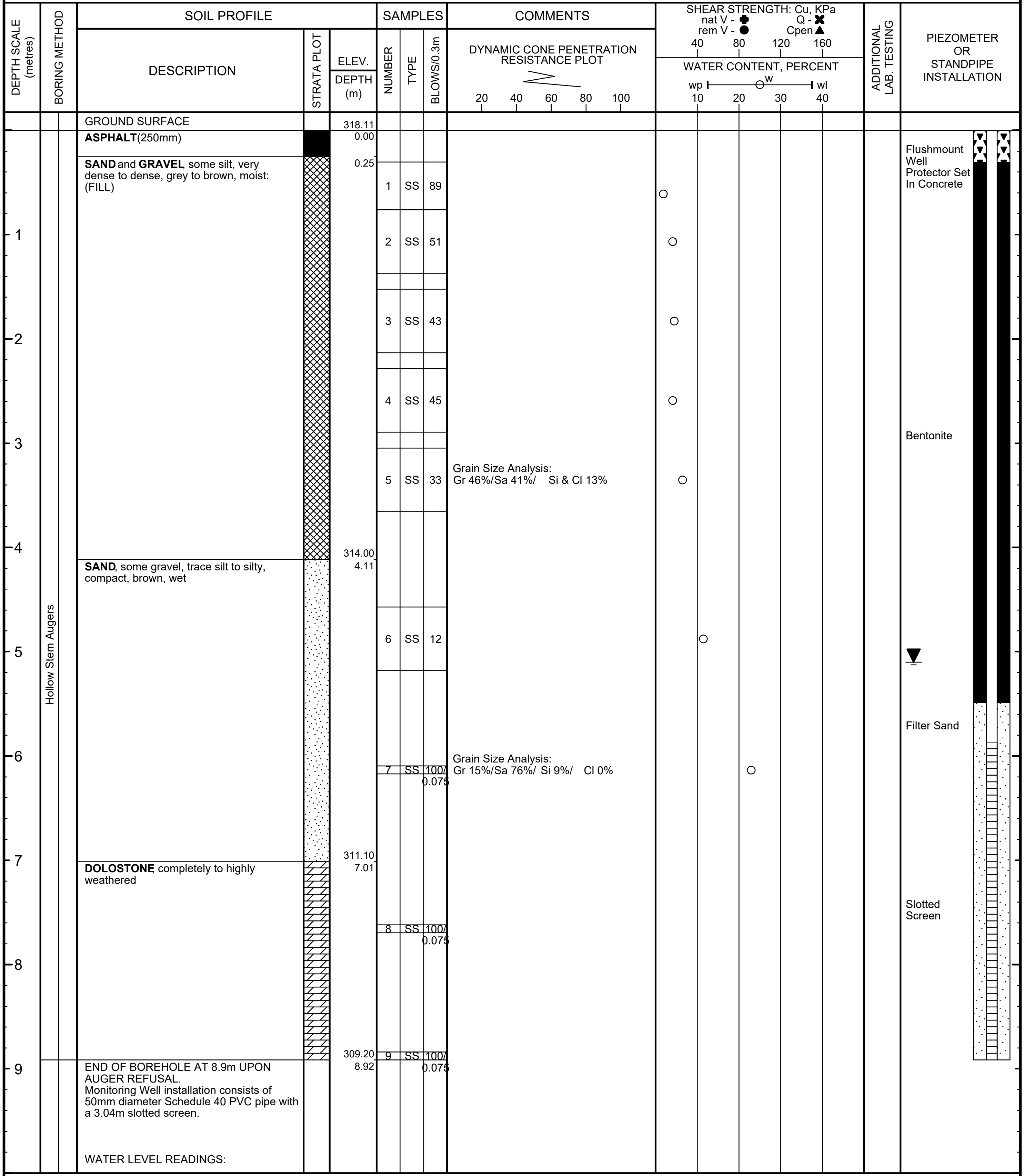
LOGGED : SM
CHECKED : JA

RECORD OF BOREHOLE 21-04

PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 20, 2021
COMPLETED : July 20, 2021

N 4 821 909.4 E 561 117.3

Project No. 30842
SHEET 1 OF 2
DATUM Geodetic



DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	318.11
ASPHALT(250mm)	0.00
SAND and **GRAVEL**, some silt, very dense to dense, grey to brown, moist: (FILL)	0.25
SAND, some gravel, trace silt to silty, compact, brown, wet	314.00 4.11
DOLOSTONE, completely to highly weathered	311.10 7.01
END OF BOREHOLE AT 8.9m UPON AUGER REFUSAL. Monitoring Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 3.04m slotted screen. WATER LEVEL READINGS:	309.20 8.92

NUMBER	TYPE	BLOWS/0.3m	COMMENTS
1	SS	89	
2	SS	51	
3	SS	43	
4	SS	45	
5	SS	33	Grain Size Analysis: Gr 46%/Sa 41%/ Si & Cl 13%
6	SS	12	
7	SS	100/ 0.075	Grain Size Analysis: Gr 15%/Sa 76%/ Si 9%/ Cl 0%
8	SS	100/ 0.075	
9	SS	100/ 0.075	

GROUNDWATER ELEVATIONS

▽ WATER LEVEL UPON COMPLETION

▼ WATER LEVEL IN WELL/PIEZOMETER
August 18, 2021

LOGGED : SM
CHECKED : JA

THURBER

THURBER2S TEL-30842.GPJ 12/6/21

RECORD OF BOREHOLE 21-04

PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 20, 2021
COMPLETED : July 20, 2021

N 4 821 909.4 E 561 117.3

Project No. 30842
SHEET 2 OF 2
DATUM Geodetic

DEPTH SCALE (metres)	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	COMMENTS DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	SHEAR STRENGTH: Cu, KPa nat V - ✚ rem V - ● Q - ✖ Cpen ▲ 40 80 120 160 WATER CONTENT, PERCENT wp ⊢ w ○ ⊣ wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
11											
12											
13											
14											
15											
16											
17											
18											
19											

DATE	DEPTH(m)	ELEV.(m)
Aug 11/21	4.31	313.80
Aug 18/21	5.10	313.01

GROUNDWATER ELEVATIONS

▽ WATER LEVEL UPON COMPLETION

▼ WATER LEVEL IN WELL/PIEZOMETER
August 18, 2021

LOGGED : SM
CHECKED : JA

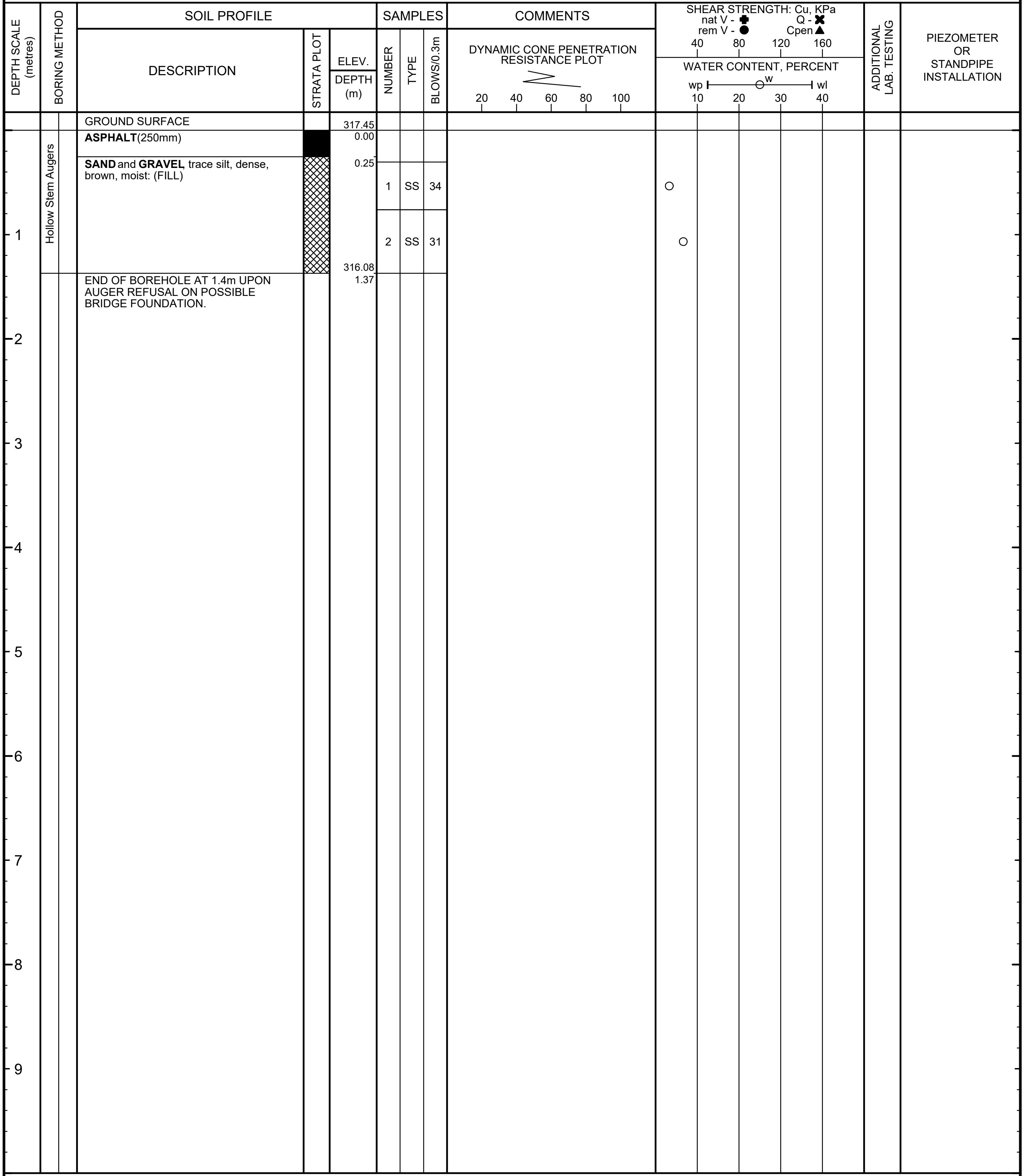
THURBER

RECORD OF BOREHOLE 21-05

PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 20, 2021
COMPLETED : July 20, 2021

N 4 821 862.2 E 561 092.8

Project No. 30842
SHEET 1 OF 1
DATUM Geodetic



BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m
	GROUND SURFACE		317.45			
Hollow Stem Augers	**ASPHALT**(250mm)	■	0.00			
Hollow Stem Augers	**SAND** and **GRAVEL**, trace silt, dense, brown, moist: (FILL)	▨	0.25	1	SS	34
				2	SS	31
	END OF BOREHOLE AT 1.4m UPON AUGER REFUSAL ON POSSIBLE BRIDGE FOUNDATION.		316.08 1.37			

GROUNDWATER ELEVATIONS

▽ WATER LEVEL UPON COMPLETION ▼ WATER LEVEL IN WELL/PIEZOMETER

LOGGED : SM
CHECKED : JA

RECORD OF BOREHOLE 21-05C

PROJECT : Guelph Revitalization Project

LOCATION : Guelph, ON

STARTED : July 30, 2021

COMPLETED : July 30, 2021

N 4 821 862.2 E 561 092.7

Project No. 30842

SHEET 1 OF 1

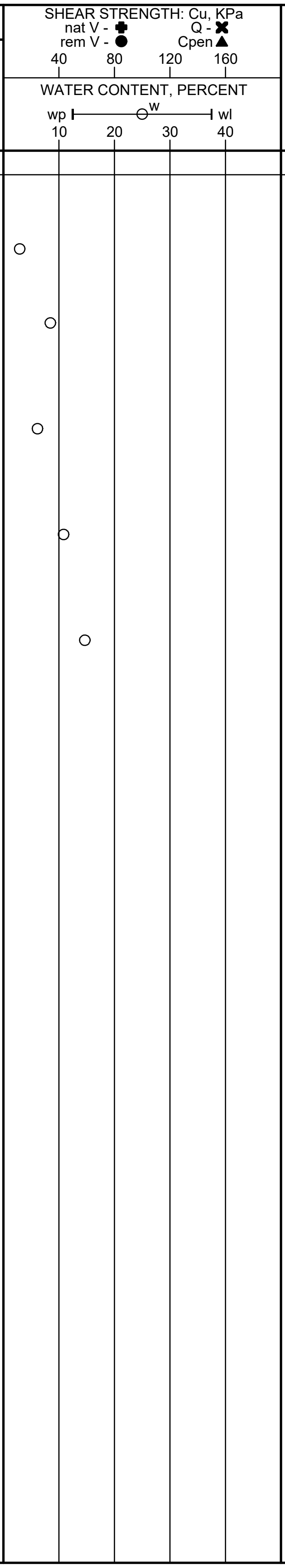
DATUM Geodetic

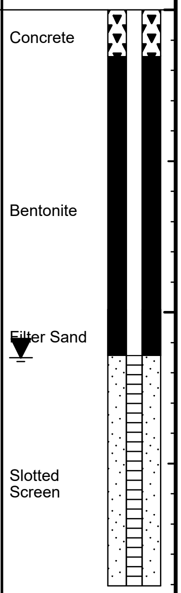
BORING METHOD: Hollow Stem Augers

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	COMMENTS
GROUND SURFACE	317.48				
ASPHALT(100mm)					
SAND and **GRAVEL**, trace silt, dense to compact, brown, moist: (FILL)	0.10	1	SS	36	
		2	SS	20	
SAND, gravelly, silty, trace clay, compact to very loose, brown, moist: (FILL)	316.03 1.45	3	SS	13	
		4	SS	5	Grain Size Analysis: Gr 32%/Sa 44%/ Si 23%/ Cl 1%
		5	SS	2	
END OF BOREHOLE AT 3.8 M UPON AUGER REFUSAL. Monitoring Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 3.04m slotted screen. Dolostone fragments	313.67 3.81				

WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEV.(m)
Aug 11/21	2.30	315.18
Aug 18/21	2.30	315.18





ADDITIONAL LAB. TESTING

GROUNDWATER ELEVATIONS

▽ WATER LEVEL UPON COMPLETION

▼ WATER LEVEL IN WELL/PIEZOMETER August 18, 2021

LOGGED : SM

CHECKED : JA

THURBER

RECORD OF BOREHOLE 21-06

PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 21, 2021
COMPLETED : July 21, 2021

N 4 821 869.2 E 561 005.2

Project No. 30842
SHEET 1 OF 1
DATUM Geodetic

BORING METHOD: Hollow Stem Augers

DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	COMMENTS
GROUND SURFACE	318.93				
ASPHALT(150mm)	0.00				
SAND and **GRAVEL**, trace silt, dense to compact, brown, damp: (FILL)	0.15	1	SS	45	
trace peat, occasional brick fragments, black		2	SS	26	
SAND and **SILT**, some clay, trace gravel to gravelly, loose to very dense, brown, moist: (FILL)	317.48 1.45	3	SS	5	Grain Size Analysis: Gr 2%/ Sa 43%/ Si 44%/ Cl 11%
	316.44 2.49	4	SS	86/ 0.200	

END OF BOREHOLE AT 2.5m UPON AUGER REFUSAL.
Monitoring Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

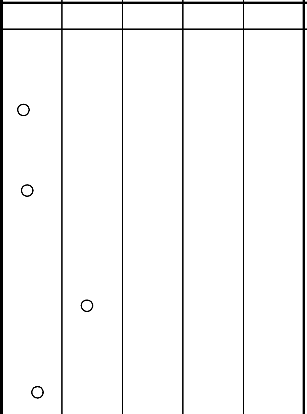
DATE	DEPTH(m)	ELEV.(m)
Aug 11/21	Dry	-
Aug 18/21	Dry	-

DEPTH SCALE (metres): 1, 2, 3, 4, 5, 6, 7, 8, 9

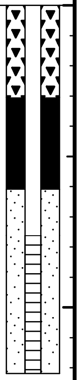
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: Cu, KPa — nat V - ✚ Q - ✖ rem V - ● Cpen ▲ — 40 80 120 160

ADDITIONAL LAB. TESTING



PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER ELEVATIONS

∇ WATER LEVEL UPON COMPLETION ▼ WATER LEVEL IN WELL/PIEZOMETER

LOGGED : SM
CHECKED : JA

THURBER

RECORD OF BOREHOLE 21-07

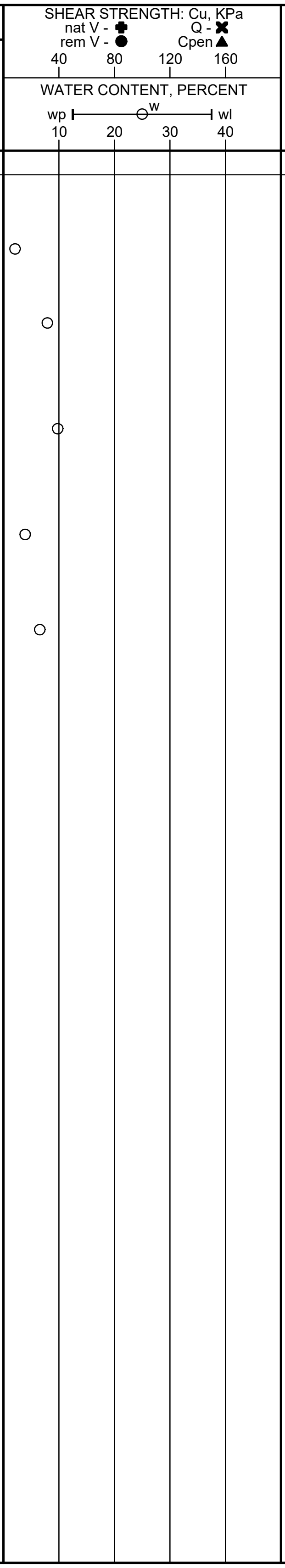
PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 21, 2021
COMPLETED : July 21, 2021

N 4 821 792.2 E 561 114.2

Project No. 30842
SHEET 1 OF 1
DATUM Geodetic

BORING METHOD: Hollow Stem Augers

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	COMMENTS
GROUND SURFACE	317.79				
ASPHALT(125mm)	0.13				
SAND and **SILT**, gravelly to some gravel, trace clay, very dense to compact, brown, moist: (FILL)		1	SS	54	
		2	SS	28	Grain Size Analysis: Gr 11%/Sa 39%/ Si 42%/ Cl 8%
SAND and **GRAVEL**, trace silt, loose, grey, moist: (FILL)	316.34 1.45	3	SS	4	
Cobbles		4	SS	9	
		5	SS	20	
END OF BOREHOLE AT 3.5m UPON AUGER REFUSAL.	314.29 3.51				



DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE (metres): 1 2 3 4 5 6 7 8 9

GROUNDWATER ELEVATIONS

▽ WATER LEVEL UPON COMPLETION
▼ WATER LEVEL IN WELL/PIEZOMETER

LOGGED : SM
CHECKED : JA

THURBER

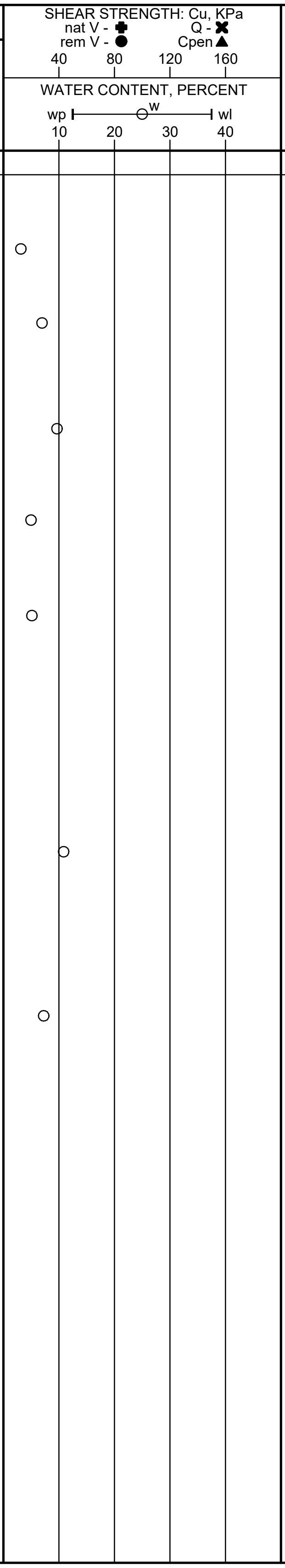
RECORD OF BOREHOLE 21-08

PROJECT : Guelph Revitalization Project
LOCATION : Guelph, ON
STARTED : July 30, 2021
COMPLETED : July 30, 2021

N 4 821 787.1 E 561 021.6

Project No. 30842
SHEET 1 OF 1
DATUM Geodetic

BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	COMMENTS (DYNAMIC CONE PENETRATION RESISTANCE PLOT)
Hollow Stem Augers	GROUND SURFACE	321.40				
	ASPHALT(125mm)					
	SAND and **GRAVEL**, trace silt, very dense, greyish brown, moist: (FILL)	0.13	1	SS	56	
	Clayey **SILT**, some sand to sandy, trace gravel, compact, brown: (TILL)	320.64 0.76	2	SS	22	
			3	SS	22	Grain Size Analysis: Gr 1%/ Sa 13%/ Si 68%/ Cl 18%
	SAND, silty, gravelly, trace clay, very dense, brown, moist: (TILL)	319.12 2.29	4	SS	101/ 0.250	
			5	SS	60/ 0.100	
	CLAY, silty, some sand, hard, grey, wet: (TILL)	317.44 3.96	6	SS	42	Grain Size Analysis: Gr 0%/ Sa 19%/ Si 48%/ Cl 33%
			7	SS	73	
	END OF BOREHOLE AT 6.3m UPON AUGER REFUSAL.	315.08 6.32				



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

GROUNDWATER ELEVATIONS

∇ WATER LEVEL UPON COMPLETION
▼ WATER LEVEL IN WELL/PIEZOMETER

LOGGED : SM
CHECKED : JA

THURBER

Appendix C

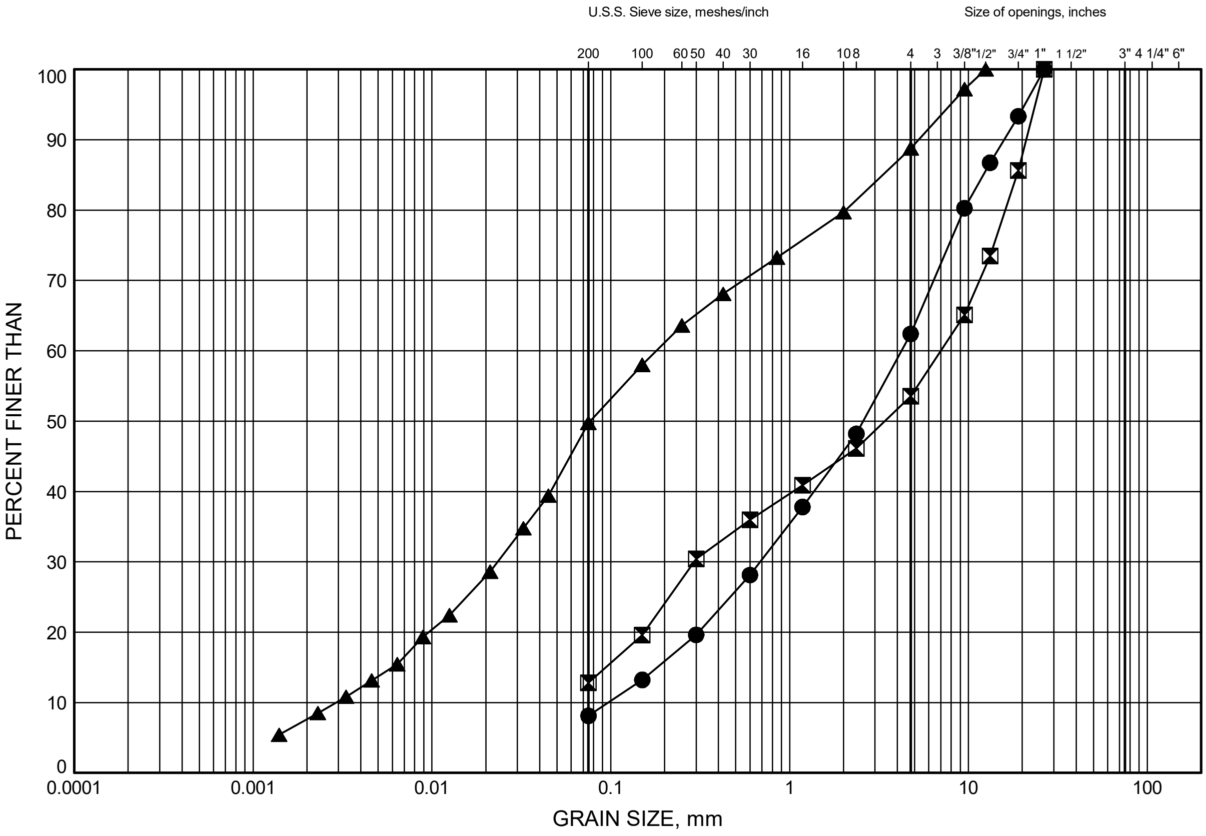
Geotechnical Laboratory Test Results

Guelph Revitalization Project

GRAIN SIZE DISTRIBUTION

FIGURE C1

Granular FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	21-02	0.53	317.68
☒	21-04	3.35	314.76
▲	21-07	1.07	316.72

GRAIN SIZE DISTRIBUTION - THURBER TEL-30842.GPJ 9/30/21

Date September 2021
Project 30842

THURBER

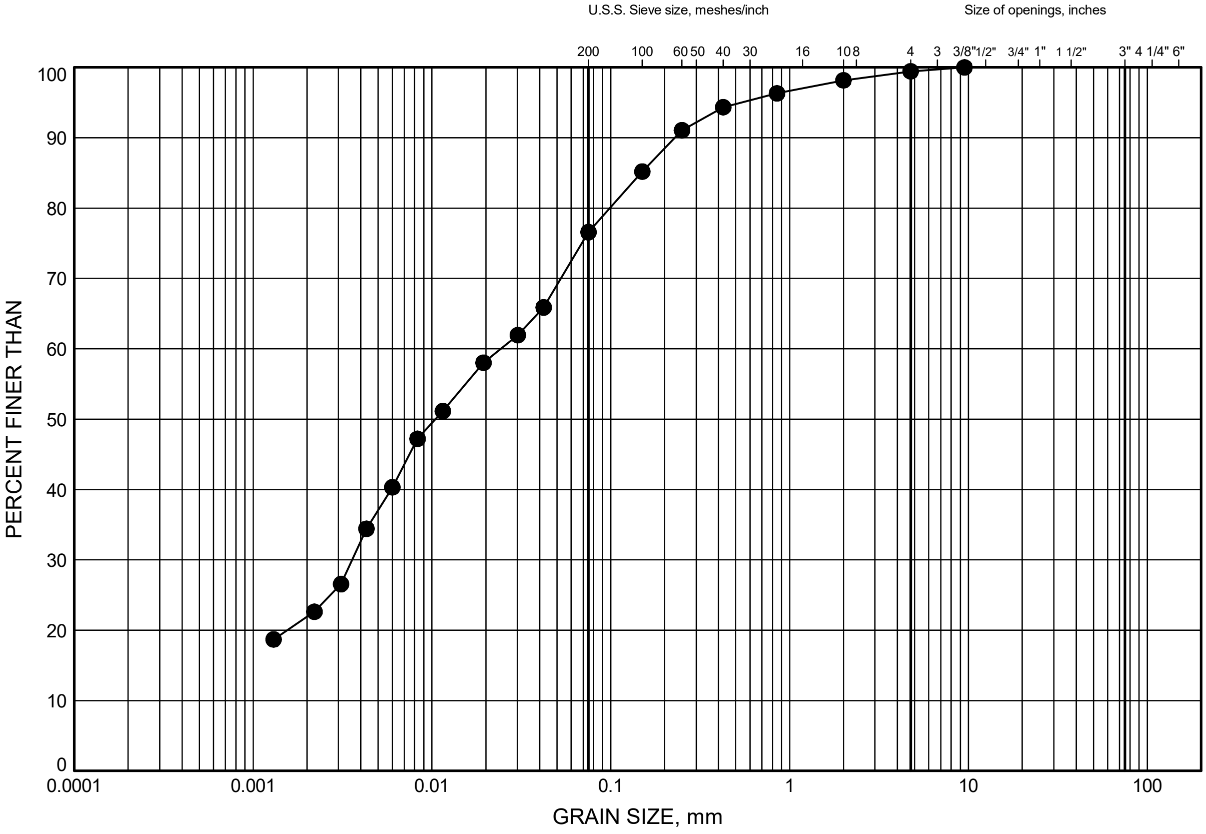
Prep'd MFA
Chkd. GL

Guelph Revitalization Project

GRAIN SIZE DISTRIBUTION

FIGURE C2

Silty CLAY FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	21-03	1.83	316.24

Date September 2021

Project 30842

THURBER

Prep'd MFA

Chkd. GL

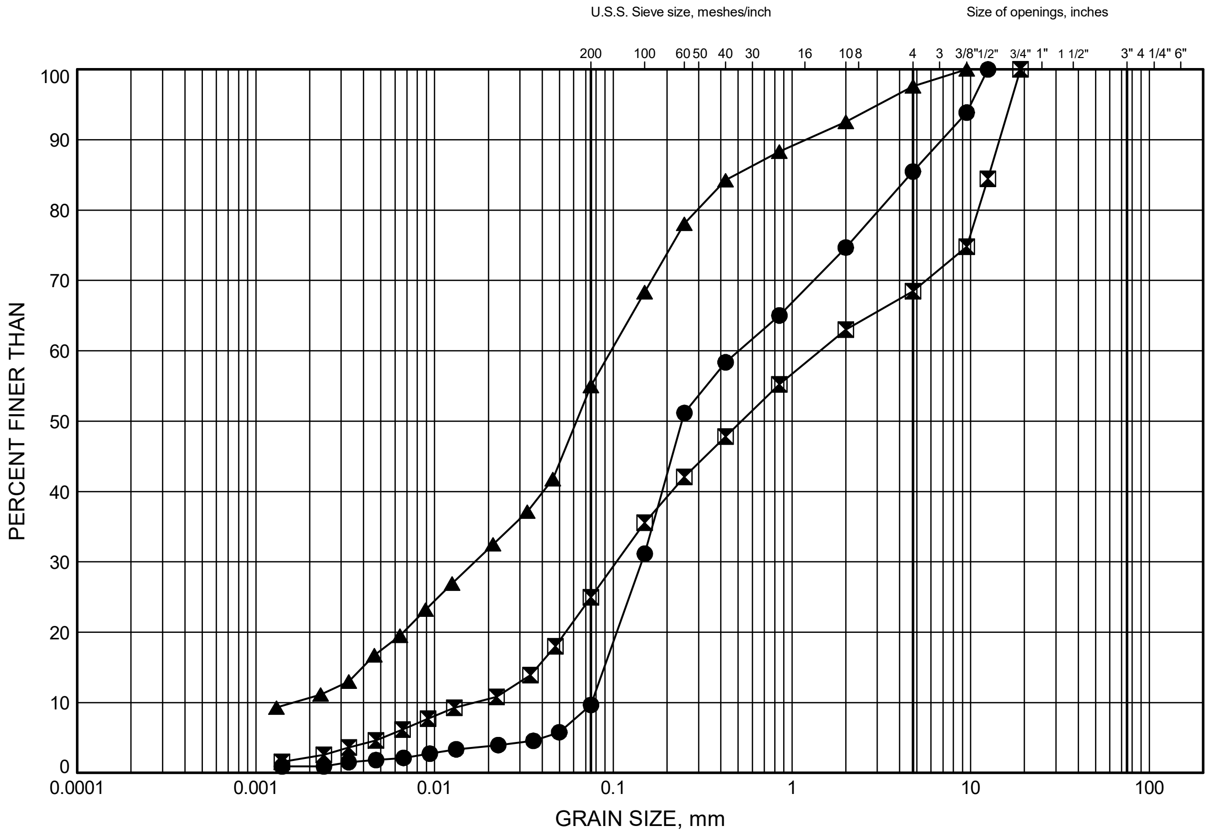
GRAIN SIZE DISTRIBUTION - THURBER TEL-30842.GPJ 9/30/21

Guelph Revitalization Project

GRAIN SIZE DISTRIBUTION

FIGURE C3

Gravelly SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	21-04	6.14	311.98
☒	21-05C	2.59	314.89
▲	21-06	1.83	317.10

Date September 2021

Project 30842

THURBER

Prep'd MFA

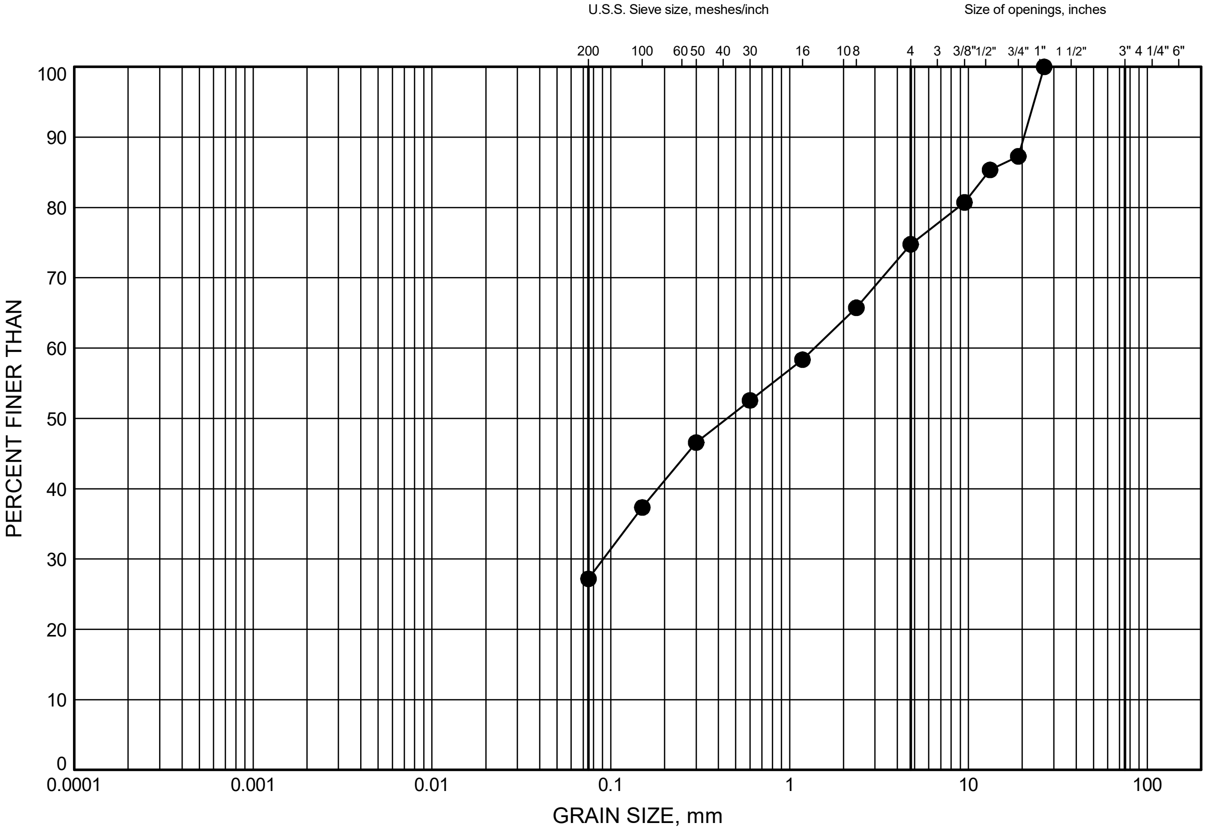
Chkd. GL

Guelph Revitalization Project

GRAIN SIZE DISTRIBUTION

FIGURE C4

Silty SAND TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	21-01	1.83	316.40

Date September 2021

Project 30842

THURBER

Prep'd MFA

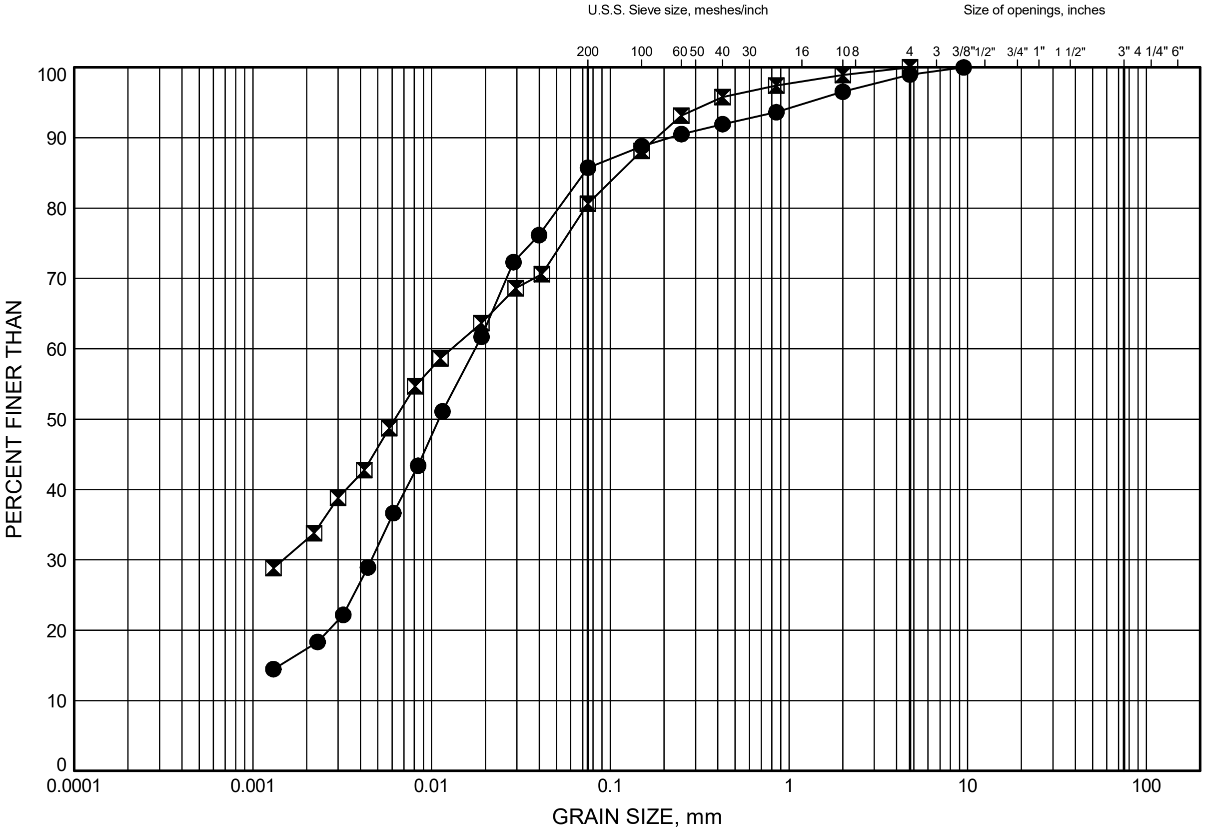
Chkd. GL

Guelph Revitalization Project

GRAIN SIZE DISTRIBUTION

FIGURE C5

Clayey SILT to Silty CLAY TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	21-08	1.83	319.57
☒	21-08	4.88	316.52

Date September 2021

Project 30842

THURBER

Prep'd MFA

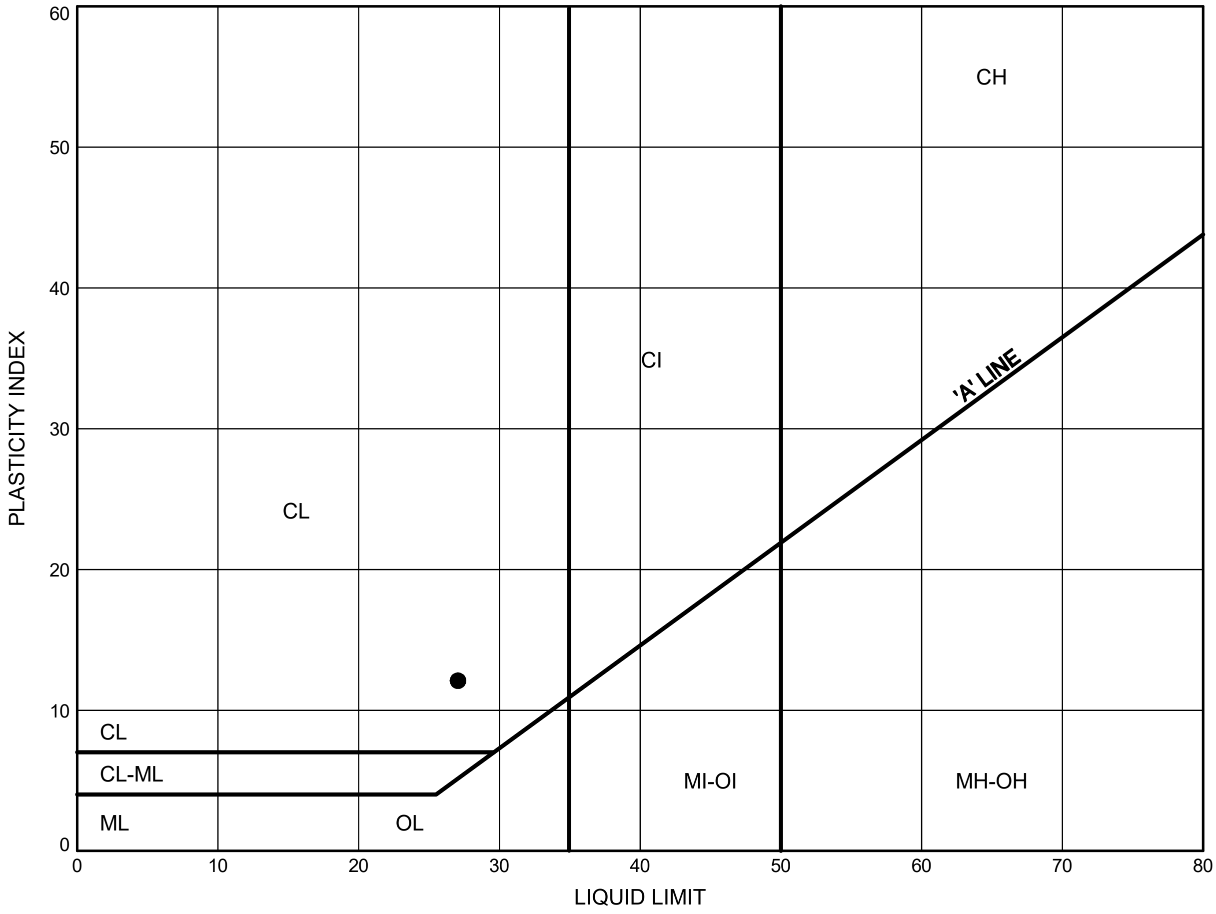
Chkd. GL

Guelph Revitalization Project

ATTERBERG LIMITS TEST RESULTS

FIGURE C6

Silty CLAY FILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	21-03	1.83	316.24

THURBALT TEL-30842.GPJ 9/30/21

Date September 2021

Project 30842

THURBER

Prep'd MFA

Chkd. GL

Appendix D

Analytical Laboratory Test Results

FINAL REPORT

CA14884-AUG21 R1

30842, Guelph Revitilization Project

Prepared for

Thurber Engineering Ltd.

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Thurber Engineering Ltd.	Project Specialist	Brad Moore Hon. B.Sc
		Laboratory	SGS Canada Inc.
Address	103, 2010 Winston Park Drive Oakville, ON L6H 5R7. Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Joshua Alexander	Telephone	705-652-2143
Telephone	613-606-7303	Facsimile	705-652-6365
Facsimile		Email	brad.moore@sgs.com
Email	jalexander@thurber.ca	SGS Reference	CA14884-AUG21
Project	30842, Guelph Revitilization Project	Received	08/10/2021
Order Number		Approved	08/16/2021
Samples	Soil (2)	Report Number	CA14884-AUG21 R1
		Date Reported	08/16/2021

COMMENTS

Temperature of Sample upon Receipt: 7 degrees C
Cooling Agent Present:Yes
Custody Seal Present:Yes

Chain of Custody Number:007521

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Brad Moore Hon. B.Sc

TABLE OF CONTENTS

PACKAGE: **- Corrosivity Index** (SOIL)

			Sample Number	5	6
			Sample Name	BH21-05C, SS3	BH21-04, SS4
			Sample Matrix	Soil	Soil
			Sample Date	30/07/2021	30/07/2021
Parameter	**Units**	**RL**		**Result**	**Result**
Corrosivity Index					
Corrosivity Index	none	1		14	13
Soil Redox Potential	mV	-		202	266
Sulphide (Na2CO3)	%	0.04		< 0.04	< 0.04
pH	pH Units	0.05		9.39	9.08
Resistivity (calculated)	ohms.cm	-9999		863	242

PACKAGE: **- General Chemistry** (SOIL)

			Sample Number	5	6
			Sample Name	BH21-05C, SS3	BH21-04, SS4
			Sample Matrix	Soil	Soil
			Sample Date	30/07/2021	30/07/2021
Parameter	**Units**	**RL**		**Result**	**Result**
General Chemistry					
Conductivity	uS/cm	2		1160	4130

PACKAGE: **- Metals and Inorganics** (SOIL)

			Sample Number	5	6
			Sample Name	BH21-05C, SS3	BH21-04, SS4
			Sample Matrix	Soil	Soil
			Sample Date	30/07/2021	30/07/2021
Parameter	**Units**	**RL**		**Result**	**Result**
Metals and Inorganics					
Moisture Content	%	0.1		8.4	4.0
Sulphate	µg/g	0.4		27	62

FINAL REPORT

Client: Thurber Engineering Ltd.
Project: 30842, Guelph Revitilization Project
Project Manager: Joshua Alexander
Samplers: N/A

PACKAGE: **- Other (ORP)** (SOIL)

			Sample Number	5	6
			Sample Name	BH21-05C, SS3	BH21-04, SS4
			Sample Matrix	Soil	Soil
			Sample Date	30/07/2021	30/07/2021
Parameter	**Units**	**RL**		**Result**	**Result**
Other (ORP)					
Chloride	µg/g	0.4		630	2600

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0180-AUG21	µg/g	0.4	<0.4	9	35	97	80	120	95	75	125
Sulphate	DIO0180-AUG21	µg/g	0.4	<0.4	5	35	98	80	120	93	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide (Na2CO3)	ECS0022-AUG21	%	0.04	< 0.04	10	20	105	80	120			

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0176-AUG21	uS/cm	2	< 2	2	20	98	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0176-AUG21	pH Units	0.05	NA	0		101			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS	Insufficient sample for analysis.
RL	Reporting Limit.
↑	Reporting limit raised.
↓	Reporting limit lowered.
NA	The sample was not analysed for this analyte
ND	Non Detect

Samples analysed as received. Solid samples expressed on a dry weight basis. “Temperature Upon Receipt” is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated. This document is issued, on the Client's behalf, by the Company under its General Conditions of Service available on request and accessible at http://www.sgs.com/terms_and_conditions.htm. The Client's attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein. Any other holder of this document is advised that information contained hereon reflects the Company's findings at the time of its intervention only and within the limits of Client's instructions, if any. The Company's sole responsibility is to its Client and this document does not exonerate parties to a transaction from exercising all their rights and obligations under the transaction documents.

This report must not be reproduced, except in full. This report supersedes all previous versions.

-- End of Analytical Report --

SGS Environment, Health & Safety

Request for Laboratory Services and CHAIN OF CUSTODY

- Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Fax: 705-652-6365 Web: www.sgs.com/environment
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361

No: 007521

Page 1 of 1

Laboratory Information Section - Lab use only

Received By: Olay Mozhi
Received Date (mm/dd/yy): 8/10/21 (mm\dd\yy)
Received Time: 10:00

Received By (signature):
Custody Seal Present: ☑
Custody Seal Intact: ☐ ✓

Cooling Agent Present: ☑ ice
Temperature Upon Receipt (°C) 7,7,7

LAB LIMS #: 0A14883-84-Aug21

REPORT INFORMATION

Company: Thurber Engineering Ltd.
Contact: Joshua Alexander
Address: 2010 Winston Park Dr.
Unit 103, Oakville, ON, L6H5R7
Phone: 6136067303
Email: jalexander@thurber.ca
Email:

INVOICE INFORMATION

☐ (same as Report Information)
Company:
Contact:
Address:

Phone:
Email: accountingON@thurber.ca

PROJECT INFORMATION

Quotation #:
P.O. #:
Project #: 30842
Site Location/ID: Guelph Revitilization Project

TURNAROUND TIME (TAT) REQUIRED

☒ Regular TAT (5-7days)

TAT's are quoted in business days (exclude statutory holidays & weekends).
Samples received after 6pm or on weekends: TAT begins next business day

RUSH TAT (Additional Charges May Apply): ☐ 1 Day ☐ 2 Days ☐ 3 Days ☐ 4 Days

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date:
Rush Confirmation ID:

NOTE: DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY

REGULATIONS

Regulation 153/04:
☐ Table 1
☐ Table 2
☐ Table 3
☐ Table ____
☐ R/P/I
☐ I/C/C
☐ A/O

Soil Texture:
☐ Coarse
☐ Medium
☐ Fine

Other Regulations:
☐ Reg 347/558 (3 Day min TAT)
☐ PWQO
☐ CCME
☐ MISA
☐ MMER
☐ Other:

Sewer By-Law:
☐ Sanitary
☐ Storm
Municipality:

RECORD OF SITE CONDITION (RSC) ☐ YES ☐ NO

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX	Field Filtered (Y/N)	Metals & Inorganics	PAH☐ ABN☐ SVOC(all)☐	PCB Total☐ Aroclor☐	PHC F1-F4☐ VOC☐ BTEX☐ BTEX/F1☐ F2-F4☐	VOC☐ BTEX☐ THM☐	Pesticides OC☐ OP☐	TCLP M&I☐ VOC☐ PCB☐ B(a)P☐ ABN☐ Ignit.☐	Water Pkg Gen.☐ Ext.☐	Sewer Use:	TCLP	Soil Corrosivity Pkg				COMMENTS:
1	TCLP	07/30/21		1	Soil	N										X					
2	BH21-05C, SS3	07/30/21		1	Soil	N											X				
3	BH21-04, SS4	07/20/21		1	Soil	N											X				
4																					
5																					
6																					
7																					
8																					
9																					
10																					
11																					
12																					

Observations/Comments/Special Instructions * Please make sure electric conductivity + SAR are apart of soil corrosivity.

Sampled By (NAME):
Signature:
Date: __/__/__ (mm/dd/yy)
Pink Copy - Client

Relinquished by (NAME): Joshua Alexander
Signature:
Date: 08/10/21 (mm/dd/yy)
Yellow & White Copy - SGS

Revision #: 1.1
Date of Issue: 04 April, 2018

Appendix E

National Building Code of Canada Seismic Hazard Values

2015 National Building Code Seismic Hazard Calculation

INFORMATION: **Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 43.547N 80.244W **User File Reference:** Macdonell Street Bridge 2021-09-29 16:04 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.117	0.066	0.038	0.010
Sa (0.1)	0.152	0.089	0.054	0.016
Sa (0.2)	0.136	0.082	0.052	0.017
Sa (0.3)	0.108	0.067	0.044	0.015
Sa (0.5)	0.082	0.052	0.035	0.012
Sa (1.0)	0.047	0.030	0.020	0.006
Sa (2.0)	0.024	0.015	0.009	0.002
Sa (5.0)	0.006	0.004	0.002	0.001
Sa (10.0)	0.002	0.002	0.001	0.000
PGA (g)	0.084	0.049	0.030	0.009
PGV (m/s)	0.066	0.040	0.025	0.007

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Appendix F

Site Photographs

Photograph 1 – Macdonell Bridge, looking East from North of West abutment

Photograph 2 – Macdonell Bridge, looking West from South of East Abutment

Appendix F

Pavement Design Analysis

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Thurber Engineering Ltd.

Flexible Structural Design Module

Guelph Revitalization - Pavement Design

20-Year Design

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	3,096,501
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	30,000 kPa
Stage Construction	1
Calculated Design Structural Number	117 mm

Simple ESAL Calculation

Performance Period (years)	20
Two-Way Traffic (ADT)	19,251
Number of Lanes in Design Direction	1
Percent of All Trucks in Design Lane	50 %
Percent Trucks in Design Direction	100 %
Percent Heavy Trucks (of ADT) FHWA Class 5 or Greater	2 %
Average Initial Truck Factor (ESALs/truck)	2
Annual Truck Factor Growth Rate	1 %
Annual Truck Volume Growth Rate	0 %
Growth	Compound
Total Calculated Cumulative ESALs	3,096,501

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	New HMA	0.42	1	140	3.6	59
2	Granular Base	0.14	1	150	3.6	21
3	Granular Subbase	0.09	1	475	3.6	43
Total	-	-	-	765	-	123