# FOUNDATION INVESTIGATION AND DESIGN REPORT TOWNLINE ROAD BRIDGE (SN 16-511) – TOWNSHIP OF ELIZABETHTOWN-KITLEY & TOWNSHIP OF RIDEAU LAKES, ONTARIO.

Project No.: CCO-23-0440-02

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September 2022

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# 1.0 INTRODUCTION

This report presents the factual data obtained from the geotechnical investigation conducted on Kitley South Elmsley Townline Road for the proposed renewal of Townline Road Bridge (SN16-511). The site investigation consisted of drilling two (2) exploratory subsurface boreholes to depths ranging to approximately ≈ 6.0 m and 7.5 m below ground surface (m bgs.) on June 01st, 2022.

The purpose of the investigation was to obtain information on the subsurface conditions at the location of Townline Road Bridge (The Bridge) and present a report on the subsurface soil stratigraphy, borehole records, and laboratory test results. The report will also include the anticipated geotechnical engineering conditions influencing the design and construction of the proposed bridge renewal, and recommendations for foundation design.

# 2.0 SITE DESCRIPTION

## 2.1 Existing Site Condition

The Townline Rd. Bridge is located approximately 2.0 km east from HWY 29 and Kitley South Elmsley Townline Road, on Kitley South Elmsley Townline Road crossing The Hutton Creek. The bridge built in 1952 is a rigid frame with vertical legs supports is orientated in an east-west direction. The overall structure width is 8.6 m, with a total deck length of 7.0 m, and 6.0 m roadway width on the deck, as indicated in OSIM inspection dated 2021-05-28.

The Kitley South Elmsley Townline Road is a local two-lane roadway with a maximum speed of 60 km/h. The road is a gravel surfaced road with gravel shoulders. The Townline Rd. Bridge is surrounded with a residential structure to the north, and farmlands and a patch of wooded areas to its south. The site location is shown in Figure 1, Appendix B.

## 2.2 Site Geology

A desktop study using the published physiography maps of the area on Ontario Geological Survey [1] [2] website indicates the site is located within the Limestone Plains. Surficial geology maps of the area indicate to Paleozoic bedrock. The Bedrock in the area is composed of dolostone, minor shale and sandstone deposits of the Beekmantown Group of the Oxford formation.

# 3.0 FIELD PROCEDURES

The staff of McIntosh Perry Consulting Engineers (McIntosh Perry) conducted a site visit prior to the planned drill-date and marked the proposed borehole locations; additionally, requisitions were submitted to obtain public utility clearance locates from Ontario One Call (ON1Call). Prior to drilling, all necessary permits were retained from respective authorities. The intended geotechnical exploration drill-date was communicated with the bridge owner through TSI Inc. regarding access and traffic control measures.

Prior to commencement of drilling, temporary condition traffic control measures were established around the work-zone in accordance with Temporary Conditions, Ontario traffic manual - Book 7 [3], to maintain unimpeded traffic flow for the duration of the drilling operation.

Two (2) boreholes were drilled using a truck-mounted boring drill rig: a 150 mm hollow stem helical auger drilling machine. The drill was advanced incrementally below the ground surface, while intermittent soil samples were taken at 0.75 m intervals. Each soil sample was retrieved with a 51 mm outside diameter (OD) Standard Penetration Test (SPT) split spoon (SS) sampler in accordance with ASTM D 1586, SPT test procedures.

Rock coring was commenced on auger refusal in boreholes BH22-1 and 2 at a depth of ≈ 2.6 m and ≈ 3.0 m bgs respectively.

Boreholes BH22-1 and 2 were cored to a depth of ≈ 6.0 m and ≈ 7.5 m bgs. respectively. A diamond-bit rock cutter, size NQ core barrel was advanced with the assistance of water-cooling method to retrieve rock core samples. The bedrock coring continued until satisfactory quality bedrock core samples were retrieved from the underlain rock.

As the samples were retrieved from the SPT sampler, they were examined, logged, hermetically sealed in plastic bags, labeled, and packaged for transportation. As each rock core samples were retrieved, the quality and the description of the core was determined, measured, logged, labeled, and packaged into rock core sample boxes in accordance with ASTM D 4220-95, “Preserving and Transporting Soil Samples”. The packages of soil and rock core samples were forwarded to McIntosh Perry Geotechnical laboratory in Ottawa for further examinations and laboratory testing.

At the end of the borehole drilling operation, the open hole was backfilled with auger-cuttings and Bentonite hole plugs and restored to its original surface condition. A summary of borehole designations, location and approximate depths are shown in Table 3-1 and the borehole locations are shown on Figure 2, Appendix B.

Table 3-1. Borehole Summary

| Borehole ID | Drilled Date | Coordinates (Geodetic) | | | Borehole | |
|---|---|---|---|---|---|---|
| | | Latitude | Longitude | Surface El. (m) | Depth (m) | Elevation El. (m) |
| BH22-1 | June 1, 2022 | N44° 50' 48.877331" | W75° 59' 35.831483" | 109.3 | 6.0 | 103.3 |
| BH22-2 | June 1, 2022 | N44° 50' 48.559996" | W75° 59' 36.599363" | 109.2 | 7.5 | 101.7 |

# 4.0 LABORATORY TEST PROCEDURES

All soil samples received at the laboratory were logged, and soil descriptions were verified by additional tactile examination. Four (4) representative soil samples from specific soil layers and depths were identified and forwarded to McIntosh Perry Geotechnical laboratory (MP Geotech lab) for detailed soil analysis. The specific soil included samples from fill, and native soil layers.

Six (6) rock core samples were tested for Unconfined Compressive strength (UCS) of intact rock cores at the McIntosh Perry Geotechnical laboratory.

The laboratory analysis included determination of natural moisture content, grain-size distribution (sieve and hydrometer) analysis, and UCS test for intact rock cores. All laboratory tests to determine the index properties were performed in accordance with the Ministry of Transportation Ontario (MTO), which adopts the American Society for Testing Materials (ASTM) test procedures.

The relevant testing procedures are listed below;

- ASTM D2216 – Laboratory Determination of Water Content of Soil and Rock by Mass
- ASTM C136 – Sieve Analysis of Fine and Coarse Aggregates (LS-602)
- LS-702 – Determination of Particle Size Analysis of Soils
- ASTM D7928 – Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis
- ASTM C117 –Materials Finer than 75 µm (No. 200) Sieve in Mineral Aggregates by Washing (LS-601)
- ASTM D7012 – Unconfined Compressive Strength of Intact Rock Cores

All samples retained from the investigation are stored at MP Geotech lab for 90 days after the final report is submitted, thereafter the soil samples are disposed of according to MP Geotech lab policies. Unless The City notifies the geotechnical laboratory in writing.

# 5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

## 5.1 Subsurface Conditions

The Site subsurface stratigraphy consists of several layers of soil, comprising of granular fill material, silty sand and gravel, and silty clay. The main soils components were divided into three (3) distinct strata and identified according to the Unified Soil Classification System (USCS) as:

1. Gravel Surface (Granular)
2. Silty Sandy Gravel
3. Bedrock

The borehole log shows a cross-section view of the subsurface soil stratigraphy of the location. The Borehole log are shown in Appendix C.

### *5.1.1 Gravel Surface (Granular)*

The thickness of the gravel road surface was ≈ 200 mm, the sample gathered from this layer was obtained by the auger grab sampling method. A representative sample from this layer was forwarded for sieve analysis. The grain-size distribution percent by weight is shown in Table 5-1.

The grain-size distribution curve for the granular material was compared to a USCS granular specifications [4]. The distribution curve approximately complied to Granular B Type I, Quarry Specification envelope of USCS specification. The grain-size curve shows slight deviation from the specification envelope. It is noteworthy that foundation drilling methods' capability for sampling granular material is limited. Bulk sampling during construction may reveal more accurate indication of the present fill material. The grain-size distribution curve for the granular material is shown in Figure 3, Appendix D

Table 5-1. Grain-Size Distribution Summary.

| Borehole | Sample | Constituent Materials in percent weight | | | |
|---|---|---|---|---|---|
| | | Gravel (%) | Sand (%) | Fines | |
| | | | | Silt (%) | Clay (%) |
| BH22-1 | GS-1 | 32 | 48 | 20 | |

### 5.1.2 Silty Sandy Gravel

The cohesionless Fill layer comprising of Silty Sandy Gravel was encountered below the ≈ 200 mm hard (gravel) surface layer in boreholes BH22-1 and 2, the Fill layer was observed from a depth of ≈ 200 mm to depth ranging to ≈ 2.3 – 2.6 m bgs. The soil properties of this layer were light to dark brown, with some grey and dry to moist.

The SPT N-Index value ranged from ≈ 4 – 23 blows/0.3 m indicating to an approximate compactness of loose to dense, with ≈ 11 – 15% of natural moisture content in the tested samples.

The blow counts for BH22-1 SS-5 and BH22-2 SS-4 indicates to very high SPT N-index values, the high blow counts encountered at these depths indicated to possible boulders and dense sandy gravel Till layer, no soil samples were present in split spoon sample BH22-1 SS-5 due to rock fragment stuck in the tip of the sampler spoon.

Three (3) representative samples from the fill layer were subjected to hydrometer analysis testing, the fill layer grain-size distribution summary is shown in Table 5-2. The grain-size distribution curve is shown in Figure 4, Appendix D.

Table 5-2. Grain Size Distribution Summary

| Borehole | Sample | Constituent Materials in percent weight | | | |
|---|---|---|---|---|---|
| | | Gravel (%) | Sand (%) | Fines | |
| | | | | Silt (%) | Clay (%) |
| BH22-1 | SS-3 | 39 | 38 | 20 | 3 |
| BH22-1 | SS-4 | 38 | 21 | 23 | 18 |
| BH22-2 | SS-4 | 15 | 38 | 38 | 9 |

### 5.1.3 Bedrock

Bedrock was encountered in both boreholes at a depth of ≈ 2.9 – 3.0 m bgs. The properties of the bedrock encountered in both boreholes indicated to dolostone with thinly to thickly horizontally laminated and bedded sandstone to minor shale deposits, light to dark grey. The structural integrity of the rock mass indicated to some horizontal fractures, and joint discontinuities, with occasional mechanical breaks caused by the rock coring activity. The rock core sample description is shown in Table 5-3, and images of the rock core are shown in Figures 5 and 6, Appendix D.

Table 5-3. Rock Core Sample Summary.

| RC Sample # | RC Run # | Depth (m) | Theoretical length of RC (m) | Length of RC recovered (m) | ∑ of length recovered pieces > 100 mm (m) | Total Core Recovery, TCR (%) | RQD (%) |
|---|---|---|---|---|---|---|---|
| BH22-1 | | | | | | | |
| 6 | 1 | 3.0 – 4.5 | 1.5 | 1.5 | 1.4 | 98 | 95 |
| 7 | 2 | 4.5 – 6.0 | 1.5 | 1.4 | 1.4 | 95 | 92 |
| BH22-2 | | | | | | | |
| 6 | 1 | 2.9 – 4.5 | 1.5 | 1.2 | 1.0 | 78 | 63 |
| 7 | 2 | 4.5 - 5.1 | 0.6 | 0.6 | 0.5 | 96 | 83 |
| 8 | 3 | 5.1 – 5.9 | 0.8 | 0.8 | 0.8 | 91 | 86 |
| 9 | 4 | 5.9 – 7.5 | 1.5 | 1.5 | 1.4 | 97 | 92 |

A total of six rock core samples from both boreholes were tested for UCS of Intact Rock Cores in accordance with ASTM D7012 Method C to determine the strength of underlain bedrock. The UCS of bedrock ranged from ≈ 106 – 227 MPa. The rock core strength analysis summary is shown in Table 5-4.

Table 5-4. Rock Core Strength Summary.

| Core # | RC Sample # | RUN # | Depth (m) | Diameter (mm) | Thickness/ Height (mm) | Density (kg/m$^3$) | Compressive Strength (MPa) | Mass of Core (g) |
|---|---|---|---|---|---|---|---|---|
| 1 | BH22-1 RC-6 | 1 | 3.0 - 4.5 | 47 | 106 | 2770 | 208 | 508 |
| 2 | BH22-1 RC-7 | 2 | 4.5 - 6.0 | 47 | 107 | 2764 | 218 | 518 |
| 3 | BH22-2 RC-6 | 2 | 2.9 - 404 | 47 | 109 | 2760 | 139 | 525 |
| 4 | BH22-2 RC-7 | 3 | 4.4 - 5.1 | 47 | 108 | 2751 | 227 | 518 |
| 5 | BH22-2 RC-8 | 4 | 5.1 - 5.9 | 47 | 109 | 2713 | 106 | 519 |
| 6 | BH22-2 RC-9 | 5 | 5.9 - 7.5 | 47 | 111 | 2749 | 187 | 534 |

The bedrock UCS of intact rock core ‘Certificate of Analysis’ and the images of the tested core samples are shown in Appendix D.

## 5.2 Chemical Analysis

Soil and water samples from the borehole BH22-2 SS-3 from the fill layer and water sample collected from the upstream of the bridge was sent for chemical analysis testing for the following; pH level, resistivity level, chloride, and sulphate concentration. The corresponding test results indicates to the following levels of concentration shown in Table 5-5. The laboratory test results, "Certificate of Analysis" is shown in Appendix D.

Table 5-5. Chemical Analysis Summary.

| Project | SAMPLE | Depth (m) | Chemical Analysis | | | |
|---|---|---|---|---|---|---|
| | | | pH (pH units) | Resistivity (Ohm.cm) | Chloride (ppm) | sulphate (ppm) |
| CCO-23-0440 | BH22-2 SS-3 | 1.6 | 7.2 | 29.6 | 88 | 29 |
| CCO-23-0440 | Water | Surface | 7.9 | 2520 | 11.3 | 1.0 |

## 5.3 Groundwater

Groundwater was not observed within the overburden soil layer, and due to the residual water accumulation from wash boring groundwater level was not measured in open boreholes. However, the groundwater is expected to be at the creek level. The groundwater level is expected to fluctuate seasonally.

# 6.0 DISCUSSIONS AND RECOMMENDATIONS

## 6.1 General

It is unknown to the foundation geotechnical team at this point the existing bridge is considered for rehabilitation or replacement. The Townline Rd. Bridge is located approximately 2.0 km east from HWY 29 and Kitley South Elmsley Townline Road, on Kitley South Elmsley Townline Road crossing The Hutton Creek. The bridge built in 1952 is a rigid frame with vertical legs supports in an east-west orientation. The overall structure width is 8.6 m, with a total deck length of 7.0 m, and 6.0 m roadway width on the deck. This section of the report provides recommendations for the renewal design of this bridge. The existing rigid frame is expected to be currently bearing on shallow strip footings.

The recommendations included in this report are based on the interpretation of factual information obtained from the boreholes advanced during this subsurface investigation. The discussions and recommendations presented are intended to provide sufficient information to the designer to assess and determine the best rehabilitation or replacement alternative for the structure.

The comments made on the construction of the culvert are intended to highlight those aspects which could impact or affect the detailed design of the replacement bridge, for which special provisions may be required in

the Contract Documents. Comments related to construction aspects are not intended to dictate construction equipment or methods. Relevant parties should make their own interpretation of the factual data presented in the report. Interpretation of the data presented may affect equipment selection, proposed construction methods, and scheduling of construction activities.

## 6.2 Ground Characterization

### *6.2.1 Overview of Subsurface Conditions*

In summary, borehole investigation indicated presence of 2.3 to 2.6 m of overburden, and 0.7 of native till over bedrock. Bedrock was encountered at 2.7 m to 3 m below existing surface. The bedrock depths correspond to El. 106.5 to 106.3 referenced elevation. Rock elevations were relatively consistent across the bridge profile. Convincingly the rock surface can be assumed to be relatively flat and consistent across the site.

The bedrock is expected be extremely weathered at the surface. Below the weathered surface bedrock, the measured RQD of 95% and 63% were noted in boreholes BH22-1 and 2 respectively.

Rock mechanical properties measured in borehole BH22-2 will likely govern the design, at RQD = 63% and UCS = 139 MPa.

Laboratory test results and rock core photos are included in Appendix D.

## 6.3 Existing Bridge Condition

As noted in the OSIM reports and appeared during site visits shows there are a major crack visible on both abutments (east and west) extended into footing and culvert's top slab (see Appendix F). The same crack has propagated from the abutment into the bridge deck. There are no as-built drawings available. However, it can be resolved that the existing footings are resting on the bedrock.

The observed cracks appear as caused by differential settlement of the structure. As noted in investigation notes and reporting in the borehole logs, the rock at the west of the creek is extremely weathered at the surface. It is concluded that founding on extremely weathered bedrock has caused permanent damages to the structure. In the absence of as built drawings it cannot be confirmed, If existing footings are bearing on native soil or on the bedrock.

Since it is foundation issue, crack repairs on the superstructure will not remediate the root cause of the damage. In case of disintegration at foundation level, the structure will be subject to further movement and cracks will appear once more after repair have been completed. The cause of movement can be attributed to seasonal changes, frost action, or due to imposition of considerable unbalanced service loads.

In our engineering opinion, restoration of the bridge is not a viable option, replacement of the bridge in its entirety should be considered.

## 6.4 Geotechnical Resistance

Geotechnical resistance is calculated for the rock surface. Most likely, construction requires installation of cofferdams and complete dewatering before proceeding with demolition, removal of the existing structure, and excavation for the proposed footings. The contractor shall be responsible to provide stamped shop drawings for the coffer dam design and construction.

Upon excavation and surface preparation, all extremely weathered rock shall be removed from the surface. Once the loose rock pieces are removed, the rock surface shall be stabilized by high-slump lean concrete of minimum 15 MPa strength to prevent further weathering and loss of integrity at the rock surface.

Once the above noted condition is provided, the following Ultimate Limit State (ULS) capacity can be used for the design. The serviceability limit state corresponding to allowable settlements is not applicable for the case of founding on rock. For the design on bedrock, only the ULS values noted in this report will govern the design. The SLS geotechnical capacity, as a limiting factor for settlement, does not apply since the loads which will cause such settlement will be significantly larger than the calculated ULS.

Ultimate Limit State (ULS) = 1550 kPa

For lateral stability of the footings on bedrock, a friction coefficient of 0.45 can be used for concrete sliding on wet and slightly weathered bedrock.

The contractor shall document the subgrade review performed and approved by a licenced geotechnical engineer.

This bearing capacity on rock surface is applicable to isolated footings, strip footings, and box culverts.

## 6.5 Option Analysis

All design options will require dewatering through installation of coffer dams. The subsurface information shown in the two drilled boreholes are relatively consistent and can be used for the design of the coffer dams by the contractors.

Due to the uncertainties associated with the quality of the rock surface, the preferred option is the cast-in-place box culvert. In this option the surface shall be prepared as explained in Section 6.4, once the rebars are placed, the culvert and the grade adjustment concrete shall be poured monolithically.

Open footing culvert is also an option. This option can be considered following recommendation offered in Section 6.4.

A single or modular box culvert would be another viable option, for this project the surface shall be prepared as outlined in Section 6.4. to receive the pre-cast box culvert. The ends of the culvert shall be designed with culvert end treatment to prevent the possibility of uplift, undermining, or damage due to ice or debris [5]. Cut-off wall shall be provided at each end of the culvert to stabilize the structure and prevent the wash-out of the fill and bedding material. The cut-off wall shall be of sufficient depth and strength to prevent undermining for the life of the structure and be integral with or securely attached to the culvert. Suitable apron shall be provided to control erosion caused by the discharge from the ends of the culvert.

Arched culvert, concrete, polymer, and CSP pipe culvert options can also be considered with recommendations offered in Section 6.4.

### *6.5.1 Recommendations for Culvert Installation*

The replacement recommendations provided here are based on the observations and measurements obtained during the geotechnical site investigation, and the following recommendations are specified for the design;

- Subgrade shall be kept dry at all times, and construction of the culvert shall not be carried out under water. Upon dewatering, excavation shall extend to the rock surface. All loose rock pieces shall be removed from site and the subgrade shall be approved by a geotechnical professional.
- OPSS Granular A following OPSS.MUNI 401 requirements, shall be used for bedding, cover, and embedment materials.
- The minimum 300 mm granular A bedding is required for all options. This is to provide uniform bedding for all noted prefabricated options and to reduce the adverse effect of rock surface undulations. The 300 mm minimum bedding is required to reduce the risk of concentrated stresses on the structure.
- A minimum of 300 mm of bedding material shall be provided, the bedding material shall be compacted and shaped to receive the bottom of the culvert, and a minimum of 300 mm cover shall be provided above the culvert.
- Bedding and cover materials shall be placed in uniform layers not exceeding 200 mm in thickness loose, and each layer shall be compacted according to OPSS 501 before subsequent layer is placed.
- The culvert bed shall be compacted and shaped to receive the bottom of the pipe or arch culverts.

- Bedding, cover, and embedment materials shall be placed on each side of the pipe and shall be completed simultaneously. Materials must be placed evenly on both ends and shall not differ in layer thickness greater than 200 mm on either end of uncompacted layers.
- Bedding and embedment of culvert should be Granular A conforming to OPSS. MUNI 1010 must be placed in loose uniform lifts no greater than 200 mm in thickness and compacted to a minimum of 100% SPMDD. If grade-raise is required, Granular B conforming to OPSS. MUNI 1010 shall be placed in loose uniform lifts no greater than 300 mm in thickness and compacted to a minimum of 98% SPMDD.
- It is necessary to provide clay seal close to both ends of the culvert to avoid the flow through the backfill, clay seal shall conform to OPSS.MUNI 1205, and clay seal shall be placed according to OPSD 802.095.

## 6.6 Lateral Earth Pressure – Seismic Loading

Seismic (earthquake) loading should be considered in the design in accordance with Clauses 4.6.5 and C4.6.5 of S6-19, for the selected PGA. Lateral capacity of abutments and wingwalls shall be checked against the current code requirements.

Table 6-2: Lateral Pressure parameters for Granular A and B and Horizontal Backfill

| Pressure Parameter | Expected Value | |
|---|---|---|
| | OPSS Granular B | Existing Sand Fill |
| Unit Weight ($\gamma$) kN/m$^3$ | 21 | 19 |
| Angle of Internal Friction ($\phi$) | 32 | 27 |
| Coefficient of Active Earth Pressure ($k_a$) | 0.31 | 0.38 |
| Coefficient of Passive Earth Pressure ($k_p$) | 3.25 | 2.66 |
| Coefficient of Earth Pressure at Rest ($k_o$) | 0.47 | 0.55 |
| Seismic Coefficient of Active Earth Pressure ($K_{AE}$) - Yielding | 0.41 | 0.49 |
| Seismic Coefficient of Active Earth Pressure ($K_{PE}$) | 2.94 | 2.38 |

The calculation for hydrostatic pressure, analysis to account for the sloped backfill behind a retaining structure, or dynamic analysis shall be conducted per CHBDC recommendations. Seismic coefficients were calculated for a PGA of 0.305 for 2% chance of exceedance in 50 years.

If the proposed replacement is rigid frame, it is recommended to backfill the abutment walls once the deck diaphragm is poured. Subsequently the backfilling shall be carried out symmetrically on both side through lifts of 0.6 m or thinner. Imbalanced backfilling may impose an inadvertent lateral shift.

Values provided for the existing fill, in Table 6-2 can be used for the design of the Temporary protection system.

## 6.7 Approach Embankments

There was no information communicated to our team regarding either grade raise or embankment widening. However, there is no concern with grade raise from a settlement perspective. Any widening beyond the toe shall be reviewed by a professional engineer. For embankment widening the toe shall be cleared of all soft and deleterious material and the subgrade shall be proof rolled. It is likely that beyond the existing embankment, removal of unsuitable overburden will expose the bedrock as subgrade.

The widening of the existing embankment shall be benched as per the OPSD 208.010. The embankment addition may be constructed using OPSS Granular A compacted to a minimum of 100% Standard Proctor Maximum Dry Density. The proposed widening shall be constructed to a minimum of 2H:1V slope or flatter.

If embankments are widened as part of this project, the side slopes embankments should be constructed with adequate erosion protection control against surface water runoff. Proper erosion control measures should be implemented during construction and for the lifetime of the bridge. This can be achieved by prompt seed and cover (OPSS 804) or sodding (OPSS 803). Some minor settlements may occur due to widening.

In case of any grade raise more than 1 m our team shall be informed to review the stability of the proposed embankment.

## 6.8 Road Reinstatement

Should road reinstatement be required over the existing embankments, as part of the bridge replacement, the recommendation included in this section can be used for the pavement design. Investigation results indicated, approximately 200 mm of gravel surface over the existing embankment fill. The following pavement structure is recommended for resurfacing the embankment upon reconstruction.

Table 6-1. Proposed Pavement Structure.

| Material | | Thickness (mm) |
|---|---|---|
| Surface | OPSS Granular M | 200 |
| Base | OPSS Granular B – Type II | 450 |

Both surface and base layers shall be compacted to a minimum of 100% Standard Proctor Maximum Dry Density.

## 6.9 Frost Protection

Based on the freezing index provided for this site, the frost penetration depth is expected to be at approximately 1.8 m below the ground surface, according to OPSD 3090.101 [5].

Frost susceptibility of the existing fill and the rock subgrade is low, therefore frost tapering is not required.

## 6.10 Corrosion Potential

Based on the test results included in Chemical Analysis section 5.2, and Appendix D, corrosion potential of the site is low to moderate. The designer shall use the test results for a product-specific design. The risk of sulphate attack on concrete is low and a general-purpose cement can be used for construction. However, bridge components that are foundered on the water should be considered as susceptible to greater corrosion loss rate.

## 6.11 Construction Considerations

As a general guideline, a minimum setback of half of the excavation height is required at all times. If this requirement cannot be accommodated, excavations can be supported by temporary shoring systems. This report does not provide information on the construction of a temporary detour embankment. Should it be required, authors shall be notified to evaluate the design and construction of detour embankments.

The overburden excavation should be completed in accordance with Ontario Regulation (O.Reg.) 213/91 under the Occupational Health and Safety Act (OHSA) with specific reference to acceptable size slopes and stabilization requirements. The general stratigraphy outlined herein can be considered an OHSA Type 4 Soil. For excavations a 3:1 horizontal to vertical side slope is mandatory for Type 4 soil.

The protection system for excavations should be in accordance with OPSS 539, Construction Specification for Temporary Protection Systems, and OPSS 902, Construction Specifications for Excavating and Backfilling – Structures. Based on the subsurface information obtained in the geotechnical boreholes, there are no specific concerns with the installation of protection system. Our investigation did not reveal presence of large size rockfill.

The contractor shall provide a coffer dam and dewatering design for the construction.

The contractor shall evaluate on-site water conditions before construction. If groundwater or running water is encountered at the time of excavation, cofferdams and pumps may be used to temporarily divert the water during the construction. The excavated subgrade must be kept dry until covered. A conventional sump and pump method should be adequate to remove the inflow water from the excavation. The water table should be lowered to a minimum of 0.6 m below the subgrade before granular compaction.

If there is any fill or unsuitable material underneath the existing culvert, it should be removed and replaced by suitable fill material conforming to the specifications of OPSS Granular A. The excavation shall be kept dry and immediately upon the removal of the final lift.

To prevent water flow through the granular backfill material, and to reduce the risk of failure due to material washout, a clay seal as per OPSD 802.095 shall be installed close to the middle of the culvert (or preferably at either end). The seal shall be 0.6 m thick and compacted to a minimum of 95% SPMDD. The clay seal shall stop below the proposed pavement structure and extend to the bottom of the bedding.

## 6.12 Corrosivity Potential

The designer is encouraged to draw a conclusion on the suitability of the material based on the chemical test results provided in the factual portion of the report. Based on the test results the corrosivity potential is mild to non-aggressive. The risk of sulphate attack on concrete is moderate to low, the designer shall use the chemical analysis report provided to determine the preferred coating option for the CSP.

# 7.0 CLOSURE

We trust this geotechnical investigation and design recommendation report meets the requirements of your project. The "Limitations of Report" presented in Appendix A are an integral part of this report. Please contact the undersigned should you have any questions or concerns.

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# GEOTECHNICAL REPORT
# TOWNLINE ROAD BRIDGE - TOWNSHIP OF ELIZABETHTOWN-KITLEY & TOWNSHIP OF RIDEAU LAKES, ONTARIO

## APPENDIX A
## LIMITATIONS OF REPORT

McINTOSH PERRY

# LIMITATIONS OF REPORT

McIntosh Perry Consulting Engineers Ltd. (McIntosh Perry) carried out the field work and prepared the report. This document is an integral part of the Foundation Investigation and Design report presented.

The conclusions and recommendations provided in this report are based on the information obtained at the borehole locations where the tests were conducted. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations where tests were conducted and conditions may become apparent during construction, which were not detected and could not be anticipated at the time of the site investigation. The benchmark level used and borehole elevations presented in this report are primarily to establish relative differenced in elevations between the borehole locations and should not be used for other purposes such as to establish elevations for grading, depth of excavations or for planning construction.

The recommendations presented in this report for design are applicable only to the intended structure and the project described in the scope of the work, and if constructed in accordance with the details outlined in the report. Unless otherwise noted, the information contained in this report does not reflect on any environmental aspects of either the site or the subsurface conditions.

The comments or recommendation provided in this report on potential construction problems and possible construction methods are intended only to guide the designer. The number of boreholes advanced at this site may not be sufficient or adequate to reveal all the subsurface information or factors that may affect the method and cost of construction. The contractors who are undertaking the construction shall make their own interpretation of the factual data presented in this report and make their conclusions, as to how the subsurface conditions of the site may affect their construction work.

The boundaries between soil strata presented in the report are based on information obtained at the borehole locations. The boundaries of the soil strata between borehole locations are assumed from geological evidences. If differing site conditions are encountered, or if the Client becomes aware of any additional information that differs from or is relevant to the McIntosh Perry findings, the Client agrees to immediately advise McIntosh Perry so that the conclusions presented in this report may be re-evaluated.

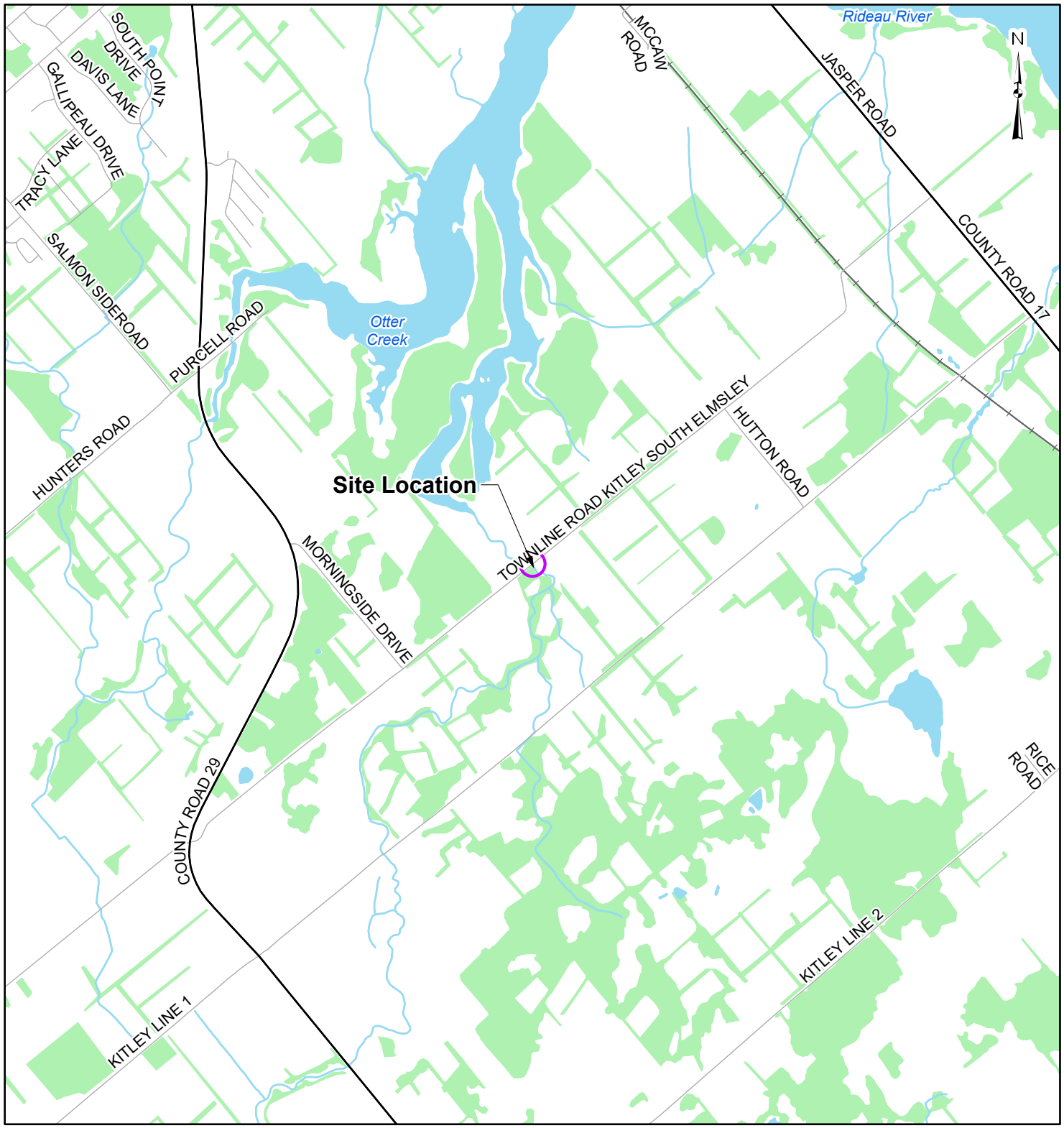
Under no circumstances shall the liability of McIntosh Perry for any claim in contract or in tort, related to the services provided and/or the content and recommendations in this report, exceed the extent that such liability is covered by such professional liability insurance from time to time in effect including the deductible therein, and which is available to indemnify McIntosh Perry. Such errors and omissions policies are available for inspection by the Client at all times upon request, and if the Client desires to obtain further insurance to protect it against any risks beyond the coverage provided by such policies, McIntosh Perry will co-operate with the Client to obtain such insurance.

McIntosh Perry prepared this report for the exclusive use of the Client. Any use which a third party makes of this report, or any reliance on or decision to be made based on it, are the responsibility of such third parties. McIntosh Perry accepts no responsibility and will not be liable for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

# GEOTECHNICAL REPORT
# TOWNLINE ROAD BRIDGE - TOWNSHIP OF ELIZABETHTOWN-KITLEY & TOWNSHIP OF RIDEAU LAKES, ONTARIO

## APPENDIX B
## SITE LOCATION

McINTOSH PERRY




LEGEND

- Local Road
- Major Road
- Railroad
- Watercourse
- Waterbody
- Wooded Area
- Approximate Site Location

500 250 0 500

Scale 1:25,000 Metres

REFERENCE

GIS data provided by the Ontario Ministry of Northern Development, Mines, Natural Resources and Forestry, 2022.

CLIENT: TSI Inc.

PROJECT: GEOTECHNICAL INVESTIGATION
TOWNLINE ROAD BRIDGE

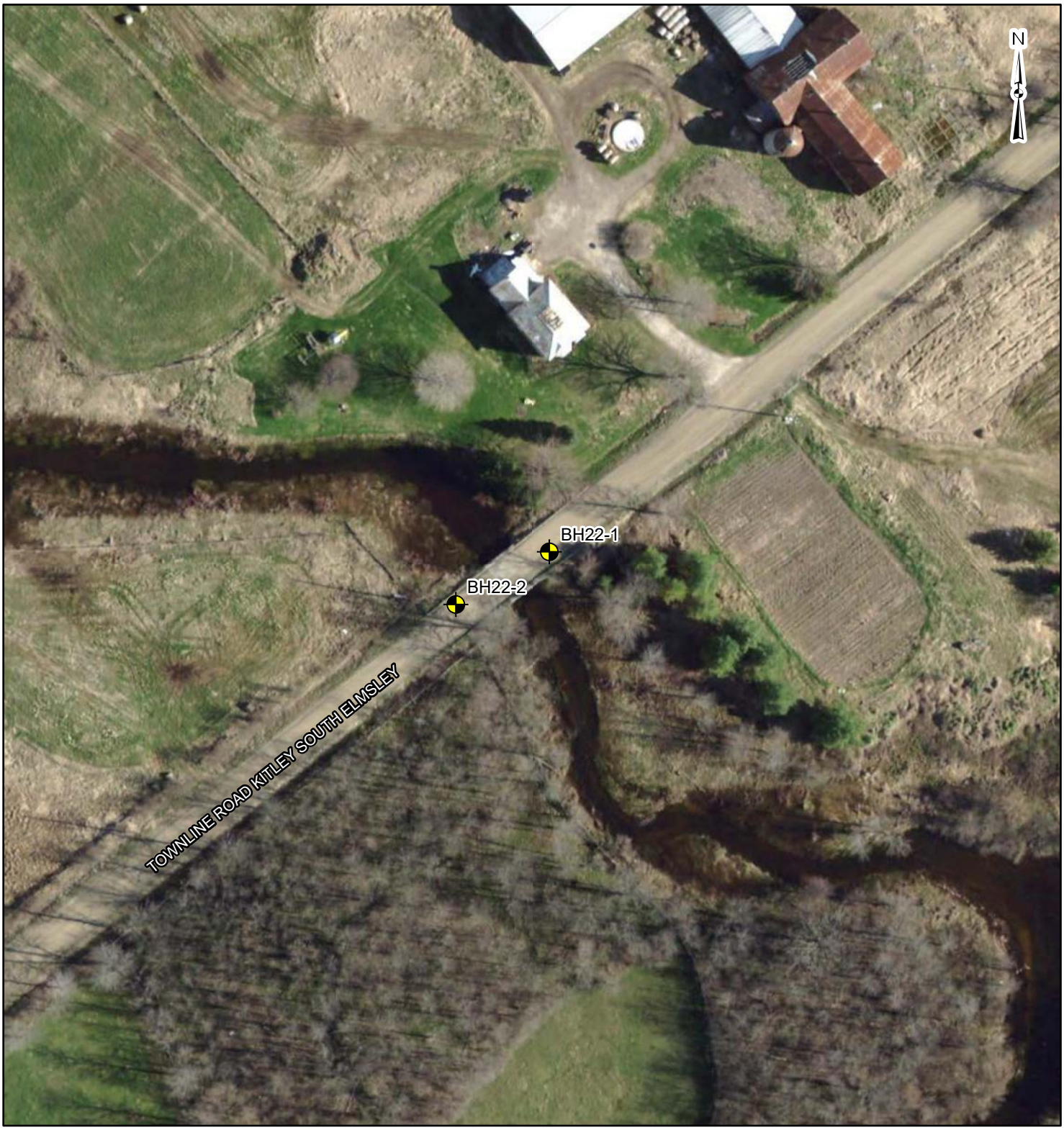
TITLE: SITE LOCATION

McINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

| | |
|---|---|
| PROJECT NO: | CCO-23-0440 |
| Date | Jun., 28, 2022 |
| GIS | MG |
| Checked By | MH |

FIGURE: 1

C:\Users\M.Grunstra\McIntosh Perry\GIS - Documents\Projects\2023\CCO\CCO-23-0440 TSI Inc Townline Rd Elizabeth Kitley\aprx\Geotech\CCO-23-0440 Townline Rd Borehole Locations.aprx




20 10 0 20

Scale 1:1,000 Metres

LEGEND

Borehole Locations

REFERENCE

GIS data provided by the Ontario Ministry of Northern Development, Mines, Natural Resources and Forestry, 2022.

CLIENT: TSI Inc.

PROJECT: GEOTECHNICAL INVESTIGATION
TOWNLINE ROAD BRIDGE

TITLE: BOREHOLE LOCATIONS

McINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

| PROJECT NO:CCO-23-0440 | | FIGURE: |
|---|---|---|
| Date | Jun., 28, 2022 | 2 |
| GIS | MG | |
| Checked By | MH | |

C:\Users\M.Grunstra\McIntosh Perry\GIS - Documents\Projects\2023\CCO\CCO-23-0440 TSI Inc Townline Rd Elizabeth Kitley\aprx\Geotech\CCO-23-0440 Townline Rd Borehole Locations.aprx

# GEOTECHNICAL REPORT
# TOWNLINE ROAD BRIDGE - TOWNSHIP OF ELIZABETHTOWN-KITLEY & TOWNSHIP OF RIDEAU LAKES, ONTARIO

## APPENDIX C
## BOREHOLE LOGS

McINTOSH PERRY

# EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ($c_u$) AS FOLLOWS:

| $C_u$ (kPa) | 0 – 12 | 12 – 25 | 25 – 50 | 50 – 100 | 100 – 200 | >200 |
|---|---|---|---|---|---|---|
| | *VERY SOFT* | *SOFT* | *FIRM* | *STIFF* | *VERY STIFF* | *HARD* |

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

| N (BLOWS/0.3m) | 0 – 5 | 5 – 10 | 10 – 30 | 30 – 50 | >50 |
|---|---|---|---|---|---|
| | *VERY LOOSE* | *LOOSE* | *COMPACT* | *DENSE* | *VERY DENSE* |

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

| RQD (%) | 0 – 25 | 25 – 50 | 50 – 75 | 75 – 90 | 90 – 100 |
|---|---|---|---|---|---|
| | *VERY POOR* | *POOR* | *FAIR* | *GOOD* | *EXCELLENT* |

JOINT AND BEDDING:

| SPACING | 50mm | 50 – 300mm | 0.3m – 1m | 1m – 3m | >3m |
|---|---|---|---|---|---|
| JOINTING | *VERY CLOSE* | *CLOSE* | *MOD. CLOSE* | *WIDE* | *VERY WIDE* |
| BEDDING | *VERY THIN* | *THIN* | *MEDIUM* | *THICK* | *VERY THICK* |

# ABBREVIATIONS AND SYMBOLS

## FIELD SAMPLING

| | | | |
|---|---|---|---|
| SS | SPLIT SPOON | TP | THINWALL PISTON |
| WS | WASH SAMPLE | OS | OSTERBERG SAMPLE |
| ST | SLOTTED TUBE SAMPLE | RC | ROCK CORE |
| BS | BLOCK SAMPLE | PH | TW ADVANCED HYDRAULICALLY |
| CS | CHUNK SAMPLE | PM | TW ADVANCED MANUALLY |
| TW | THINWALL OPEN | FS | FOIL SAMPLE |

## STRESS AND STRAIN

| | | |
|---|---|---|
| $u_w$ | kPa | PORE WATER PRESSURE |
| $r_u$ | 1 | PORE PRESSURE RATIO |
| $\sigma$ | kPa | TOTAL NORMAL STRESS |
| $\sigma'$ | kPa | EFFECTIVE NORMAL STRESS |
| $\tau$ | kPa | SHEAR STRESS |
| $\sigma_1, \sigma_2, \sigma_3$ | kPa | PRINCIPAL STRESSES |
| $\varepsilon$ | % | LINEAR STRAIN |
| $\varepsilon_1, \varepsilon_2, \varepsilon_3$ | % | PRINCIPAL STRAINS |
| E | kPa | MODULUS OF LINEAR DEFORMATION |
| G | kPa | MODULUS OF SHEAR DEFORMATION |
| $\mu$ | 1 | COEFFICIENT OF FRICTION |

## MECHANICALL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $m_v$ | $kPa^{-1}$ | COEFFICIENT OF VOLUME CHANGE |
| $c_c$ | 1 | COMPRESSION INDEX |
| $c_s$ | 1 | SWELLING INDEX |
| $c_a$ | 1 | RATE OF SECONDARY CONSOLIDATION |
| $c_v$ | $m^2/s$ | COEFFICIENT OF CONSOLIDATION |
| H | m | DRAINAGE PATH |
| $T_v$ | 1 | TIME FACTOR |
| U | % | DEGREE OF CONSOLIDATION |
| $\sigma'_{vo}$ | kPa | EFFECTIVE OVERBURDEN PRESSURE |
| $\sigma'_p$ | kPa | PRECONSOLIDATION PRESSURE |
| $\tau_f$ | kPa | SHEAR STRENGTH |
| c' | kPa | EFFECTIVE COHESION INTERCEPT |
| $\Phi'$ | $^{-o}$ | EFFECTIVE ANGLE OF INTERNAL FRICTION |
| $c_u$ | kPa | APPARENT COHESION INTERCEPT |
| $\Phi_u$ | $^{-o}$ | APPARENT ANGLE OF INTERNAL FRICTION |
| $\tau_R$ | kPa | RESIDUAL SHEAR STRENGTH |
| $\tau_r$ | kPa | REMOULDED SHEAR STRENGTH |
| $S_t$ | 1 | SENSITIVITY = $c_u / \tau_r$ |

## PHYSICAL PROPERTIES OF SOIL

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $\rho_s$ | $kg/m^3$ | DENSITY OF SOLID PARTICLES | e | 1,% | VOID RATIO | $e_{min}$ | 1,% | VOID RATIO IN DENSEST STATE |
| $\gamma_s$ | $kN/m^3$ | UNIT WEIGHT OF SOLID PARTICLES | n | 1,% | POROSITY | $I_D$ | 1 | DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$ |
| $\rho_w$ | $kg/m^3$ | DENSITY OF WATER | w | 1,% | WATER CONTENT | D | mm | GRAIN DIAMETER |
| $\gamma_w$ | $kN/m^3$ | UNIT WEIGHT OF WATER | $s_r$ | % | DEGREE OF SATURATION | $D_n$ | mm | N PERCENT – DIAMETER |
| $\rho$ | $kg/m^3$ | DENSITY OF SOIL | $w_L$ | % | LIQUID LIMIT | $C_u$ | 1 | UNIFORMITY COEFFICIENT |
| $\gamma$ | $kN/m^3$ | UNIT WEIGHT OF SOIL | $w_P$ | % | PLASTIC LIMIT | h | m | HYDRAULIC HEAD OR POTENTIAL |
| $\rho_d$ | $kg/m^3$ | DENSITY OF DRY SOIL | $w_s$ | % | SHRINKAGE LIMIT | q | $m^3/s$ | RATE OF DISCHARGE |
| $\gamma_d$ | $kN/m^3$ | UNIT WEIGHT OF DRY SOIL | $I_P$ | % | PLASTICITY INDEX = $(W_L - W_L)$ | v | m/s | DISCHARGE VELOCITY |
| $\rho_{sat}$ | $kg/m^3$ | DENSITY OF SATURATED SOIL | $I_L$ | 1 | LIQUIDITY INDEX = $(W - W_P)/ I_P$ | i | 1 | HYDAULIC GRADIENT |
| $\gamma_{sat}$ | $kN/m^3$ | UNIT WEIGHT OF SATURATED SOIL | $I_C$ | 1 | CONSISTENCY INDEX = $(W_L - W) / 1_P$ | k | m/s | HYDRAULIC CONDUCTIVITY |
| $\rho'$ | $kg/m^3$ | DENSITY OF SUBMERED SOIL | $e_{,max}$ | 1,% | VOID RATIO IN LOOSEST STATE | j | $kN/m^3$ | SEEPAGE FORCE |
| $\gamma'$ | $kN/m^3$ | UNIT WEIGHT OF SUBMERGED SOIL | | | | | | |

PROJECT NO.: **CCO-23-0440**
PROJECT: Townline Road Bridge
CLIENT: TSI Inc
PROJECT LOCATION: South Elmsley

DRILLING DATA
Date: Jun-01-2022
Method: Hollow Stem Auger
Diameter: 150
BH Location: N 4967559.6 E 344855

BH No: **BH22-1**
DATUM: MTM Zone 9
ENCL NO.: 1

| ELEV / DEPTH | DESCRIPTION | STRATA PLOT | NUMBER | TYPE | "N" BLOWS / 0.3 m | GROUND WATER CONDITIONS | DEPTH | ELEVATION | Remarks and Grain Size Distribution (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|
| 109.3 | Gravel Road Surface | | | | | | 0.0 | 109 | |
| 0.0 / 109.1 | **Gravel Road Surface: 200 mm** | | | | | | | | |
| 0.2 | **Fill: Silty Sand and Gravel**, light brown to brown, compact, dry to moist. | | 1 | GRAB | Auger | | | | 32 48 (20) |
| | | | 2 | SS | 16 | | 1.0 | 108 | |
| | | | 3 | SS | 22 | | 2.0 | | 39 38 20 3 |
| 107.0 | | | | | | | | 107 | |
| 2.3 | **Silty Sandy Gravel**, grey, stiff, moist. | | 4 | SS | 11 | | | | 38 21 23 18 |
| 106.3 | | | 5 | SS | 3/ 0.75 mm | | 3.0 | 106 | NR, SS-5 |
| 3.0 | **Bedrock** - Horizontally interbeded limestone and dolostone, light to dark grey, laminated, slightly to moderately fractureed. - Very strong, excellent quality | | 6 | RC | RQD = 95% | | 4.0 | 105 | Spoon Bouncing @ 3.1 m bgs. Run 1: TCR = 98%, UCS = 208 MPa Casing start @ 2.9 m bgs. |
| 104.8 | | | | | | | | | |
| 4.5 | **Bedrock** - Horizontally interbeded limestone and dolostone, light to dark grey, laminated, slightly to moderately fractureed. - Very strong, excellent quality | | 7 | RC | RQD = 92% | | 5.0 | 104 | Run 2: TCR = 95%, UCS = 218 MPa |
| 103.3 | | | | | | | | | |
| 6.0 | **END OF BOREHOLE** - Bedrock start @ 3.0 m bgs. | | | | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80

SHEAR STRENGTH (kPa)
Field. Shear Vane (x) & Sensitivity
● Quick Triaxial ○ Unconfined
20 40 60 80

PLASTIC LIMIT $w_P$ | NATURAL MOISTURE CONTENT $w$ | LIQUID LIMIT $w_L$
WATER CONTENT (%)
10 20 30 40 50 60 70 80 90
11
15

Unit Weight (kN/m³)
Pocket Penetro. (kPa)

GRAPH NOTES
30 Upper value = Field Vane Shear Strength
3 Lower value = Vane Sensitivity
○ 8 =3% Strain at Failure

1MP SOIL LOG TOWNLINE RD SMITHS FALLS.GPJ MP_OTTAWA_FOUNDATIONS.GDT 22-9-23

PROJECT NO.: **CCO-23-0440**
PROJECT: Townline Road Bridge
CLIENT: TSI Inc
PROJECT LOCATION: South Elmsley

DRILLING DATA
Date: Jun-01-2022
Method: Hollow Stem Auger
Diameter: 150
BH Location: N 4967549.7 E 344838.2

BH No: **BH22-2**
DATUM: MTM Zone 9
ENCL NO.: 2

| ELEV / DEPTH | DESCRIPTION | STRATA PLOT | SAMPLES NUMBER | TYPE | "N" BLOWS / 0.3 m | GROUND WATER CONDITIONS | DEPTH / ELEVATION | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80) / SHEAR STRENGTH (kPa) Field. Shear Vane (x) & Sensitivity ● Quick Triaxial ○ Unconfined (20 40 60 80) | PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT ($w_P$ w $w_L$) WATER CONTENT (%) (10 20 30 40 50 60 70 80 90) | Remarks and Grain Size Distribution (%) Unit Weight (kN/m³) Pocket Penetro. (kPa) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 109.2 / 0.0 | Gravel Road Surface | | | | | | 0.0 / 109 | | | |
| 109.0 / 0.2 | **Gravel Road Surface: 200 mm** | | | | | | | | | |
| | **Fill: Silty Sand and Gravel**, light to dark brown, compact, dry to moist. | | 1 | GRAB | Auger | | | | | |
| | | | 2 | SS | 23 | | 1.0 / 108 | | | |
| | | | 3 | SS | 4 | | 2.0 / 107 | | | Sampler spoon bouncing @ 2.6 m bgs. |
| 106.5 / 2.7 | | | 4 | SS | 47 | | | | | Coring start @ 2.9 m bgs. |
| 106.2 / 2.9 | **Bedrock**, weathered rock, gravel, light to dark grey. | | 5 | RC | RQD = 0% | | 3.0 / 106 | | | Run 1: TCR = 25%, RQD = 0% |
| | **Bedrock**<br>- Horizontally interbeded limestone and dolostone, light to dark grey, laminated, slightly to moderately fractureed.<br>- Very strong, fair quality | | 6 | RC | RQD = 63% | | | | | Run 2: TCR = 78%, USC = 139 Mpa |
| 104.7 / 4.5 | **Bedrock**<br>- Horizontally interbeded limestone and dolostone, light to dark grey, laminated, slightly to moderately fractureed.<br>- Very strong, good quality | | 7 | RC | RQD = 83% | | 4.0 / 105 | | | Run 3: TCR = 96%, UCS = 227 MPa |
| 104.1 / 5.1 | **Bedrock**<br>- Horizontally interbeded limestone and dolostone, light to dark grey, laminated, slightly to moderately fractureed. | | 8 | RC | RQD = 86% | | 5.0 / 104 | | | Core barrel stuck @ 5.1 m bgs. Run 4: TCR = 91%, UCS = 106 MPa |
| 103.2 / 5.9 | - Very strong, good quality<br>**Bedrock**<br>- Horizontally interbeded limestone and dolostone, light to dark grey, laminated, slightly to moderately fractureed.<br>- Very strong, excellent quality | | 9 | RC | RQD = 92% | | 6.0 / 103 | | | Run 5: TCR = 97%, UCS = 187 MPa |
| 101.7 / 7.5 | **END OF BOREHOLE**<br>- Bedrock start @ 2.6 m bgs. | | | | | | 7.0 / 102 | | | |

GRAPH NOTES
30 / 3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity
○ ε=3% Strain at Failure

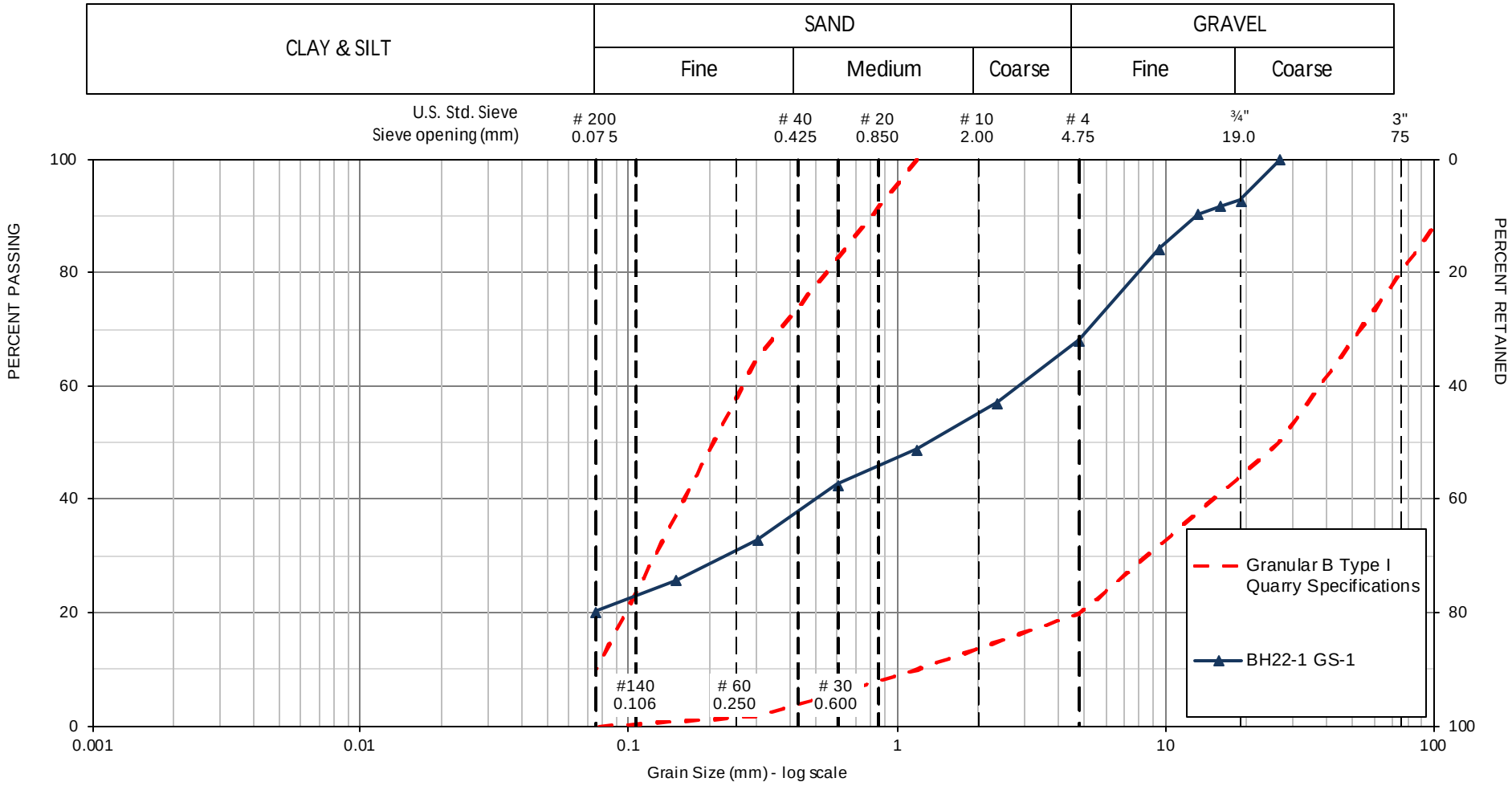
1MP SOIL LOG TOWNLINE RD SMITHS FALLS.GPJ MP_OTTAWA_FOUNDATIONS.GDT 22-9-23

# GEOTECHNICAL REPORT
# TOWNLINE ROAD BRIDGE - TOWNSHIP OF ELIZABETHTOWN-KITLEY & TOWNSHIP OF RIDEAU LAKES, ONTARIO

## APPENDIX D
## LABORATORY TEST RESULTS

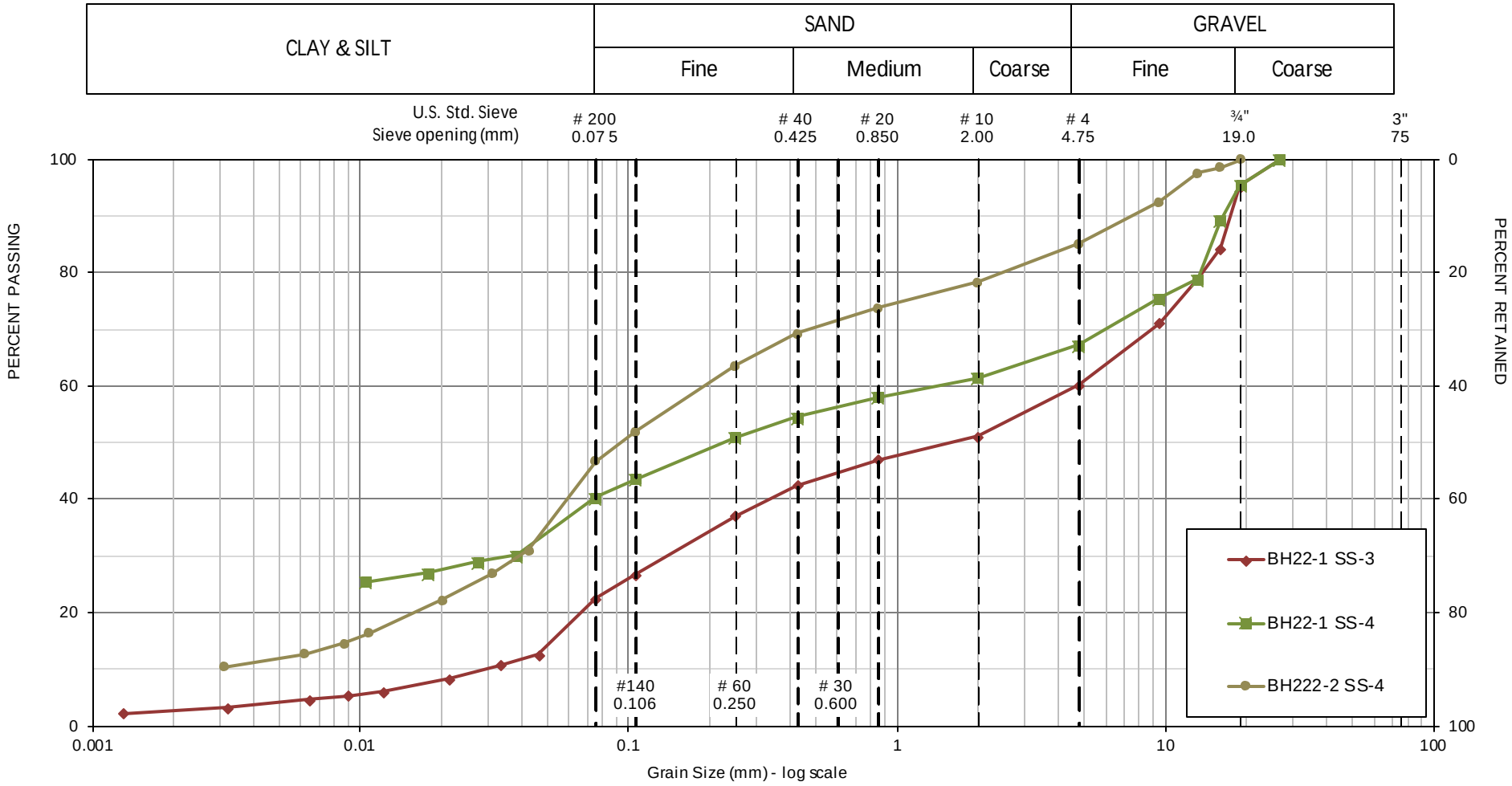
McINTOSH PERRY

# Unified Soil Classification System (USCS)




| | | |
|---|---|---|
| McINTOSH PERRY | Grain-Size Distribution Curve | Figure No. 3 |
| | Fill | |
| | Townline Road Bridge | CCO-23-0440-02 |

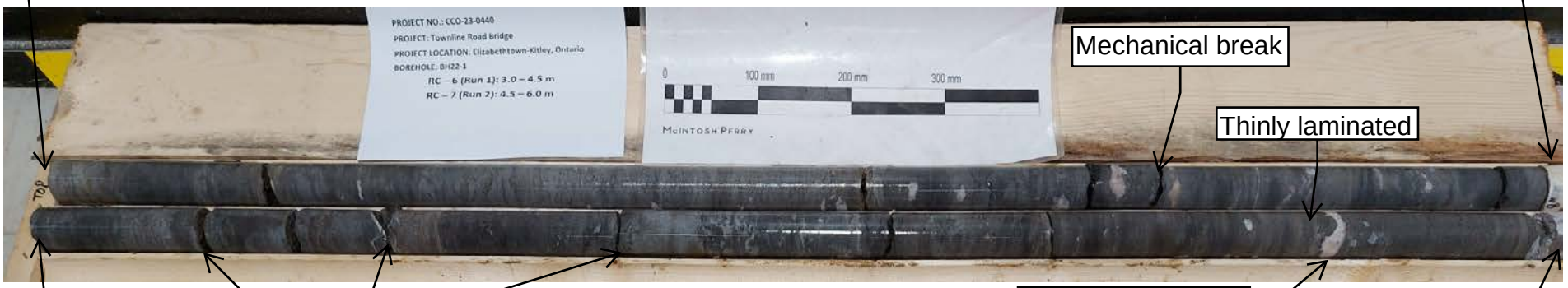
# Unified Soil Classification System (USCS)




McINTOSH PERRY

| Grain-Size Distribution Curve | Figure No. 4 |
|---|---|
| Silty Sand and Gravel to Silty Sand | |
| Townline Road Bridge | CCO-23-0440-02 |

BH22-1
Townline Road, South Elsmsley.




3.0 m

4.5 m

Mechanical break

Thinly laminated

Fractures

Thickly laminated

4.5 m

6.0 m

Horizontally interbedded limestone and dolostone, light to dark grey, laminated, slightly to moderately fracture. Possible mineral composition: Calcite, and dolomite.

McINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A 1L0
Tel: 613-836-2194 Fax: 613-836-3742
www.mcintoshperry.com

Client: TSI Inc.
Project Name: Townline Road Bridge (SN16-511)
Project Location: Twp Elizabethtown-Kitley & Twp Rideau Lakes

Project No.: CCO-23-0440 Figure No.: 5

BH22-2
Townline Road, South Elsmsley.


Horizontally interbedded limestone and dolostone, light to dark grey, laminated, slightly to moderately fracture. Possible mineral composition: Calcite, and dolomite.

McINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A 1L0
Tel: 613-836-2194 Fax: 613-836-3742
www.mcintoshperry.com

Client: TSI Inc.
Project Name: Townline Road Bridge (SN16-511)
Project Location: Twp Elizabethtown-Kitley & Twp Rideau Lakes
Project No.: CCO-23-0440 Figure No.: 6

McINTOSH PERRY

# Unconfined Compressive Strength of Intact Rock Cores

# ASTM D7012 Method C

| | | | |
|---|---|---|---|
| **Project No.:** | CCO-23-0440-02-02 | **Date Issued:** | June 16,2022 |
| **Lab No.:** | OL-22048 | **Report No.:** | 1 of 2 |
| **Project Name:** | Townline Bridge | | |

| | | | | | |
|---|---|---|---|---|---|
| **Core No.:** | 1 | **Moisture Condition:** | Dry as received | | |
| **Borehole Location:** | BH22-1 | **RC/Run:** | RC-6 / Run-1 | **Depth (ft):** | 3.0-4.5m |
| **Date Sampled:** | June 1,2022 | **Received:** | June 8,2022 | **Tested:** | June 15,2022 |
| **Core No.:** | 2 | **Moisture Condition:** | Dry as received | | |
| **Borehole Location:** | BH22-1 | **RC/Run:** | RC-7 / Run-2 | **Depth (ft):** | 4.5-6.0m |
| **Date Sampled:** | June 1,2022 | **Received:** | June 8,2022 | **Tested:** | June 15,2022 |
| **Core No.:** | 3 | **Moisture Condition:** | Dry as received | | |
| **Borehole Location:** | BH22-2 | **RC/Run:** | RC-6 / Run-2 | **Depth (ft):** | 2.9-4.4m |
| **Date Sampled:** | June 1,2022 | **Received:** | June 8,2022 | **Tested:** | June 15,2022 |

| **Core No. :** | 1 | 2 | 3 |
|---|---|---|---|
| **Diameter (mm)** | 47.2 | 47.2 | 47.2 |
| **Thickness/Height (mm)** | 106.3 | 107.3 | 108.7 |
| **Density (Kg/m$^3$)** | 2770 | 2764 | 2760 |
| **Compressive Strength (Mpa)** | 208.1 | 217.6 | 138.5 |
| **Mass of Core (Kg)** | 0.508 | 0.518 | 0.525 |
| **Description of Failure** | 2 | 1 & 2 | 3 |

**Remarks:** Type #2 Reasonably well-formed cone on one end, vertical cracks through core.

Core# 2 Type 1 &2 - Reasonably well-formed cones on both ends, vertical columnar cracks through out middle of core.

Type #3 Columnar vertical cracking through both ends, no well-formed cones.

**Reviewed By:** _____ **Date:** June 16,2022

Jason Hopwood-Jones
Laboratory Manager

McINTOSH PERRY

# Unconfined Compressive Strength of Intact Rock Cores

## ASTM D7012 Method C

| | | | |
|---|---|---|---|
| **Project No.:** | CCO-23-0440-02-02 | **Date Issued:** | June 16,2022 |
| **Lab No.:** | OL-22048 | **Report No.:** | 2 of 2 |
| **Project Name:** | Townline Bridge | | |

| | | | | | |
|---|---|---|---|---|---|
| **Core No.:** | 4 | **Moisture Condition:** | Dry as received | | |
| **Borehole Location:** | BH22-2 | **RC/Run:** | RC-7 / Run-3 | **Depth (ft):** | 4.4-5.1m |
| **Date Sampled:** | June 1,2022 | **Received:** | June 8,2022 | **Tested:** | June 15,2022 |
| **Core No.:** | 5 | **Moisture Condition:** | Dry as received | | |
| **Borehole Location:** | BH22-2 | **RC/Run:** | RC-8 / Run-4 | **Depth (ft):** | 5.1-5.9m |
| **Date Sampled:** | June 1,2022 | **Received:** | June 8,2022 | **Tested:** | June 15,2022 |
| **Core No.:** | 6 | **Moisture Condition:** | Dry as received | | |
| **Borehole Location:** | BH22-2 | **RC/Run:** | RC-9 / Run-5 | **Depth (ft):** | 5.9-7.5m |
| **Date Sampled:** | June 1,2022 | **Received:** | June 8,2022 | **Tested:** | June 15,2022 |

| **Core No. :** | 4 | 5 | 6 |
|---|---|---|---|
| **Diameter (mm)** | 47.2 | 47.2 | 47.2 |
| **Thickness/Height (mm)** | 107.6 | 109.3 | 110.6 |
| **Density (Kg/m$^3$)** | 2751 | 2713 | 2749 |
| **Compressive Strength (Mpa)** | 226.7 | 105.8 | 186.9 |
| **Mass of Core (Kg)** | 0.518 | 0.519 | 0.534 |
| **Description of Failure** | 03-Jan | 3 | 3 |

**Remarks:** Note: All cores at approximately 100 to 200 Mpa started to flake shards of rock vertically.

**Reviewed By:**  **Date:** June 16,2022

Jason Hopwood-Jones
Laboratory Manager

McIntosh Perry 104-215 Menten Place Nepean, ON K2H 9C1 Ph.: 613-453-0751 email: j.hopwood-jones@mcintoshperry.com



| Core # | RC Sample # | RUN # | Depth (m) | Diameter (mm) | Thickness/ Height (mm) | Density (kg/m$^3$) | Compressive Strength (MPa) | Mass of Core (g) |
|---|---|---|---|---|---|---|---|---|
| 1 | BH22-1 RC-6 | 1 | 3.0 - 4.5 | 47 | 106 | 2770 | 208 | 508 |

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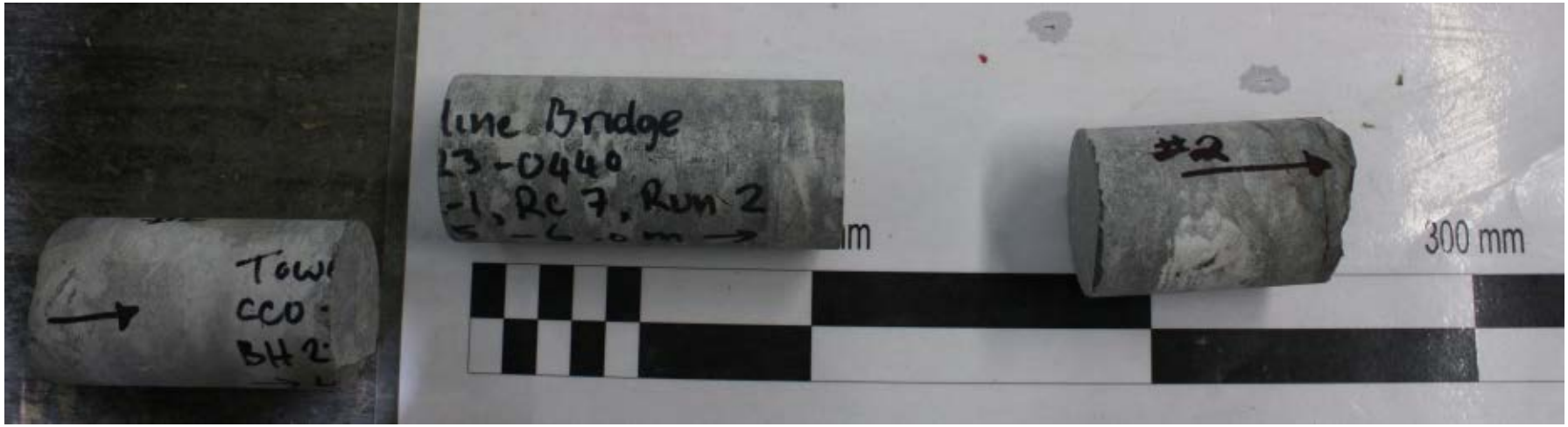
Project Name: Townline Road Bridge
Project Location: Twp Elizabethtown-Kitley & Twp Rideau Lakes

Project No.: CCO-23-0440 Figure No.: 7





| Core # | RC Sample # | RUN # | Depth (m) | Diameter (mm) | Thickness/ Height (mm) | Density (kg/m$^3$) | Compressive Strength (MPa) | Mass of Core (g) |
|---|---|---|---|---|---|---|---|---|
| 2 | BH22-1 RC-7 | 2 | 4.5 - 6.0 | 47 | 107 | 2764 | 218 | 518 |

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Project Name: Townline Road Bridge
Project Location: Twp Elizabethtown-Kitley & Twp Rideau Lakes
Project No.: CCO-23-0440 Figure No.: 8



| Core # | RC Sample # | RUN # | Depth (m) | Diameter (mm) | Thickness/ Height (mm) | Density (kg/m$^3$) | Compressive Strength (MPa) | Mass of Core (g) |
|---|---|---|---|---|---|---|---|---|
| 3 | BH22-2 RC-6 | 2 | 2.9 - 404 | 47 | 109 | 2760 | 139 | 525 |

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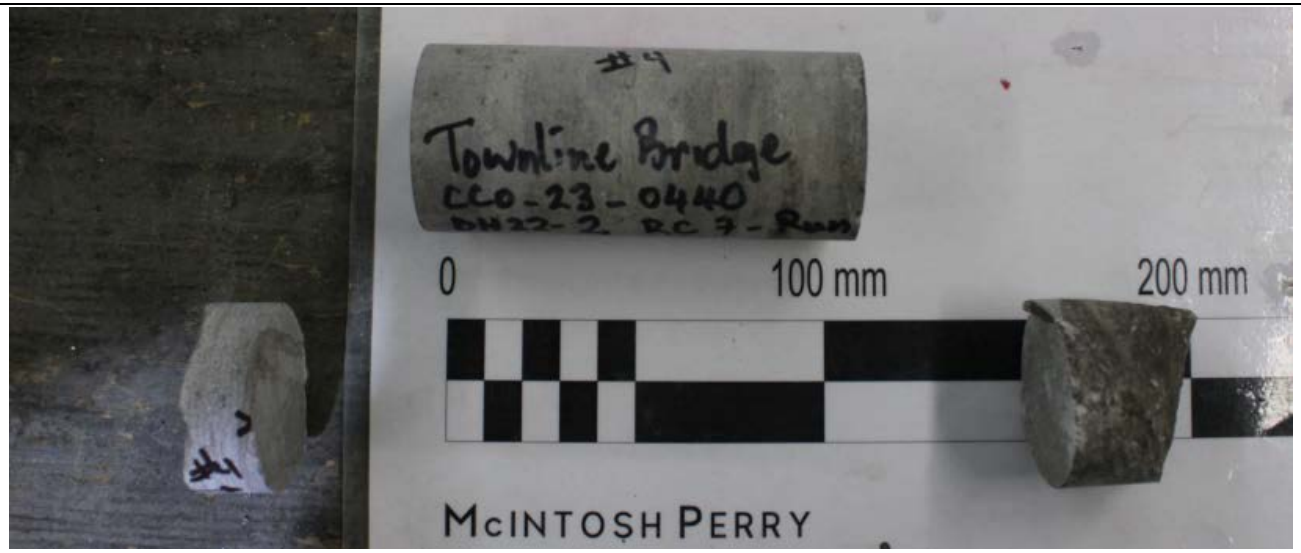
Project Name: Townline Road Bridge
Project Location: Twp Elizabethtown-Kitley & Twp Rideau Lakes
Project No.: CCO-23-0440 Figure No.: 9





| Core # | RC Sample # | RUN # | Depth (m) | Diameter (mm) | Thickness/ Height (mm) | Density (kg/m$^3$) | Compressive Strength (MPa) | Mass of Core (g) |
|---|---|---|---|---|---|---|---|---|
| 4 | BH22-2 RC-7 | 3 | 4.4 - 5.1 | 47 | 108 | 2751 | 227 | 518 |

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Project Name: Townline Road Bridge
Project Location: Twp Elizabethtown-Kitley & Twp Rideau Lakes

Project No.: CCO-23-0440 Figure No.: 10



| Core # | RC Sample # | RUN # | Depth (m) | Diameter (mm) | Thickness/ Height (mm) | Density ($kg/m^3$) | Compressive Strength (MPa) | Mass of Core (g) |
|---|---|---|---|---|---|---|---|---|
| 5 | BH22-2 RC-8 | 4 | 5.1 - 5.9 | 47 | 109 | 2713 | 106 | 519 |

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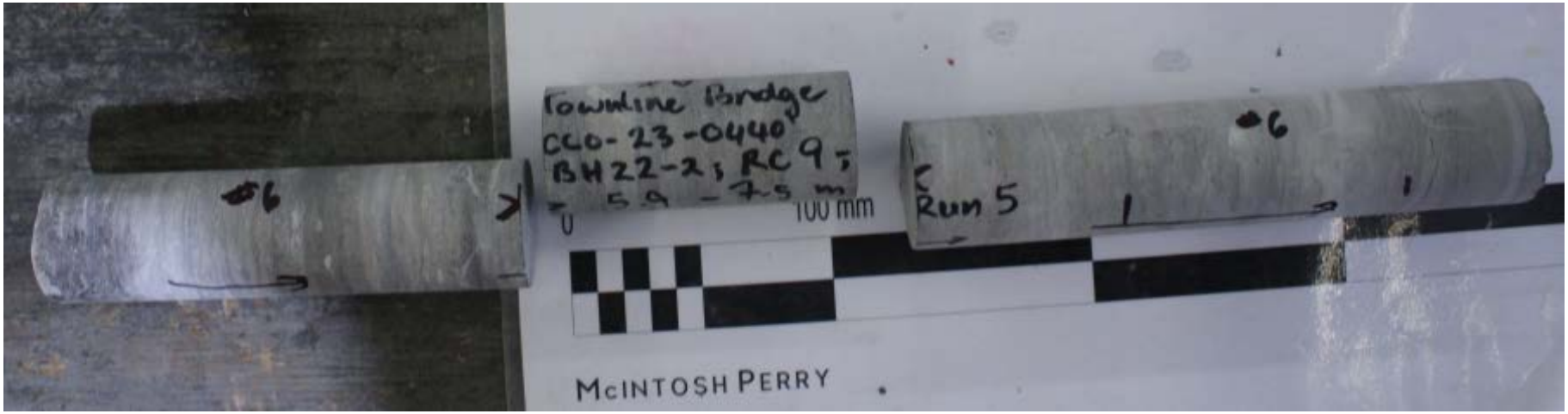
Client: TSI Inc.
Project Name: Townline Road Bridge
Project Location: Twp Elizabethtown-Kitley & Twp Rideau Lakes

Project No.: CCO-23-0440 Figure No.: 11





| Core # | RC Sample # | RUN # | Depth (m) | Diameter (mm) | Thickness/ Height (mm) | Density (kg/m$^3$) | Compressive Strength (MPa) | Mass of Core (g) |
|---|---|---|---|---|---|---|---|---|
| 6 | BH22-2 RC-9 | 5 | 5.9 - 7.5 | 47 | 111 | 2749 | 187 | 534 |

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Client: TSI Inc.
Project Name: Townline Road Bridge
Project Location: Twp Elizabethtown-Kitley & Twp Rideau Lakes

Project No.: CCO-23-0440 Figure No.: 12

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# Certificate of Analysis

**McIntosh Perry Consulting Eng. (Nepean)**

215 Menten Place, Unit 104
Nepean, ON K2H9C1
Attn: Mizral Hameem

Client PO: CCO-23-0440
Project: Townline Road Bridge
Custody:

Report Date: 29-Jul-2022
Order Date: 25-Jul-2022

**Order #: 2231092**

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

| Paracel ID | Client ID |
|---|---|
| 2231092-01 | BH22-2 SS-3 |

Approved By:

Dale Robertson, BSc
Laboratory Director

Certificate of Analysis

Report Date: 29-Jul-2022

**Client: McIntosh Perry Consulting Eng. (Nepean)**

Order Date: 25-Jul-2022

**Client PO: CCO-23-0440**

**Project Description: Townline Road Bridge**

## Analysis Summary Table

| Analysis | Method Reference/Description | Extraction Date | Analysis Date |
|---|---|---|---|
| Anions | EPA 300.1 - IC, water extraction | 28-Jul-22 | 28-Jul-22 |
| pH, soil | EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext. | 26-Jul-22 | 26-Jul-22 |
| Resistivity | EPA 120.1 - probe, water extraction | 26-Jul-22 | 26-Jul-22 |
| Solids, % | Gravimetric, calculation | 26-Jul-22 | 26-Jul-22 |

Certificate of Analysis

Report Date: 29-Jul-2022

**Client: McIntosh Perry Consulting Eng. (Nepean)**

Order Date: 25-Jul-2022

**Client PO: CCO-23-0440**

**Project Description: Townline Road Bridge**

| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | BH22-2 SS-3 | - | - | - |
| | **Sample Date:** | 01-Jun-22 12:00 | - | - | - |
| | **Sample ID:** | 2231092-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 79.9 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 7.20 [1] | - | - | - |
| Resistivity | 0.10 Ohm.m | 29.6 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 88 [1] [3] | - | - | - |
| Sulphate | 5 ug/g dry | 29 [1] [3] | - | - | - |

## Method Quality Control: Blank

| Analyte | Result | Reporting Limit | Units | Source Result | %REC | %REC Limit | RPD | RPD Limit | Notes |
|---|---|---|---|---|---|---|---|---|---|
| **General Inorganics** | | | | | | | | | |
| Resistivity | ND | 0.10 | Ohm.m | | | | | | |

Certificate of Analysis

Report Date: 29-Jul-2022

**Client: McIntosh Perry Consulting Eng. (Nepean)**

Order Date: 25-Jul-2022

**Client PO: CCO-23-0440**

**Project Description: Townline Road Bridge**

## Method Quality Control: Duplicate

| Analyte | Result | Reporting Limit | Units | Source Result | %REC | %REC Limit | RPD | RPD Limit | Notes |
|---|---|---|---|---|---|---|---|---|---|
| **General Inorganics** | | | | | | | | | |
| pH | 7.25 | 0.05 | pH Units | 7.24 | | | 0.1 | 2.3 | |
| Resistivity | 59.3 | 0.10 | Ohm.m | 60.3 | | | 1.8 | 20 | |
| **Physical Characteristics** | | | | | | | | | |
| % Solids | 90.6 | 0.1 | % by Wt. | 90.3 | | | 0.3 | 25 | |

**Qualifier Notes:**

***Login Qualifiers :***

Sample - One or more parameter received past hold time - Chloride, pH, Sulphate

*Applies to samples: BH22-2 SS-3*

***Sample Qualifiers :***

1 : Holding time had been exceeded upon receipt of the sample at the laboratory or prior to the analysis being requested.

3 : Subcontracted analysis - Testmark.

**Sample Data Revisions**

None

**Work Order Revisions / Comments:**

None

**Other Report Notes:**

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

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# Certificate of Analysis

**McIntosh Perry Consulting Eng. (Nepean)**

215 Menten Place, Unit 104
Nepean, ON K2H9C1
Attn: Mizral Hameem

Client PO: CCO 230440
Project: CCO 230440, Townline
Custody: 13354

Report Date: 6-Jul-2022
Order Date: 30-Jun-2022

**Order #: 2227439**

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

| Paracel ID | Client ID |
|---|---|
| 2227439-01 | SW-22-1 |

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis

Report Date: 06-Jul-2022

**Client: McIntosh Perry Consulting Eng. (Nepean)**

Order Date: 30-Jun-2022

**Client PO: CCO 230440**

**Project Description: CCO 230440, Townline**

## Analysis Summary Table

| Analysis | Method Reference/Description | Extraction Date | Analysis Date |
|---|---|---|---|
| Anions | EPA 300.1 - IC | 5-Jul-22 | 5-Jul-22 |
| pH | EPA 150.1 - pH probe @25 °C | 6-Jul-22 | 6-Jul-22 |
| Resistivity | EPA 120.1 - probe | 6-Jul-22 | 6-Jul-22 |

Certificate of Analysis

Report Date: 06-Jul-2022

**Client:** McIntosh Perry Consulting Eng. (Nepean)

Order Date: 30-Jun-2022

**Client PO:** CCO 230440

**Project Description:** CCO 230440, Townline

| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | SW-22-1 | - | - | - |
| | **Sample Date:** | 30-Jun-22 07:30 | - | - | - |
| | **Sample ID:** | 2227439-01 | - | - | - |
| | **MDL/Units** | Water | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.1 pH Units | 7.9 | - | - | - |
| Resistivity | 0.01 Ohm.m | 25.2 | - | - | - |
| **Anions** | | | | | |
| Chloride | 1.0 mg/L | 11.3 | - | - | - |
| Sulphate | 1.0 mg/L | <1.0 | - | - | - |

Certificate of Analysis

Report Date: 06-Jul-2022

**Client: McIntosh Perry Consulting Eng. (Nepean)**

Order Date: 30-Jun-2022

**Client PO: CCO 230440**

**Project Description: CCO 230440, Townline**

## Method Quality Control: Blank

| Analyte | Result | Reporting Limit | Units | Source Result | %REC | %REC Limit | RPD | RPD Limit | Notes |
|---|---|---|---|---|---|---|---|---|---|
| **Anions** | | | | | | | | | |
| Chloride | ND | 1.0 | mg/L | | | | | | |
| Sulphate | ND | 1.0 | mg/L | | | | | | |
| **General Inorganics** | | | | | | | | | |
| Resistivity | ND | 0.01 | Ohm.m | | | | | | |

## Method Quality Control: Duplicate

| Analyte | Result | Reporting Limit | Units | Source Result | %REC | %REC Limit | RPD | RPD Limit | Notes |
|---|---|---|---|---|---|---|---|---|---|
| **General Inorganics** | | | | | | | | | |
| pH | 7.0 | 0.1 | pH Units | 7.0 | | | 1.0 | 10 | |
| Resistivity | 25.2 | 0.01 | Ohm.m | 25.2 | | | 0.1 | 20 | |

Certificate of Analysis

Report Date: 06-Jul-2022

**Client: McIntosh Perry Consulting Eng. (Nepean)**

Order Date: 30-Jun-2022

**Client PO: CCO 230440**

**Project Description: CCO 230440, Townline**

## Method Quality Control: Spike

| Analyte | Result | Reporting Limit | Units | Source Result | %REC | %REC Limit | RPD | RPD Limit | Notes |
|---|---|---|---|---|---|---|---|---|---|
| **Anions** | | | | | | | | | |
| Chloride | 10.6 | 1.0 | mg/L | ND | 106 | 85-115 | | | |
| Sulphate | 11.1 | 1.0 | mg/L | ND | 111 | 86-114 | | | |

**Qualifier Notes:**

None

**Sample Data Revisions**

None

**Work Order Revisions / Comments:**

None

**Other Report Notes:**

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

# GEOTECHNICAL REPORT
# TOWNLINE ROAD BRIDGE - TOWNSHIP OF ELIZABETHTOWN-KITLEY & TOWNSHIP OF RIDEAU LAKES, ONTARIO

## APPENDIX E
## SEISMIC HAZARD CALCULATION

McINTOSH PERRY

# 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:** 44.847N 75.993W **User File Reference:** Townline Road Bridge 2022-06-28 13:54 UT

**Requested by:** McIntosh Perry Consulting Engineering Ltd.

| 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) | 0.225 | 0.138 | 0.089 | 0.030 |
| Sa (0.1) | 0.283 | 0.179 | 0.119 | 0.043 |
| Sa (0.2) | 0.254 | 0.162 | 0.109 | 0.042 |
| Sa (0.3) | 0.204 | 0.131 | 0.089 | 0.034 |
| Sa (0.5) | 0.155 | 0.099 | 0.067 | 0.025 |
| Sa (1.0) | 0.086 | 0.055 | 0.036 | 0.013 |
| Sa (2.0) | 0.043 | 0.027 | 0.018 | 0.005 |
| Sa (5.0) | 0.012 | 0.007 | 0.004 | 0.001 |
| Sa (10.0) | 0.004 | 0.003 | 0.002 | 0.001 |
| PGA (g) | 0.159 | 0.100 | 0.066 | 0.024 |
| PGV (m/s) | 0.131 | 0.080 | 0.051 | 0.017 |

**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

# GEOTECHNICAL REPORT
# TOWNLINE ROAD BRIDGE - TOWNSHIP OF ELIZABETHTOWN-KITLEY & TOWNSHIP OF RIDEAU LAKES, ONTARIO

## APPENDIX F
## RELEVANT PHOTOS

McINTOSH PERRY


06/05/2022 10:25


06/05/2022 10:25


06/05/2022 10:27


06/05/2022 10:27

# References

[1] Ministry of Northern Development, Mines, Natural Resources and Forestry, "OGS Earth," 2022. [Online]. Available: https://www.geologyontario.mndm.gov.on.ca/ogsearth.html. [Accessed 07 06 2022].

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[11] Government of Ontario, "Ontario Regulation 213/91," 01 January 2022. [Online]. Available: https://www.ontario.ca/laws/regulation/910213#BK42. [Accessed 09 May 2022].