



**THURBER** ENGINEERING LTD.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT  
WIDENING OF THE HIGHWAY 401 OVERPASS OVER GANARASKA RIVER  
NORTHUMBERLAND COUNTY – PORT HOPE, ONTARIO  
SITE 21-231, G.W.P. 4078-14-00  
ASSIGNMENT NO.: 4014-E-0014**

**GEOCRES NO.: 30M16-62**

Report to:  
**WSP CANADA**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual data obtained from a preliminary foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the proposed widening of the Highway 401 overpass structure located at the Ganaraska River in Port Hope, Ontario. Thurber carried out the investigation as a subconsultant to WSP Canada (WSP), under Agreement No. 4014-E-0014.

Base plan mapping was provided by WSP for the preparation of this report.

The purpose of this preliminary investigation was to explore the subsurface conditions at the site and, based on this data, provide a borehole location plan, record of boreholes, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions.

**2 SITE DESCRIPTION**

Site 21-231 is located on Highway 401, approximately 0.45 km west of the Highway 401 County Road 28 underpass in Port Hope, Ontario. The location of the structure is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

Highway 401 at the location of the bridge has three through lanes in each direction. Traffic volumes on this section of Highway 401 are understood to be 50,000 AADT (2016). It is noted that for project orientation purposes, Highway 401 will be assumed to be oriented east-west and the Ganaraska River to be oriented north-south with river flow to the south. The eastbound and westbound lanes are separated by a median barrier wall. There are steel beam guide rails located along the outside lanes east and west of the bridge and a concrete barrier where the highway crosses over the river.

Based on the historical contract documents, the existing three-span, steel plate girder structure is approximately 73.0 m long, and 29.0 m wide.

The site is located within a physiographic region known as the Iroquois Plain. This area was formed by a body of water known as Lake Iroquois and is characterized by lacustrine deposits of sand, silts and clays. Along Highway 401, within the project limits, the principal overburden consists of till and clay with occasional drumlins (Chapman and Putnam, 1984).

The lands surrounding the site consist of the Port Hope Conservation Area to the north of Highway 401. Corbitt's Dam and residential properties are located to the south of Highway 401. Storm water drainage in the area is to existing catch basins and the Ganaraska River. Site photographs

showing the general conditions at the site are presented along Highway 401 and the Ganaraska River are presented in Appendix D.

The existing approach embankments range in height from approximately 10.7 m to 12.1 m. The embankment side slopes extend down at approximately 2H:1V. The slopes are vegetated with a combination of long grass and brush.

### **3 SITE INVESTIGATION AND FIELD TESTING**

#### **3.1 Previous Investigations**

The GEOCRETS Report 30M16-7 dated July 12<sup>th</sup>, 1957 presents the results of the investigations carried out for the design and construction of the bridge structure and approach embankments. The investigation included six boreholes for the structure. A supplemental approach embankment investigation was also carried out that included three short boreholes to refusal and five auger probe holes; the results were included as an addendum to the original investigation report. Two of the structure boreholes were advanced approximately 3.0 m into the limestone bedrock while all approach boreholes were advanced to refusal on inferred bedrock.

Prior to construction of the bridge and Highway 401 in 1957, the stratigraphy in the area of the bridge was generally described as surficial deposits of organic silt, overlying a thin deposit of very dense silty coarse sand. The overburden soil at the site is underlain by sound limestone bedrock based on AXT rock coring. The Borehole Logs indicated the bedrock surface at around elevation 295 to 297 feet (89.9 to 90.5 m).

The report noted concerns regarding the stability of the embankments. Failure of the underlying soil was predicted during placement of the embankment fill (up to 35 feet (10.7 m) of fill). It was recommended in the supplemental investigation letter that a trench 1.5 m (5 ft.) deep, 9.1 m (30 ft.) wide and 30.5 m (100 ft.) long be excavated and the excavated material be replaced with properly compacted granular fill before commencing the embankment fill placement. It is not known if this subgrade treatment was carried out.

#### **3.2 Site Investigation**

The site investigation and field testing program was carried out between May 30<sup>th</sup> and June 1<sup>st</sup>, 2016.

Thurber contacted Ontario One Call in advance of the field investigation to obtain utility locates/clearances in the vicinity of the intended borehole locations. In addition, MTO traffic operations was contacted to obtain ATMS Fibre utility locates and RW Electric was contacted to obtain MTO electric line locates for the project limits.

The site investigation and field testing program included advancing four boreholes. The northing, easting and elevation of the boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A and are summarized in Table 3-1. The site is within MTM Zone 10. The elevations were surveyed relative to the first order vertical benchmark tablet number PBM 61-69 set in the south face of the west abutment of the Choate Road Overpass which has a geodetic elevation of 109.880 m.

**Table 3-1: Borehole Summary**

Borehole No.	Drilled Location	Approximate Northing (m)	Approximate Easting (m)	Ground Surface Elevation (m)	Sample Termination Depth (m)
401	West abutment – WB Lane 3	4 870 654.5	401 513.7	106.0	14.6
402	West abutment – EB Lane 3	4 870 631.4	401 517.2	106.0	19.7
403	East abutment – WB Lane 3	4 870 667.1	401 602.1	103.3	19.8
404	East abutment – EB Lane 3	4 870 645.1	401 606.0	103.4	15.3

The boreholes were advanced with CME truck mounted drill rigs equipped with hollow stem augers. The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Split spoon samples were collected at regular depth intervals in the boreholes via the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586-11. All soil samples recovered from the boreholes were placed in moisture-proof containers and the samples were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing. Bedrock was cored in Boreholes 402 and 403 with HQ size coring equipment following ASTM Standard D6032-08. Bedrock core samples were stored in core boxes for transport.

A 25 mm diameter standpipe piezometer was installed in Borehole 403 to allow for the measurement of the groundwater level at the site. Piezometer construction details are illustrated on the Record of Borehole sheet for Borehole 403, provided in Appendix B. The piezometer was decommissioned in accordance with Ontario MOE Regulation 903 on May 31, 2016, after the final water level measurement.

The boreholes without piezometer installations were backfilled with a low-permeability combination of auger cuttings and bentonite pellets in accordance with Ontario MOE Regulation 903 as amended. All boreholes were capped with 150 mm of cold patch asphalt underlain by 150 mm layer of concrete.

### 3.3 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples. Grain size distribution analyses and Atterberg Limit testing were also carried out on selected samples to MTO and ASTM standards. All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were measured. The laboratory test results are presented on the Record of Borehole sheets in Appendix B and are illustrated on the figures in Appendix C.

Chemical analysis for determination of pH, resistivity, soluble sulphate and chloride concentrations was carried out on two soil samples. A copy of the chemical analysis results is provided in Appendix C.

## 4 DESCRIPTION OF SUBSURFACE CONDITIONS

### 4.1 Overview / General

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata Drawing included in Appendix A. An overall description of the stratigraphy is given in the following sections, however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions. It must be recognized that soil and groundwater conditions may vary between and beyond sampled locations.

In general, the stratigraphy in the area of the boreholes is characterized by an asphalt pavement underlain by embankment fill overlying native organic silt, overlying glacial till, and underlain by limestone bedrock. The upper portion of the embankment fill generally consisted of silty sand with gravel with occasional cobbles. The lower portion of the embankment fill was more variable and included clay, silty sand some gravel, sandy silt, sandy clay and silty sand.

More detailed descriptions of the individual strata are presented below.

### 4.2 Asphalt

The four boreholes were advanced through the existing Highway 401 pavement structure. The thickness of the asphalt measured at the borehole locations ranged from 240 mm to 350 mm.

### 4.3 Fill

#### 4.3.1 Upper Embankment Fill: Silty Sand with Gravel

A fill layer consisting predominantly of silty sand and gravel was encountered below the asphalt surface in all the boreholes. The top of this layer ranged from elevation 105.7 m to 103.0 m. The thickness of this layer ranged from 4.6 m to 5.2 m. The SPT 'N' values ranged from 6 blows for 0.3 m of penetration to greater than 100 indicating a loose to very dense condition; but typically compact to dense.

The moisture content of the samples tested ranged from 2% to 12%. The results of grain size analysis conducted on four samples of this material are summarized in Table 4-1 and are illustrated on Figure C1 in Appendix C.

**Table 4-1: Gradation Results for Silty Sand with Gravel Fill**

Soil Particle	Percentage (%)
Gravel	20 to 27
Sand	60 to 68
Silt and Clay	10 to 17

#### 4.3.2 Lower Embankment Fill: Variable

The embankment fill encountered beneath the upper granular embankment fill was variable in composition and included clay, silty sand some gravel, sandy silt, sandy clay and silty sand. The top of this layer ranged from elevation 101.1 m to 97.8 m. The thickness of this layer ranged from 4.1 m to 7.6 m. The SPT 'N' values ranged from 8 to 56 for 0.3 m of penetration indicating a loose to very dense condition; but typically compact to dense. Some of the clay fill was identified as having a firm consistency.

The moisture content of the samples tested ranged from 3% to 30%. The results of grain size analysis conducted on four samples of this material are summarized in Table 4-2 and are illustrated on Figures C2 in Appendix C.

**Table 4-2: Gradation Results for Lower Embankment Fill**

Soil Particle	Percentage (%)	
Gravel	0 to 15	
Sand	9 to 89	
Silt	10	32 to 56
Clay		9 to 35

The results of Atterberg Limits testing completed on three samples of the fine-grained embankment fill are summarized in Table 4-3 and are illustrated on Figure C6 in Appendix C. It should be noted that one sample of embankment fill was found to be non-plastic.

**Table 4-3: Atterberg Limit Results for Lower Embankment Fill**

Parameter	Value
Liquid Limit	21 to 36
Plastic Limit	13 to 17
Plasticity Index	8 to 19

#### 4.4 Organic Silt (ML-OL to MH-OH)

A stratum of organic silt with varying amounts of sand and clay was encountered beneath the fill layers in all boreholes. The top of this layer ranged from elevation 93.7 m to 93.3 m. The thickness of this layer ranged from 2.0 m to 2.9 m. The SPT 'N' values ranged from 3 to 12 indicating a very loose to compact condition.

The moisture content of the samples tested ranged from 14% to 99%. The results of grain size analysis testing conducted on five samples of this material are summarized in Table 4-4 and are illustrated on Figure C3 in Appendix C.



**Table 4-4: Gradation Results for Organic Silt**

Soil Particle	Percentage (%)
Gravel	0
Sand	3 to 36
Silt	51 to 74
Clay	13 to 28

The results of Atterberg Limits testing completed on five samples of this material are summarized in Table 4-5 and are illustrated on Figure C7 in Appendix C. The test results indicate an organic silt ranging from low to high plasticity (ML-OL to MH-OH).

**Table 4-5: Atterberg Limit Results for Organic Silt**

Parameter	Value
Liquid Limit	25 to 54
Plastic Limit	21 to 36
Plasticity Index	4 to 19

#### **4.5 Clay (CL)**

A brown clay deposit was encountered below the organic silt strata in Borehole 402. The top of this layer was identified at elevation 91.5 m. The thickness of this layer is 0.3 m.

The moisture content of the sample tested was 31%. The results of a grain size analysis tests indicated a gravel content of 0%, sand content of 16%, a silt content of 61% and a clay content 23%. Grain size analysis results are illustrated on Figure C4 in Appendix C.

The results of Atterberg Limits testing completed on this material indicated a liquid limit of 33, a plastic limit of 18, and a plasticity index of 15, indicating a clay of low plasticity (CL). Atterberg Limits analysis results for the clay are illustrated on Figure C8 in Appendix C.

#### **4.6 Glacial Till**

A stratum of glacial till consisting predominantly of silty sand and gravel with varying amounts of clay was encountered beneath the organic silt or clay strata in all boreholes except Borehole 401. The top of this layer, where observed, ranges from elevation 91.2 m to 90.8 m. The thickness of this layer ranged from 0.2 m to 3.2 m. The SPT 'N' values ranged from 9 to greater than 100 indicating a loose to very dense condition. Although no cobbles or boulders were encountered within the glacial till, it should be noted that glacial tills inherently contain cobbles and/or boulders.

The moisture content of the sample tested ranged from 7% to 30%. The results of a grain size analysis testing conducted on three samples of this material are summarized in Table 4-6 and are illustrated on Figure C5 in Appendix C.

**Table 4-6: Gradation Results for Glacial Till**

Soil Particle	Percentage (%)	
Gravel	16 to 36	
Sand	28 to 81	
Silt	3 to 19	33
Clay		11

The results of Atterberg Limits testing completed on three samples of the fines of this material found two samples to be non-plastic and one sample to have a liquid limit of 21, a plastic limit of 15, and a plasticity index of 6, indicating silty, clayey sand (SC-SM) till in that instance. Atterberg Limit analysis results for the glacial till are illustrated on Figure C9 in Appendix C.

#### 4.7 Bedrock

The overburden materials were underlain by a grey limestone bedrock. The bedrock surface ranges from elevation 87.6 m to 91.0 m in the boreholes where rock was cored. Photographs of the bedrock core are provided in Appendix B. Table 4-7 below summarizes the depths and elevations of the bedrock surface.

**Table 4-7 Top of Bedrock Elevation**

Location	Borehole	Ground Surface Elevation (m)	Depth Below Existing Grade (m)	Top of Bedrock Elevation (m)
West Abutment	401	106.0	14.5	91.5**
	402	106.0	15.0	91.0*
East Abutment	403	103.3	15.7	87.6*
	404	103.4	14.9	88.4**

\* Bedrock surface proven by coring

\*\* Inferred Bedrock by SPT/auger refusal

Boreholes 402 and 403 were advanced into the bedrock by coring with HQ-size coring equipment. The bedrock within the top 0.5 m in Borehole 402 was moderately weathered and could be penetrated with the drill rig augers.

The bedrock encountered in Borehole 403 and below the weathered bedrock in Borehole 402 had a total core recovery ranging from 94% to 100%, the solid core recovery ranging from 83% and 100% and the Rock Quality Designation (RQD) ranging from 42% to 91%. Based on the RQD value the bedrock is classified as poor to excellent quality, but typically fair.

#### 4.8 Groundwater

The groundwater level was measured in the piezometer installed in Borehole 403 on May 31, 2016, at a depth of 8.4 m below the top of the existing asphalt; corresponding to an elevation of 94.9 m.

This observation is considered a short-term reading and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

#### 4.9 Analytical Test Results

Two samples of the fill encountered at the site were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The analysis results are summarized in the Table 4-8. A copy of the test results is provided in Appendix C.

**Table 4-8: Results of Chemical Analysis**

<b>Borehole</b>	<b>Sample</b>	<b>Depth (m)</b>	<b>pH</b>	<b>Resistivity (Ohm-cm)</b>	<b>Conductivity (µS/cm)</b>	<b>Chloride (µg/g)</b>	<b>Sulphate (µg/g)</b>
401	SS4	2.6	8.4	1280	782	365	21
403	SS3	1.8	8.3	826	1210	758	76

## 5 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the borehole locations and ground surface elevations relative to a benchmark provided by WSP Canada. Terex Drilling Solutions of Concord, Ontario supplied and operated the drill rigs to carry out the drilling, sampling, and in-situ testing. All work was performed within short duration TL-29 right lane closures in conformance with the requirements set in Ontario Book 7; all signs, barrels and traffic control personnel were provided by Direct Traffic Management of Mississauga, Ontario. The drilling, and sampling operations in the field were supervised on a full-time basis by Mr. Justin Gray and Christopher Murray of Thurber. Laboratory testing was carried out by Thurber in its MTO-approved laboratory in Ottawa.

Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.

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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 GENERAL**

This section of the report presents an interpretation of the factual data obtained from the preliminary foundation investigation conducted by Thurber for the widening of the Highway 401 overpass structure located at the Ganaraska River in Port Hope, Ontario. Preliminary Geotechnical recommendations are provided to assist the design team in designing a suitable foundation for the proposed bridge widening.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The following sections address preliminary geotechnical recommendations for the widening of the existing bridge structure. The discussions and recommendations presented in this report are based on the information provided by WSP and on the factual data obtained during the course of this investigation.

**6.1 Existing Foundations**

The existing 3 span bridge was constructed in 1959. Based on the historical contract documents, the abutments were designed to be supported by steel BP12x53 piles driven into bedrock. The piles supporting the abutments are shown to include a back row of vertical piles and two front rows of piles battered at 1H:4V with approximate pile lengths of 9 m at the east abutment and 10.5 m at the west abutment. The design load for the piles was not noted on the contract drawings. The piers are supported on 3.05 m diameter concrete caissons socketed approximately 0.3 m into bedrock. The abutment foreslopes are identified as being sloped at 2H:1V and covered with hand-laid rip-rap.

The GEOCRES report for this site, dated July 12<sup>th</sup>, 1957, summarizing the findings of the investigation for the bridge structure and approach embankments indicated that the organic silt stratum on both sides of the bridge would undergo large settlement and fail under the superimposed load of the approach embankments (in the order of 35 feet (10.7 m) in height. This

was predicted to occur during or shortly after construction; keying the foundations into the bedrock to resist the lateral thrust from movement of the embankment or removing the poor subsoil were presented as options. The results of the borehole drilling indicate that they did not remove the organic silt (see Boreholes).

## 6.2 Proposed Structure

It is understood that future plans include widening of this section of Highway 401 from 6 to 8 lanes. Based on information provided by WSP, it is understood that the proposed work includes the widening of the existing bridge and approach embankments to either both the north and south or just to the north. The profile of Highway 401 is not expected to change.

## 6.3 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-14.

In accordance with CHBDC CSA S6-14, the analysis and design of structures takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation, Ontario (MTO).

It is understood that MTO has designated this structure as follows:

**Table 6-1: Bridge Structure Classification**

Criteria	Classification	CHBDC Section
Importance Category	Major Route Bridge	4.4.2
Consequence Classification	Typical Consequence	6.5.1

Accordingly, a consequence factor ( $\Psi$ ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances. If the consequence classification changes, the geotechnical assessment will need to be reviewed and revised.

The frost penetration depth at this site is 1.4 m as per OPSD 3090.101.

## 6.4 Preliminary Geotechnical Assessment

Based on the results of the field and laboratory investigation, the review of historical information, and the information provided by WSP with regards to the proposed project requirements, the geotechnical foundation design considerations include:

- The existing fill and native soils will not offer bearing resistance to support bridge abutments on shallow foundations; deep foundations will be required to provide the required geotechnical resistance and to ensure performance compatible with the existing structure foundations.

- Both approaches include a high fill (approximately 10 m to 12 m) constructed above organic silt. Historical documents indicate a concern regarding settlement and stability of the approach embankments during construction. The implications of any new fill on both settlement, embankment stability and additional loading on the existing structure will need to be assessed; and
- The configuration or the widening of Highway 401 has not yet been confirmed but may include widening to both the north and south sides or widening only to the north side.

## **7 SEISMIC CONSIDERATIONS**

### **7.1 Spectral and Peak Acceleration Hazard Values**

The seismic hazard data for the CHBDC is based on the fifth generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values ( $S_a(T)$ ) for the reference ground condition (Site Class C) for a range of periods ( $T$ ) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA), which is 0.112g at this site.

### **7.2 CHBDC Seismic Site Classification**

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy.

Based on the soil and bedrock conditions encountered below the anticipated bridge foundation elevation, the site is classified as a Seismic Site Class D in accordance with Table 4.1 of the CHBDC.

### **7.3 Seismic Liquefaction**

The soils beneath the anticipated founding elevation and water table consist of fill material, loose organic silt and compact to very dense glacial till deposits. Based on the PGA value of 0.112g, a de-aggregated earthquake magnitude of 6.15, the subsurface conditions encountered at the drilled locations at this site and using the Seed & Idriss Simplified Method for liquefaction assessment, the non-cohesive foundation soils are considered not susceptible to liquefaction during a seismic event.

The thin native clay deposit encountered in Borehole 402 at this site is classified as moderately susceptible to cyclic mobility based on the criteria presented by Bray et al (2004) and the measured moisture content and Atterberg limits.

## 8 FOUNDATION DESIGN ALTERNATIVES

The results of the field and laboratory investigation and historical data indicate that the site soil stratigraphy consists of fill materials overlying organic silt and clay overlying glacial till, underlain by limestone bedrock.

Approximate key elevations are as follows:

- |  |  |
|--|--|
| • Existing top of asphalt surface at the abutments | 106.0 m West side<br>103.3 m East side |
| • Top of glacial till                              | 90.8 to 91.2 m                         |
| • Top of bedrock                                   | 91.0 m West side<br>87.6 m East side   |

The following foundation alternatives were considered for the bridge widening.:

- Spread footings
- Caissons (drilled shaft piles)
- Steel Piles (H-piles and pipe piles)
- Drilled-in Pipe Piles (down-the-hole hammer)
- Micropiles

These foundation alternatives are presented in the following sections and evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks and consequences. The evaluation is summarized in the table provided in Appendix E. A preferred foundation for the widening alternative from a geotechnical engineering perspective is recommended.

### 8.1 Shallow Foundations

The existing fill, clay and organic silt will not offer sufficient bearing resistance to support bridge abutments, piers or retaining walls on shallow foundations. In addition, for widening of the existing bridge abutments and piers, deep foundations are recommended for compatibility with the existing abutment and pier foundations.

Shallow foundations bearing on undisturbed glacial till might be feasible if retaining walls are required near the toe of the embankment slopes.

### 8.2 Caissons

Caisson foundations particularly when they are socked into bedrock, can offer high axial and lateral geotechnical resistance.

Caissons are the recommended foundation type for widening of the existing piers which are supported on caissons.



Caissons are considered feasible at this site and may be the preferred foundation option at some foundation locations, however, restrictions or special measures would be required for caissons that require rock sockets if they are located too close to existing deep foundations end-bearing on the bedrock. These measures may include extending a steel casing down into the socket, limiting the number of caissons that can be left open at any given time, and monitoring of the existing structure.

### **8.3 Driven Steel Piles**

Steel piles driven to refusal on or within the bedrock are feasible for widening of the bridge abutments and would be compatible with the driven steel piles supporting the existing abutments.

The use of either steel H-piles or pipe piles could be considered at this site, however, pipe piles fitted with a driving shoe to help seat the piles into the rock cause greater soil displacement than steel piles increasing the potential for disturbance of adjacent piles. The use of driven steel H-pile sections is recommended for the abutment widenings from a geotechnical perspective.

### **8.4 Drilled-in Pipe Piles**

Drilled-in pipe piles are feasible from a foundations perspective at this site and would offer moderate to high geotechnical axial and lateral resistance. Drilled in pipe piles may be preferable over driven steel piles if large lateral resistance is required.

### **8.5 Micropiles**

Micropiles would provide low to moderate axial compression loads. A larger number is required to achieve the same overall resistance as driven steel piles or caissons. In addition, they provide low lateral resistance for an individual micro-pile. Lateral resistance is often achieved by installing groups of micro-piles at different inclinations. The cost for micropiles is typically higher than for driven steel piles or caissons in order to achieve similar geotechnical resistance. Micropiles can offer an advantage where installation is required in areas with limited vertical clearance or for applications that also require significant uplift resistance.

### **8.6 Preliminary Recommended Foundation**

Based on the evaluation of foundation alternatives presented above, the recommended foundation approach from a geotechnical perspective is to support the widened abutments on steel H-piles end-bearing on or within the bedrock and to support the piers on caissons socketed into the bedrock.

Preliminary foundation recommendations and considerations are presented in the following sections.

## 9 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

### 9.1 Deep Foundations – Caissons for Piers

The widened bridge piers may be founded on concrete caissons set into the bedrock. The geotechnical resistance will depend on both the caisson diameter and the socket length in sound bedrock. For design purposes, the upper 0.5 m of bedrock should be assumed to be weathered or highly fractured and should not be included in the design socket length. The factored geotechnical resistances provided below may be used for preliminary design are provided in Table 9-1 below:

**Table 9-1: Caisson Axial Capacity**

Caisson Diameter (m)	Socket Length in Sound Rock (m)	Factored Geotechnical Resistance (Axial Compression (kN))	
		ULS	SLS
		Static ( $\phi_{gu}=0.4$ )	Static ( $\phi_{gs}=0.8$ )
1.2	3.0	4,500	N/A <sup>(1)</sup>
1.5	3.0	6,600	N/A <sup>(1)</sup>
3.0	3.0	22,000	N/A <sup>(1)</sup>

#### NOTES:

1. The SLS condition will not govern for piles end-bearing in or on the bedrock.

The factored geotechnical resistances include the following factors:

- Consequence factor ( $\Psi$ ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
  - $\phi_{gu} = 0.4$  (static analysis; typical degree of understanding)
  - $\phi_{gs} = 0.8$  (static analysis; typical degree of understanding)

Caissons must be installed in accordance with OPSS.PROV 903.

### 9.2 Deep Foundations – Driven Steel Piles

The widened portion of the abutments may be founded on steel HP 310x110 piles end-bearing on or in the bedrock.

The estimated pile tip elevations based on piles reaching refusal at the bedrock surface are summarized in Table 9-2.

**Table 9-2: Estimated Pile Tip Elevations**

<b>Foundation Element</b>	<b>Approximate Underside of Pile Cap Elevation (m)</b>	<b>Estimated Pile Tip Elevation (top of bedrock) (m)</b>	<b>Estimated Pile Length (m)</b>
East Abutment	99.4	87.6	11.8
West Abutment	101.6	91.0	10.6

The design parameters for axial resistance of Grade 350W HP 310x110 steel piles driven to refusal on or in the limestone bedrock can be taken as:

- Factored vertical geotechnical resistances at ULS 2,000 kN
- The SLS condition will not govern for piles founded in or on the bedrock

The factored geotechnical resistances provided include the following factors:

- Consequence factor ( $\Psi$ ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
  - $\phi_{gu} = 0.4$ , ULS (static analysis; typical degree of understanding)

Driven piles must be installed in accordance with OPSS.PROV 903. The potential for conflict with the existing steel piles must be checked.

As the piles are to be driven to bedrock the pile tips of the new piles at the site should be protected from damage during driving with pile tip protection from an approved manufacturer such as Titus Steel (standard H-Point) or approved equivalent.

The appropriate pile driving note is "Piles to be driven to bedrock."

### **9.3 Deep Foundations – Lateral Resistance**

The lateral resistance of the soil acting against the deep foundations (steel piles or caissons) is dependent on several factors including the effective overburden stress and is therefore directly dependent on the proposed geometry of the proposed bridge and retaining walls. Once the geometry has been determined, the soil response to lateral loading from the contemplated deep foundations in the form of p-y curves should be evaluated.

Depending on the pile spacing and the direction of the load, the lateral pile response could be influenced by group interaction effects. Accordingly, we recommend applying p-multipliers (i.e. soil spring reduction factors) following the procedure described in Section C6.11.3 of the CAN-CSA-S6-14 Bridge Code Commentary.

### **9.4 Deep Foundations – Downdrag Loads**

A clay fill layer was identified within the embankment fill at the east approach and compressible organic silt was found below the embankment fill at both abutment locations. Settlement of these compressible soils is expected to occur if additional embankment fill is placed to widen the embankment. This settlement would result in downdrag loads on deep foundations. The

magnitude of the downdrag loads will need to be confirmed once the extent of organic silt and geometry of the proposed widening has been determined.

## 9.5 Frost Protection

The frost penetration depth at this site is 1.4 m as per OPSD 3090.101. Accordingly, a minimum of 1.4 m of earth cover, or equivalent insulation, must be provided above the base of the pile caps to serve as frost protection.

## 9.6 Lateral Earth Pressures

Structure backfill should be in accordance with OPSS 902. All backfill material should consist of Granular A, or Granular B Type II meeting the specifications of OPSS.PROV 1010.

Compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501.

The lateral earth pressure parameters provided in Table 9-3 and 9-4 in the sections below are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in the design.

### 9.6.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil (kN/m<sup>3</sup>), to be adjusted for groundwater depth

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table 9-3.

**Table 9-3: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	Existing Embankment Fill
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	22.0	20.5
Angle of Internal Friction, $\phi$	35°	30°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.50
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided. A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Section 6.12.3 of the CHBDC.

## 9.6.2 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section C4.6.5 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} F(PGA) \cdot PGA$  for structures that allow for 25 mm to 50 mm of movement, and
- $k_h = F(PGA) \cdot PGA$  for non-yielding walls

The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002 for a yielding structure with respect to the assessment of seismically induced lateral earth pressures.

The recommended seismic lateral earth pressure parameters for use in the design that are provided in Table 9-4 assume the following:

- Horizontal back-slope behind the wall
- Seismic Site Class of E, and a PGA with a 2% probability of exceedance in 50 years of 0.112g; as outlined in Section 7.0

**Table 9-4: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	Existing Embankment Fill
Soil Unit Weight, $kN/m^3$ , $\gamma$	22.0	20.5
Angle of Internal Friction, $\phi$	35°	30°
<b>Non-Yielding Wall</b>		
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.39	0.47
<b>Yielding Wall</b>		
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.33	0.39

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K\gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth, d (kPa)

d = depth below the top of the wall (m)

K = static earth pressure coefficient

( $K_a$  for yielding walls,  $K_o$  for non-yielding walls)

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

H = total height of the wall (m)

## 10 EMBANKMENT SETTLEMENT AND STABILITY

Based on the limited preliminary borehole data available, an analysis was carried out to assess the settlement resulting from the widening of approach embankments at this site. For this analysis, all widening was assumed to happen to the north. If the widening is carried out on both sides of the highway then slightly lower settlements would be expected. The results indicated that a settlement of 30 mm can be expected at the east approach embankment and 20 mm can be expected at the west approach embankment. The settlement is expected to be at a maximum value at the new edge of embankment and linearly reduces to approximately 5 mm at the existing edge of pavement for both approaches. The settlement should be relatively quick and is expected to take place within a month of fill placement. Further investigation should be carried out during the detailed design phase to ensure that the native soils encountered within the preliminary boreholes extends beyond the existing embankments as assumed within the settlement models.

A preliminary global stability assessment was carried out on both the existing approach embankments and the widened embankments. The existing east and west foreslopes were found to be stable under both static and seismic conditions. The embankment side slopes of both the existing and widened embankments were found to be marginally stable with a 1.4 factor of safety under static conditions and will need to be further assessed during the detailed design phase of this project. It is noted that for the purposes of this analysis all widening was assumed to happen to the north with traditional 2H:1V side slopes with a 2 m bench included at mid-height of the widened embankment as both approach embankments are in excess of 8 m.

The potential impact of additional embankment loads and settlement on the existing structure during construction will also need to be assessed.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. Normal slope vegetation should be established as soon as possible after completion of the embankment fills to control surficial erosion in general accordance with OPSS 804.

## 11 CONSTRUCTION CONSIDERATIONS

### 11.1 Excavations and Temporary Protection Systems

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills at the site should

be classified as Type 3 in accordance with OHSA. Excavations within the existing embankment fill should be supported or sloped no steeper than 1H:1V up from the base of the excavation. If excavations extend down into the organic silt, the excavation should be carried out based on Type 4 soil with excavation slopes no steeper than 3H:1V up from the base of the excavation.

It is anticipated that temporary protection systems will be required at some locations to either limit the extent of excavation or to protect the existing highway embankment. Temporary protection systems should be provided in accordance with OPSS.PROV 539. Where the protection system is supporting an existing roadway or embankment, Performance Level 2 is typically appropriate; Performance Level 1a or 1b may be required if the excavation is in close proximity to the existing structure foundations. The required performance level will need to be reviewed during detailed design once the geometry of the proposed bridge and retaining walls has been determined.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor.

## **11.2 Dewatering**

Subgrade preparation and construction of foundations must be carried out in the dry. All excavations for foundation construction must be dewatered prior to the placement of concrete, as per OPSS 902 and NSSP FOUN0003.

The Contractor must be prepared to control the groundwater and surface water flow at the site to permit construction in a dry and stable excavation. Water from either surface flow and/or groundwater must be diverted away from the excavation at all times. Groundwater perched within the embankment fill and surface runoff will tend to seep into and accumulate in open excavations.

Excavations for widening of the abutments are expected to be within the existing embankment fills, above the groundwater level. Dewatering at these locations is expected to be primarily related to removal of surface run-off. Therefore, it is anticipated that conventional sump and pump techniques should be sufficient at these locations. The piers are located along the banks of the Ganaraska River and excavations for caissons will extend below the groundwater and river water levels. Dewatering requirements may be substantial, including cofferdams, if caissons are to be constructed in the dry. Concrete placement using tremie techniques may be required. Dewatering requirements will need to be reviewed during detailed design. Designer fill-ins for FOUN0003 should be determined at that time.

Dewatering design and decisions regarding dewatering must be carried out by the Contractor.

## **11.3 Erosion Protection**

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site as per OPSS 805.



## 11.4 Construction Concerns

Potential construction concerns include, but are not necessarily limited to, the following:

- The potential for conflict with existing deep foundations during installation of new deep foundations
- The stability of the high fills during construction.
- The potential for movements of the existing structure and/or highway embankment during excavation for new foundations. Appropriate temporary protection systems must be provided.
- Damage to piles tips during driving may occur as piles are to be driven to bedrock, tips should be protected from damage during driving.
- Dewatering of the caisson excavations for widening of the piers may be difficult.
- Confirmation that the granular backfill is adequately placed and compacted to specifications.
- Vibration monitoring during pile or caisson installation for the widening may be required.

The successful performance of the embankments will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations by qualified geotechnical personnel will be required during construction to confirm that the foundation recommendations are correctly implemented and material specifications are met.

## 11.5 Recommendations for Further Work in Detailed Design

Additional foundation investigation and design will be required during the detail design stage to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report. The additional work should include:

- Boreholes in the area of widening for the piers and abutments and within the footprint of proposed embankment widenings.
- Further assessment of the strength and compressibility of the organic silt beneath proposed embankment widenings.
- Further assessment of the groundwater level and permeability of the site soils and bedrock to refine dewatering estimates.
- Further assessment of the potential for liquefaction or cyclic mobility of the fine-grained soils beneath the embankments.
- Additional assessment of the potential for sulphate attack on buried concrete and the corrosiveness of the subsurface environment is required for the area along the Ganaraska River (i.e. near the piers).
- A monitoring plan for the existing structure may need to be developed depending on the potential for interaction between the proposed work and the existing structure.



## 12 CLOSURE

Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Christopher Murray, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.

*P. Carnaffan*  
for

Christopher Murray, M.A.Sc., P.Eng.  
Geotechnical Engineer



Paul Carnaffan, P.Eng.  
Principal  
Senior Geotechnical Engineer

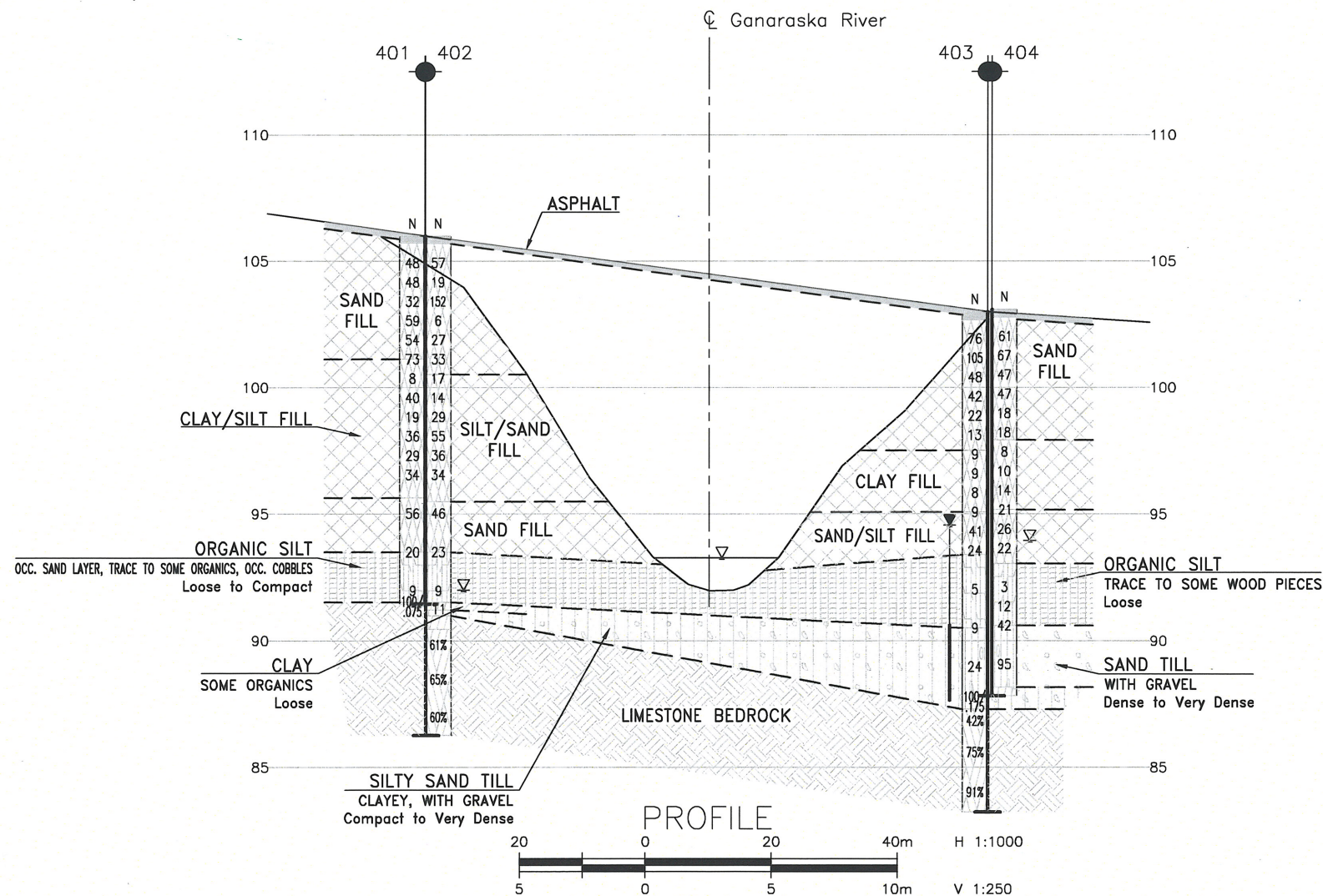
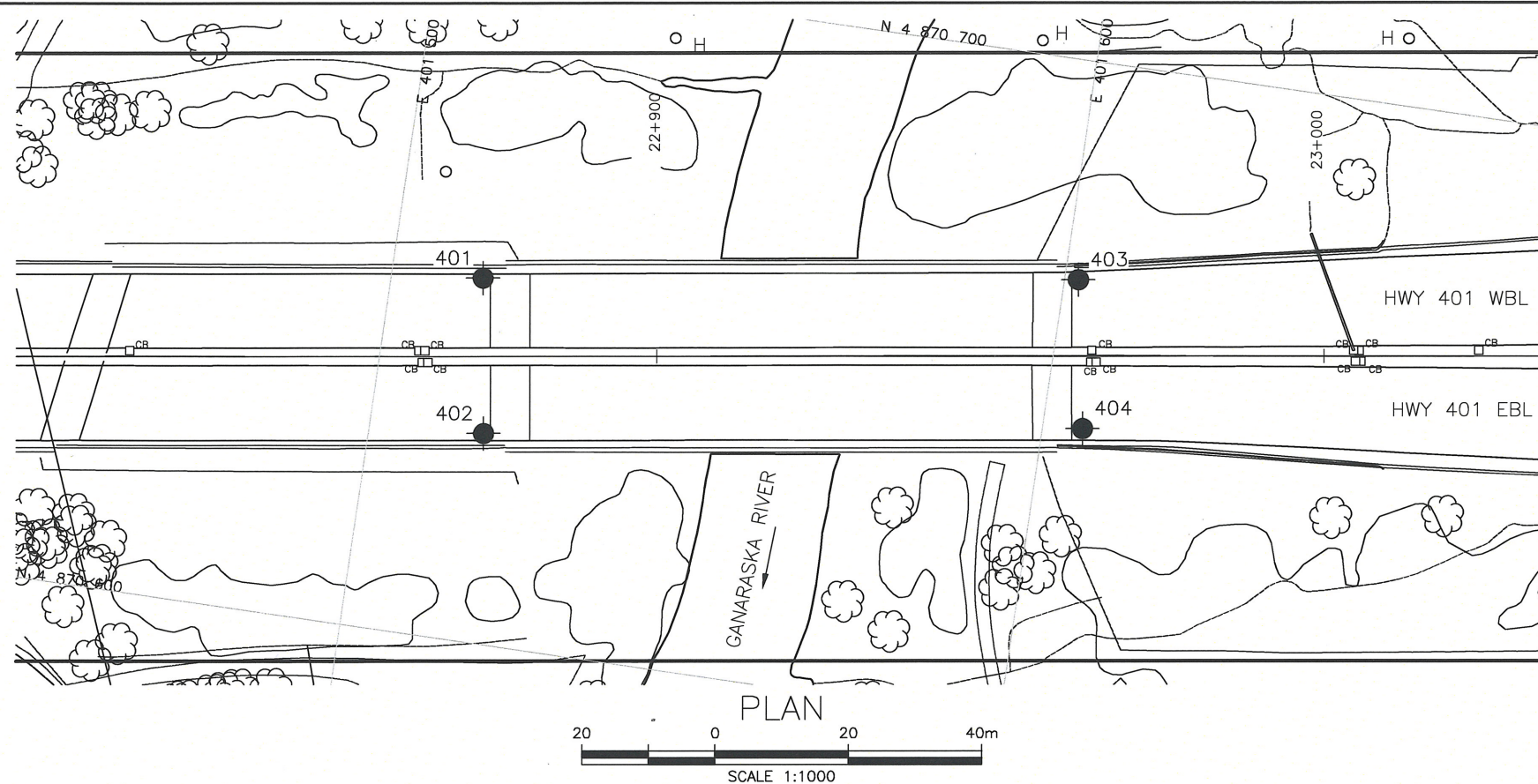


Dr. P.K. Chatterji, P.Eng.  
Review Principal  
Senior Geotechnical Engineer

## **Appendix A**

### **Borehole Location Plan and Stratigraphic Drawings**





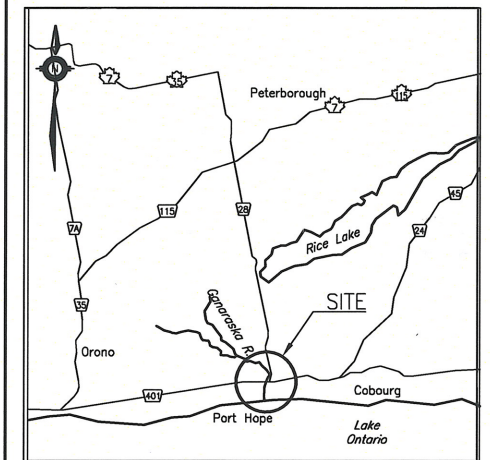
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 4078-14-00

HIGHWAY 401  
GANARASKA RIVER  
BRIDGE WIDENING  
BOREHOLE LOCATIONS AND SOIL STRATA

**wsp**

**THURBER ENGINEERING LTD.**



### LEGEND

●	Borehole
⊙	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60' Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
⊥	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
401	106.0	4 870 654.5	401 513.7
402	106.0	4 870 631.4	401 517.2
403	103.3	4 870 667.1	401 602.1
404	103.4	4 870 645.1	401 606.0

### NOTES

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 10.

GEOCREs No. 30M16-62

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	JG	CHK -	CODE
DRAWN	MFA	CHK JG	SITE 21-231
			STRUCT
			DWG 1
			DATE APR 2018





## **Appendix B**

### **Record of Borehole Sheets**



## **SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS**

### **TERMINOLOGY DESCRIBING COMMON SOIL GENESIS**

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

### **TERMINOLOGY DESCRIBING SOIL STRUCTURE:**

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

### **RECOVERY:**

For soil samples, the recovery is recorded as the length of the soil sample recovered.

### **N-VALUE:**

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

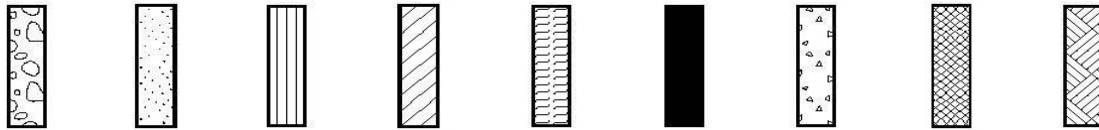
### **DYNAMIC CONE PENETRATION TEST (DCPT):**

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.



### STRATA PLOT:

Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders  
Cobbles  
Gravel      Sand      Silt      Clay      Organics      Asphalt      Concrete      Fill      Bedrock

### TEXTURING CLASSIFICATION OF SOILS

Classification	Particle Size
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

### TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Undrained Shear Strength (kPa)
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

### SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

### TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	SPT "N" Value
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

### MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	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	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, 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.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note -  $W_L$  = Liquid Limit



## EXPLANATION OF ROCK LOGGING TERMS

### ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
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 structures are preserved.

### 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 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

### DISCONTINUITY SPACING

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

### STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1



# RECORD OF BOREHOLE No 401

1 OF 2

METRIC

GWP# 4078-14-00 LOCATION Site 21-231, MTM Zone 10: N 4 870 654.5 E 401 513.7 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY JAG  
 DATUM Geodetic DATE 2016.05.31 - 2016.05.31 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
106.0														
0.0														
105.7	280 mm ASPHALT													
0.3	Silty sand with gravel Dense to very dense Brown FILL		1	GS										
			2	SS	48		105							
			3	SS	48		104							
			4	SS	32									
	-gravel and cobbles		5	SS	59		103							
			6	SS	54		102							
			7	SS	73		101							
101.1	-gravel and cobbles		8	SS	8		100							
4.9	Sandy silt to sandy clay Loose to dense Brown FILL		9	SS	40		99							
			10	SS	19		98							
			11	SS	36		97							
			12	SS	29									
			13	SS	34									

Continued Next Page




+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 401

2 OF 2

METRIC

GWP# 4078-14-00 LOCATION Site 21-231, MTM Zone 10: N 4 870 654.5 E 401 513.7 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY JAG  
 DATUM Geodetic DATE 2016.05.31 - 2016.05.31 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
Continued From Previous Page							WATER CONTENT (%)							
							20 40 60							
95.6	Sand with silt Very dense Brown FILL		14	SS	56	95								
10.4														
93.5	Sandy Organic SILT (ML-OL) Compact to loose Dark brown - some wood fragments		15	SS	20	94								
12.5														
91.5			16	SS	9	93								
91.5	Weathered Limestone Bedrock		17	SS	100	92								
14.6														
14.6	End of Borehole on Inferred Bedrock				75mm									






ONTMT4S GANARASKA RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/4/18

# RECORD OF BOREHOLE No 402

1 OF 3

METRIC

GWP# 4078-14-00 LOCATION Site 21-231, MTM Zone 10: N 4 870 631.4 E 401 517.2 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY JAG  
 DATUM Geodetic DATE 2016.01.06 - 2016.01.06 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa													
106.0								20	40	60	80	100									
0.0	300 mm ASPHALT																				
105.7																					
0.3	Silty sand with gravel, occasional cobbles Loose to very dense Brown FILL		1	GS																	
			2	SS	57																
			3	SS	19																
			4	SS	12																
			5	SS	6																
100.5	- cobbles																				
			6	SS	27																
			7	SS	33																
5.5	Silty sand with gravel, occasional cobbles Compact to very dense Brown to greyish-brown FILL		8	SS	17																
9	SS		14																		
10	SS		29																		
11	SS		55																		
100.5																					
			12	SS	36																
5.5																					
			13	SS	34																

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 402

2 OF 3

METRIC

GWP# 4078-14-00 LOCATION Site 21-231, MTM Zone 10: N 4 870 631.4 E 401 517.2 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY JAG  
 DATUM Geodetic DATE 2016.01.06 - 2016.01.06 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100							
								20 40 60 80 100							
Continued From Previous Page							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				PLASTIC LIMIT W <sub>P</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%)				
95.5															
10.5	Silty sand Dense Brown to greyish-brown FILL		14	SS	46		95								
							94								
93.5			15	SS	23										
12.5	Organic SILT (MH-OH) Loose to compact Greyish-brown to dark brown						93								
			16	SS	9		92								
91.5															
14.5	CLAY (CL), some organics														
91.2	Firm		17	SS	11										
14.8	Dark brown						91								
15.0	Silty, Clayey SAND (SC-SM) with gravel TILL														
90.5	Compact														
15.5	Grey														
	Moderately weathered BEDROCK - augered to 15.5 m		1	HQ			90								
	BEDROCK Limestone Slightly weathered Thinly to moderately bedded Fair Quality Grey		2	HQ			89								
			3	HQ			88								
86.3							87								
19.7	End of Borehole														

ONTMT4S GANARASKA RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/4/18

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 402

3 OF 3

METRIC

GWP# 4078-14-00 LOCATION Site 21-231, MTM Zone 10: N 4 870 631.4 E 401 517.2 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY JAG  
 DATUM Geodetic DATE 2016.01.06 - 2016.01.06 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
	Continued From Previous Page																
	Groundwater level was measured in open borehole at 14.0m BGS (elev. 92.0 m) on 2016/06/01																






ONTMT4S GANARASKA RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/4/18

# RECORD OF BOREHOLE No 403

1 OF 3

METRIC

GWP# 4078-14-00 LOCATION Site 21-231, MTM Zone 10: N 4 870 667.1 E 401 602.1 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY JAG  
 DATUM Geodetic DATE 2016.05.30 - 2016.05.30 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
103.3								20 40 60 80 100							
0.0	350 mm ASPHALT														
103.0															
0.3	Silty sand with gravel Compact to very dense Brown FILL		1	GS			103								
			2	SS	76		102								
			3	SS	105										
			4	SS	48		101								
			5	SS	42		100								
			6	SS	22		99								
			7	SS	13		98								
97.8			8	SS	9										
5.5	Clay, trace sand Firm Brown to greyish-brown FILL		9	SS	9		97								
			10	SS	8	96									
			11	SS	9	95									
95.4	Silty sand some gravel Compact to dense Brown to greyish-brown FILL		12	SS	41		94								
			13	SS	24										
93.7															
9.6	Organic SILT (MH-OH) occasional wood pieces														

ONTMT4S GANARASKA RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/4/18

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

## METRIC

[illegible]



# RECORD OF BOREHOLE No 404

1 OF 2

METRIC

GWP# 4078-14-00 LOCATION Site 21-231, MTM Zone 10: N 4 870 645.1 E 401 606.0 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY JAG  
 DATUM Geodetic DATE 2016.01.06 - 2016.01.06 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								WATER CONTENT (%)					
103.4													
0.0	240 mm ASPHALT												
103.0													
0.3	Sand with silt and gravel Compact to dense Brown FILL		1	GS									
			2	SS	61								
			3	SS	67								
			4	SS	47								
			5	SS	47								
			6	SS	18								
			7	SS	18								
98.2													
5.2	Clay Firm Brown FILL		8	SS	8								
			9	SS	10								
			10	SS	14								
95.4			11	SS	21								
7.9	Sand with silt Compact Greyish-brown FILL												
			12	SS	26								
			13	SS	22								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 404

2 OF 2

METRIC

GWP# 4078-14-00 LOCATION Site 21-231, MTM Zone 10: N 4 870 645.1 E 401 606.0 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY JAG  
 DATUM Geodetic DATE 2016.01.06 - 2016.01.06 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W <sub>P</sub>	W	W <sub>L</sub>		
	Continued From Previous Page							20   40   60   80   100							
93.3 10.1	<b>Organic SILT (MH-OH)</b> Very loose to compact Dark brown						93								0   3   74   23
			14	SS	3		92								
			15	SS	12										
90.9 12.5	<b>SAND (SP)</b> with gravel <b>TILL</b> Dense to very dense Brown to grey		16	SS	42		91								16   81   3 (SI+CL)
			17	SS	95		90								
88.4 14.9 88.1	<b>Weathered Limestone BEDROCK</b> - augered to 15.3 m		18	SS	100/ 25mm		89								
15.3	End of Borehole on inferred bedrock Groundwater level was measured in open borehole at 9.1 m BGS (elev. 94.3 m) on 2016/06/01														

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
15  
10  
5  
0  
(%) STRAIN AT FAILURE

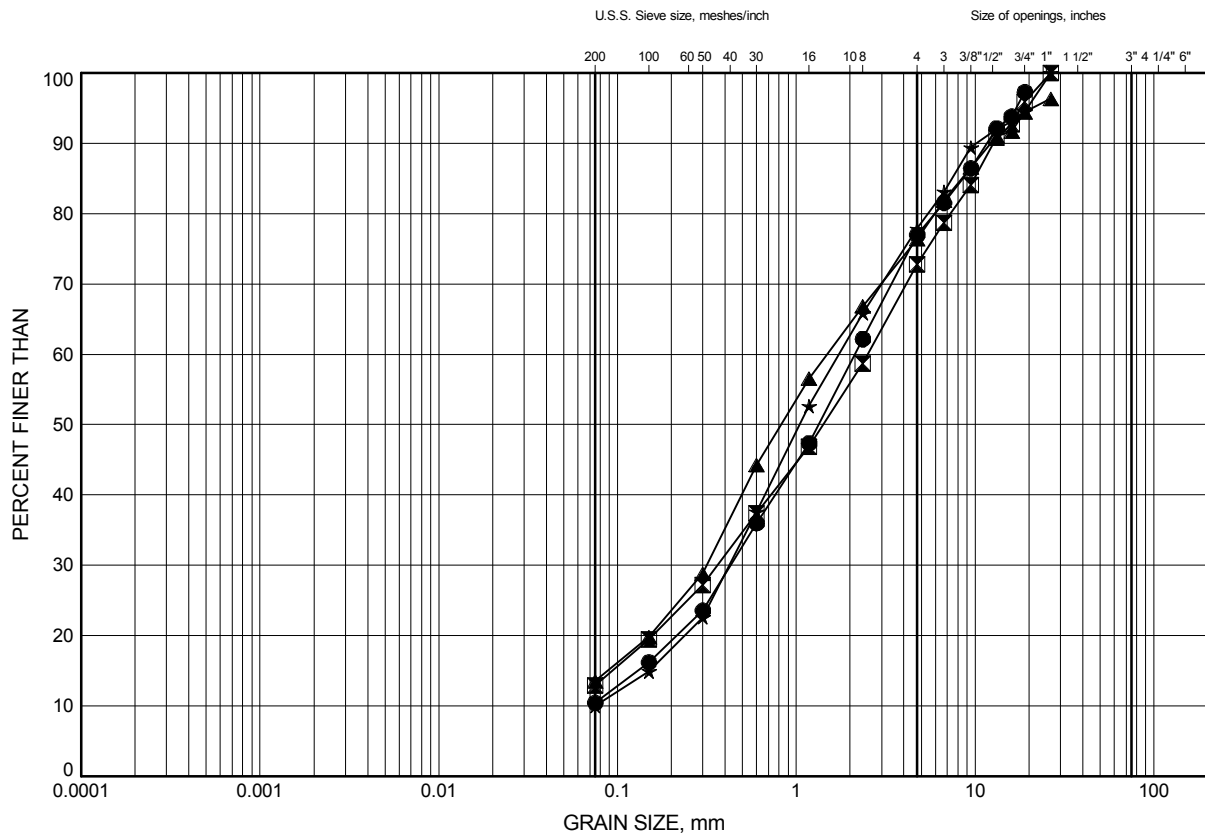
**Appendix C**  
**Laboratory Testing**

## **Appendix C.1**

### **Grain Size Analysis Figures**

## GRAIN SIZE DISTRIBUTION

## Upper Embankment Fill: Silty Sand with Gravel



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	401	1.07	104.93
⊠	402	4.11	101.90
▲	403	2.59	100.72
★	404	3.35	100.01

Date April 2018

GWP# 4078-14-00

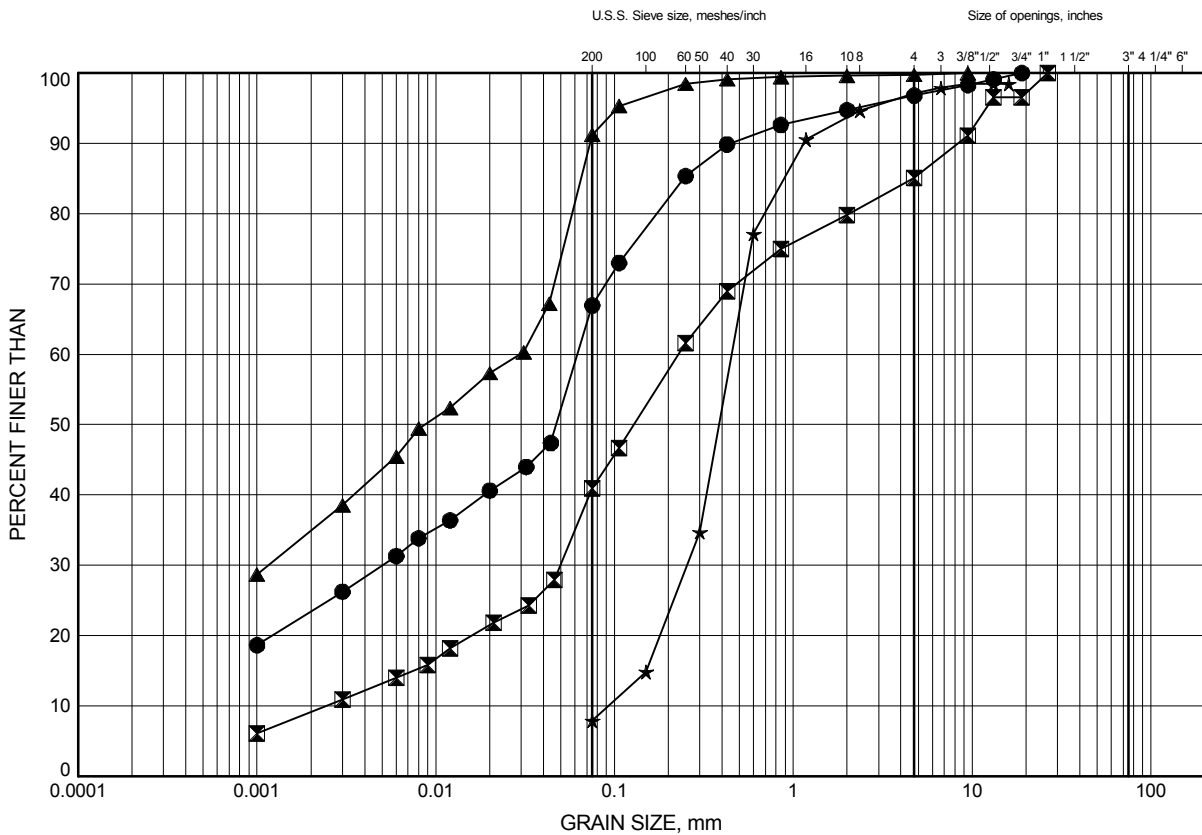


Prep'd CM

Chkd. PC

## GRAIN SIZE DISTRIBUTION

## Lower Embankment Fill: Variable



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	401	7.16	98.84
⊠	402	8.69	97.32
▲	403	6.40	96.91
★	404	9.45	93.92

Date April 2018

GWP# 4078-14-00

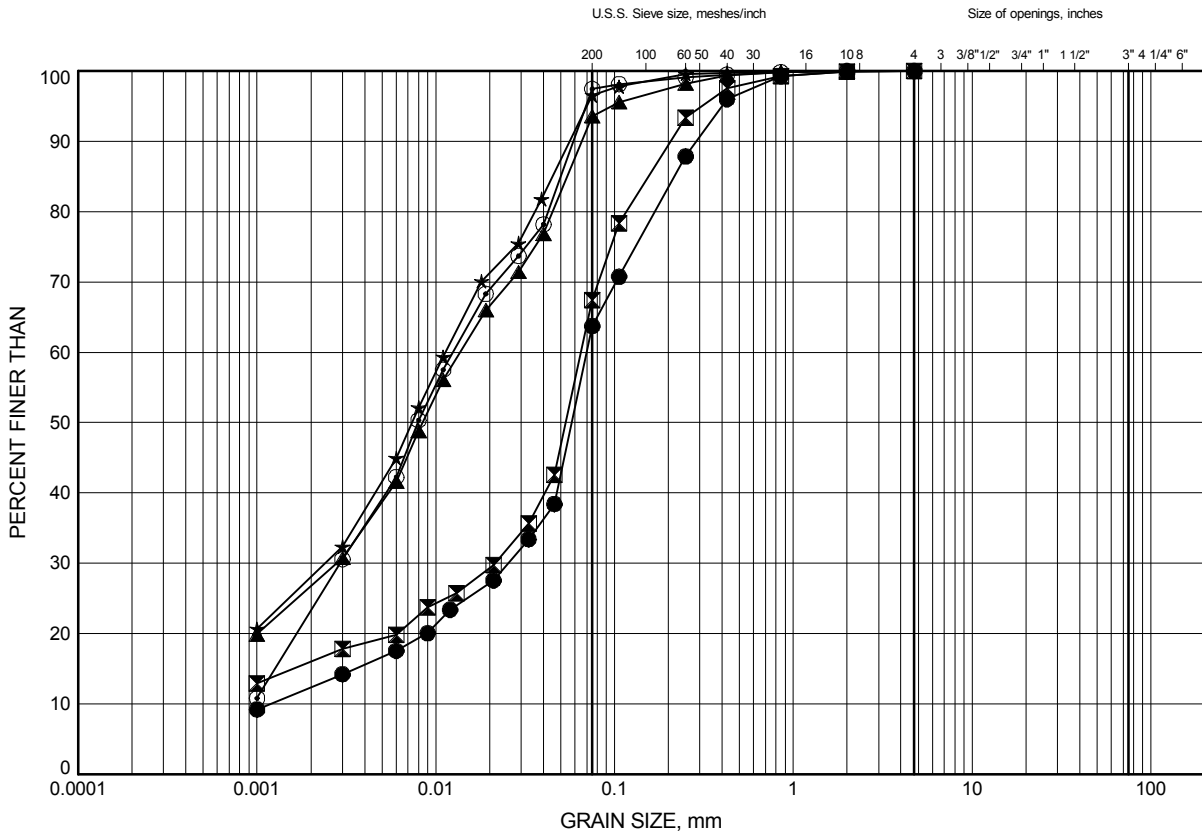


Prep'd CM

Chkd. PC

## GRAIN SIZE DISTRIBUTION

## Organic Silt (ML-OL to MH-OH)



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	401	12.65	93.35
⊠	401	14.02	91.98
▲	402	14.02	91.99
★	403	10.97	92.34
⊙	404	10.97	92.39

Date April 2018

GWP# 4078-14-00

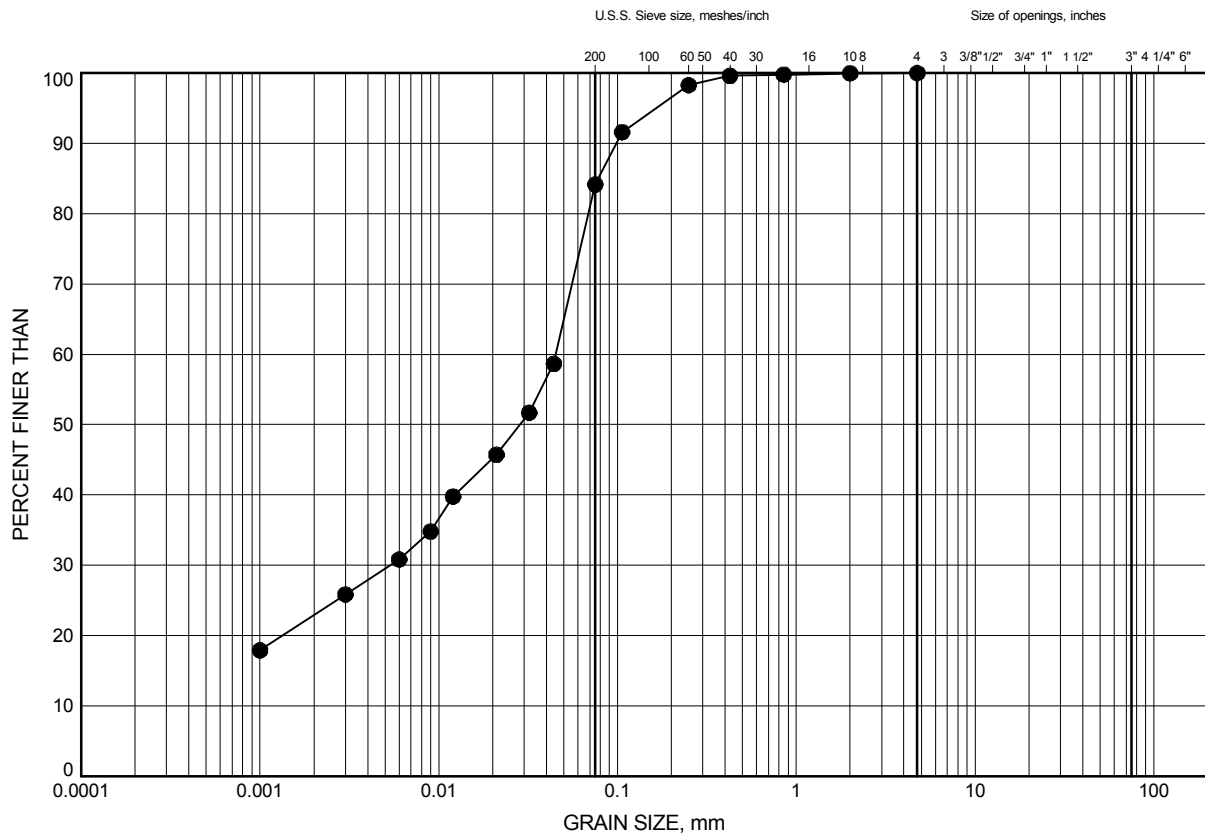


Prep'd CM

Chkd. PC

## GRAIN SIZE DISTRIBUTION

Clay (CL)



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	402	14.63	91.38

Date April 2018

GWP# 4078-14-00



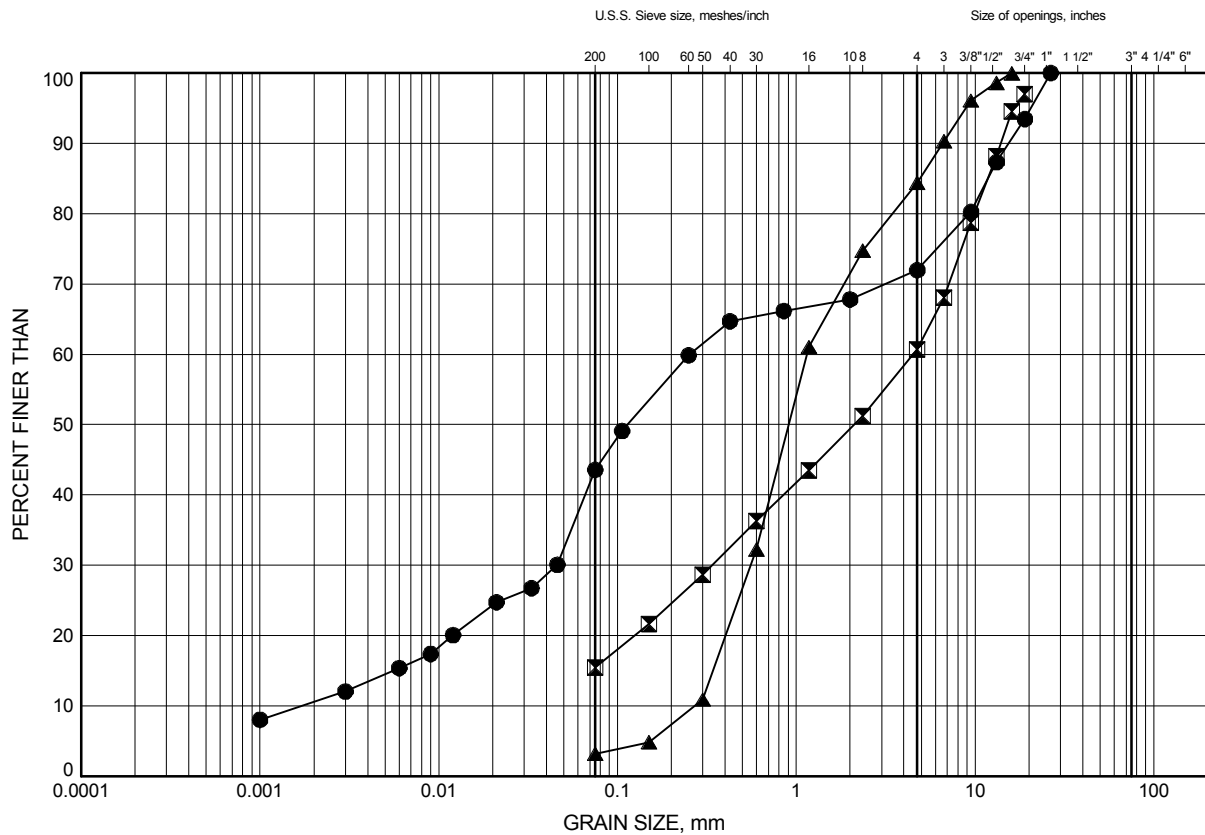
Prep'd CM

Chkd. PC



## GRAIN SIZE DISTRIBUTION

## Glacial Till



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	402	14.94	91.07
⊠	403	14.02	89.29
▲	404	13.72	89.65

Date April 2018

GWP# 4078-14-00



Prep'd CM

Chkd. PC

## **Appendix C.2**

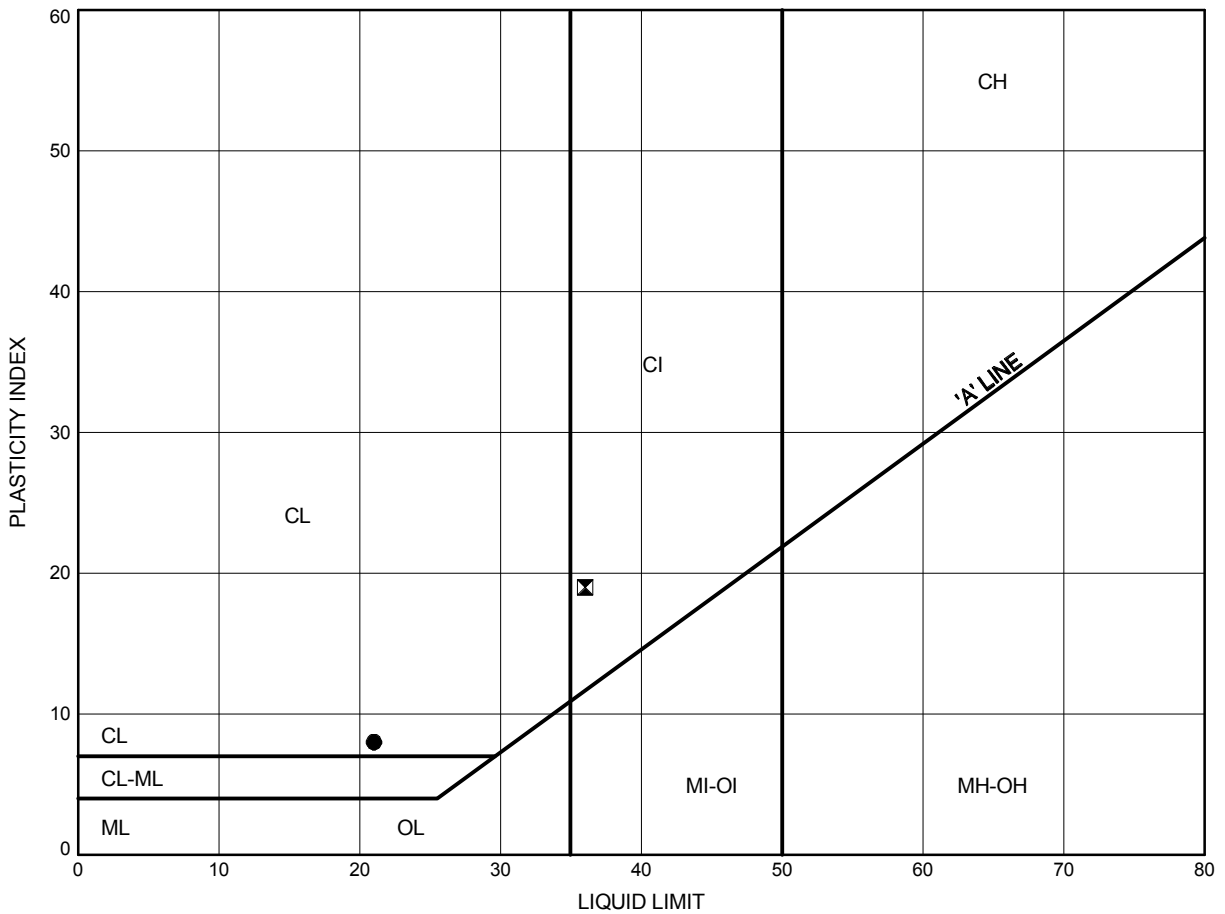
### **Atterberg Limit Analysis Figures**

Site 21-231 - Highway 401 Overpass at Ganaraska River

# ATTERBERG LIMITS TEST RESULTS

FIGURE C6

## Lower Embankment Fill



### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	401	7.16	98.84
⊠	403	6.40	96.91

Date April 2018  
GWP# 4068-14-00



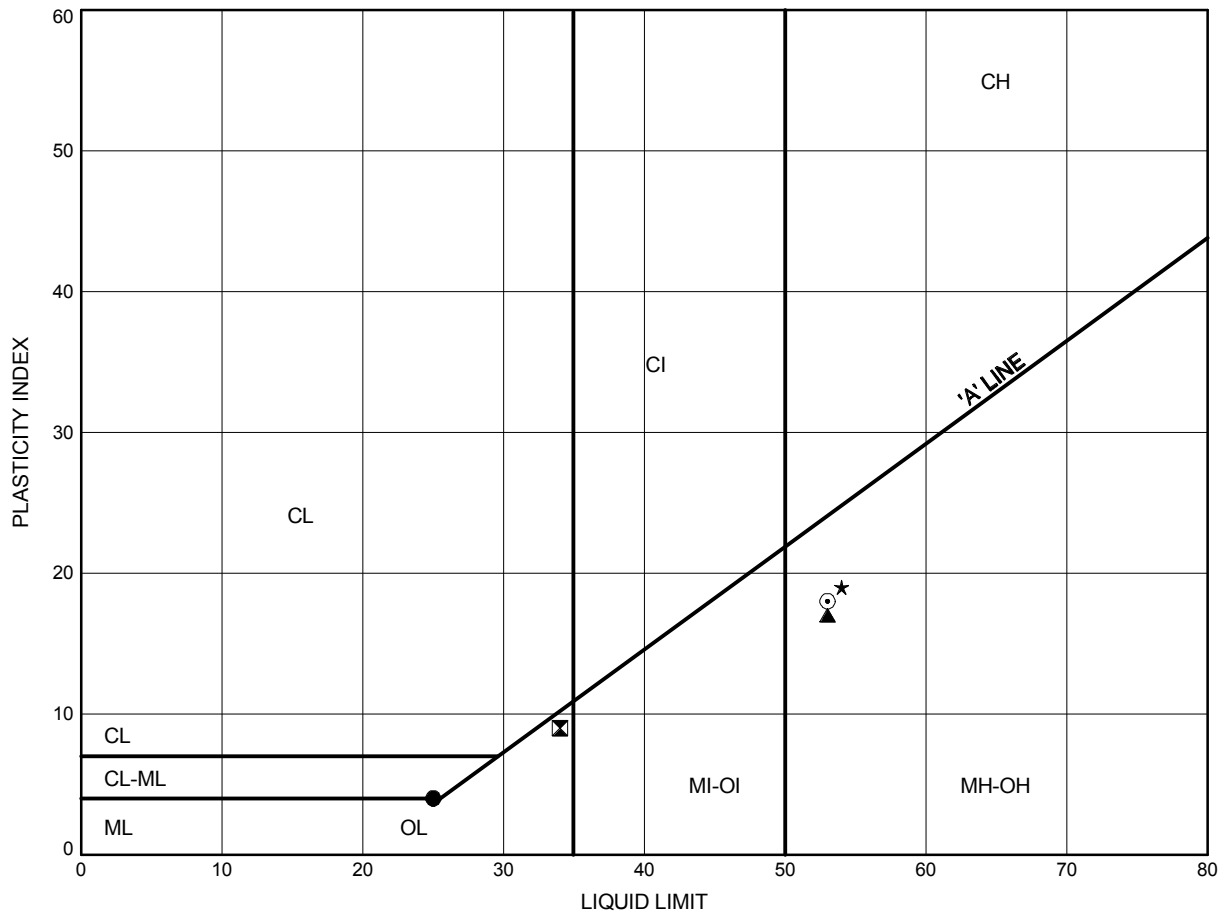
Prep'd CM  
Chkd. PC

Site 21-231 - Highway 401 Overpass at Ganaraska River

# ATTERBERG LIMITS TEST RESULTS

FIGURE C7

## Organic Silt (ML-OL to MH-OH)



### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	401	12.65	93.35
⊠	401	14.02	91.98
▲	402	14.02	91.99
★	403	10.97	92.34
⊙	404	10.97	92.39

Date April 2018  
GWP# 4078-14-00

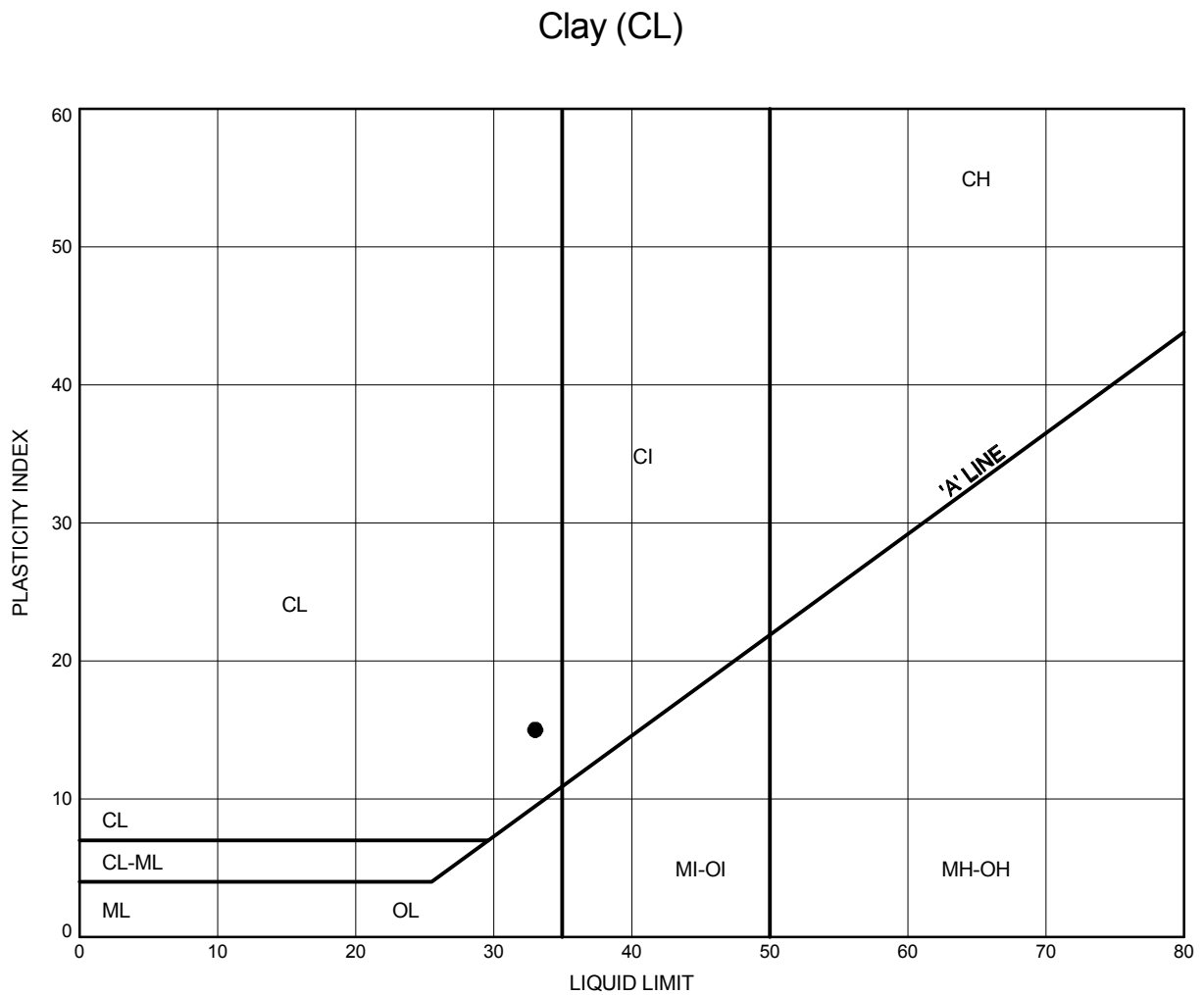


Prep'd CM  
Chkd. PC

Site 21-231 - Highway 401 Overpass at Ganaraska River

# ATTERBERG LIMITS TEST RESULTS

FIGURE C8



## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	402	14.63	91.38

Date April 2018  
GWP# 4078-14-00

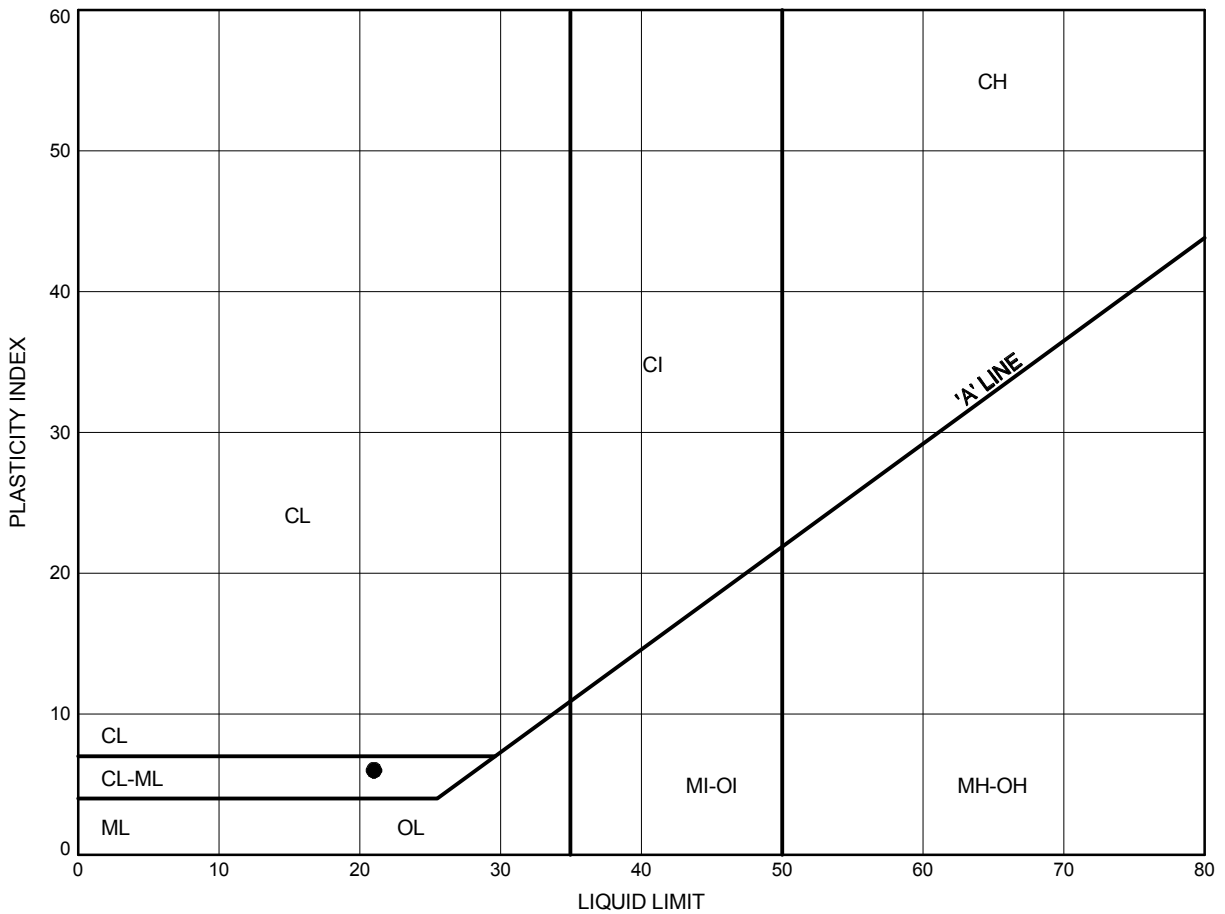


Prep'd CM  
Chkd. PC

Site 21-231 - Highway 401 Overpass at Ganaraska River  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE C9

Glacial Till: Silty Clayey Sand with Gravel



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	402	14.94	91.07

Date April 2018  
 GWP# 4078-14-00



Prep'd CM  
 Chkd. PC

### **Appendix C.3**

#### **Analytical Testing Results**

## Certificate of Analysis

**Thurber Engineering Ltd.**

2460 Lancaster Rd, Suite 104  
Ottawa, ON K1B4S5  
Attn: Chris Murray

Client PO:  
Project: 19-5161-263  
Custody: 27349

Report Date: 15-Jun-2016  
Order Date: 13-Jun-2016

**Order #: 1625054**

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

Paracel ID	Client ID
<del>1625054-01</del>	<del>301 SS3 (5'-7')</del>
<del>1625054-02</del>	<del>304 SS4 (7'6"-9'6")</del>
1625054-03	401 SS4 (7'6"-9'6")
1625054-04	403 SS3 (5'-7')

Approved By:



Mark Foto, M.Sc.  
Lab Supervisor



Certificate of Analysis  
Client: Thurber Engineering Ltd.  
Client PO:

Report Date: 15-Jun-2016  
Order Date: 13-Jun-2016  
Project Description: 19-5161-263

### Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	14-Jun-16	14-Jun-16
Conductivity	MOE E3138 - probe @25 °C, water ext	15-Jun-16	15-Jun-16
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	13-Jun-16	14-Jun-16
Resistivity	EPA 120.1 - probe, water extraction	15-Jun-16	15-Jun-16
Solids, %	Gravimetric, calculation	14-Jun-16	14-Jun-16

Certificate of Analysis  
**Client:** Thurber Engineering Ltd.  
**Client PO:**

Report Date: 15-Jun-2016

Order Date: 13-Jun-2016

**Project Description: 19-5161-263**

		<b>Client ID:</b>	301 SS3 (5'-7')	304 SS4 (7'6"-9'6")	401 SS4 (7'6"-9'6")	403 SS3 (5'-7')
		<b>Sample Date:</b>	31-May-16	01-Jun-16	31-May-16	30-May-16
		<b>Sample ID:</b>	1625054-01	1625054-02	1625054-03	1625054-04
		<b>MDL/Units</b>	Soil	Soil	Soil	Soil
<b>Physical Characteristics</b>						
% Solids	0.1 % by Wt.		90.5	91.9	95.2	97.5
<b>General Inorganics</b>						
Conductivity	5 uS/cm		1220	1260	782	1210
pH	0.05 pH Units		8.21	8.35	8.44	8.31
Resistivity	0.10 Ohm.m		8.17	7.96	12.8	8.26
<b>Anions</b>						
Chloride	5 ug/g dry		650	670	365	758
Sulphate	5 ug/g dry		27	48	21	76

Certificate of Analysis  
**Client: Thurber Engineering Ltd.**  
**Client PO:**

Report Date: 15-Jun-2016  
Order Date: 13-Jun-2016  
**Project Description: 19-5161-263**

### Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
<b>Anions</b>									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
<b>General Inorganics</b>									
Conductivity	ND	5	uS/cm						
Resistivity	ND	0.10	Ohm.m						

Certificate of Analysis  
Client: Thurber Engineering Ltd.  
Client PO:

Report Date: 15-Jun-2016  
Order Date: 13-Jun-2016  
Project Description: 19-5161-263

**Method Quality Control: Duplicate**

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
<b>Anions</b>									
Chloride	629	5	ug/g dry	670			6.3	20	
Sulphate	48.2	5	ug/g dry	48.0			0.4	20	
<b>General Inorganics</b>									
Conductivity	1190	5	uS/cm	1220			3.1	6.2	
pH	12.25	0.05	pH Units	12.27			0.2	10	
Resistivity	8.80	0.10	Ohm.m	8.17			7.4	20	
<b>Physical Characteristics</b>									
% Solids	53.8	0.1	% by Wt.	52.9			1.7	25	

Certificate of Analysis  
Client: Thurber Engineering Ltd.  
Client PO:

Report Date: 15-Jun-2016  
Order Date: 13-Jun-2016  
Project Description: 19-5161-263

**Method Quality Control: Spike**

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
<b>Anions</b>									
Chloride	755	5	ug/g	670	84.4	78-113			
Sulphate	148	5	ug/g	48.0	99.6	78-111			

Certificate of Analysis  
Client: Thurber Engineering Ltd.  
Client PO:

Report Date: 15-Jun-2016  
Order Date: 13-Jun-2016  
Project Description: 19-5161-263

**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.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.  
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

#### **Appendix C.4**

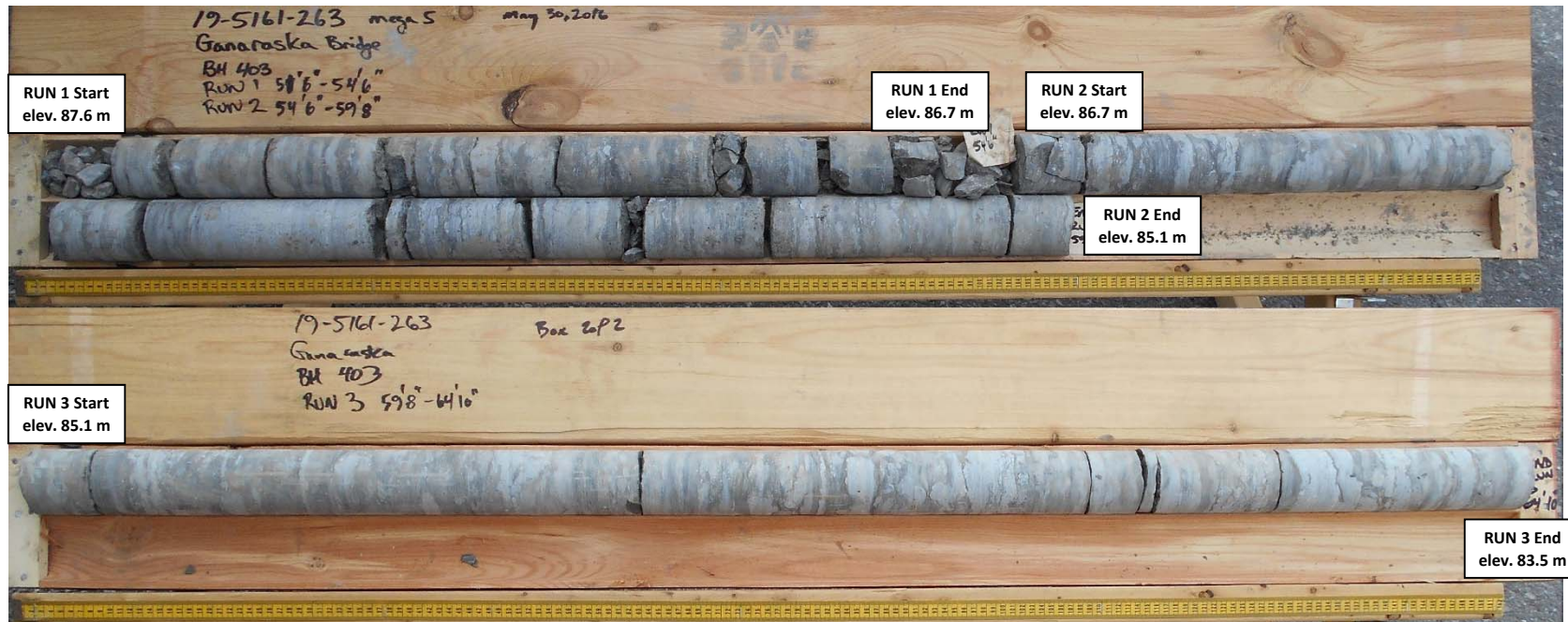
#### **Bedrock Core Photos**

**Borehole 402**  
**RUN 1 to 3 (of 3)**  
**Elevation 90.5 m to 86.3 m**





**Borehole 403**  
**RUN 1 to 3 (of 3)**  
**Elevation 87.6 m to 83.5 m**



**Foundation Investigation**  
**Highway 401 – Ganaraska River Bridge**  
**Site 21-231**  
**Township of Hope, Ontario**

**GWP: 4068-14-00**  
**Project No.: 19-5161-263**

**Appendix D**  
**Selected Site Photographs**



**Photo 1: Looking east along the north side of the Hwy 401 Ganaraska River Bridge**



**Photo 2: Looking east along the north widening area at the Hwy 401 Ganaraska River Bridge**





**Photo 3: Looking east from the west foreslope under the Hwy 401 Ganaraska River Bridge**



**Photo 4: Highway 401 Overpass at The Ganaraska River east foreslope looking from the North**





**Photo 5: Highway 401 Overpass at The Ganaraska River west foreslope looking from the North**

## **Appendix E**

### **Foundation Alternatives Comparisons**

**Comparison of Foundation Alternatives**

<b>Comment</b>	<b>Drilled-in Pipe Piles</b>	<b>Driven Steel H-Piles</b>	<b>Caissons</b>	<b>Micropiles</b>	<b>Spread Footings on Native Soils</b>
<b>Advantages:</b>	<p>Moderate to high geotechnical resistance</p> <p>Better alignment control than caissons</p> <p>Higher lateral resistance than driven piles if socketed into rock</p>	<p>Compatible with existing piles at abutments</p> <p>High axial resistance</p>	<p>High axial geotechnical resistance.</p>	<p>Provide resistance in both tension and compression</p> <p>Easily installed in areas with limited headroom</p>	<p>Quicker installation and lower costs than deep foundations</p>
<b>Disadvantages:</b>	<p>Higher cost than driven piles</p> <p>Fewer contractors with appropriate equipment</p>	<p>Low lateral resistance for short piles</p> <p>Depth to rock may be too short at some foundation elements</p>	<p>Can be difficult to clean and inspect the base.</p> <p>Likely requires concrete to be placed using tremie techniques.</p>	<p>Low resistance to lateral load</p> <p>Lower axial compression resistance than other deep foundation options</p> <p>Higher cost than other alternatives for similar resistance</p>	<p>Provide a lower geotechnical resistance than deep foundations</p> <p>Native soils providing acceptable bearing resistance are too deep at this site to be practical for abutments</p>
<b>Risks / Consequences</b>	<p>Potential impact on adjacent driven piles</p>	<p>Piles are damaged during driving / add additional piles to pile group</p>			
<b>Relative Cost:</b>	<b>Moderate</b>	<b>Moderate</b>	<b>High</b>	<b>High</b>	<b>Low to Moderate</b>
<b>Conclusion:</b>	<b>FEASIBLE</b>	<b>FEASIBLE</b>	<b>FEASIBLE</b>	<b>FEASIBLE</b>	<b>NOT RECOMMENDED</b>

## **Appendix F**

**GSC Seismic Hazard Calculation  
Non-Standard Special Provisions  
List of Referenced Specifications**



# 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

April 27, 2018

Site: 43.9694 N, 78.2941 W User File Reference: Hwy 401 Ganaraska River Bridge

Requested by: C. Murray, Thurber Engineering

**National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)**

Sa(0.05)	Sa(0.1)	<b>Sa(0.2)</b>	Sa(0.3)	<b>Sa(0.5)</b>	<b>Sa(1.0)</b>	<b>Sa(2.0)</b>	<b>Sa(5.0)</b>	<b>Sa(10.0)</b>	PGA (g)	PGV (m/s)
0.163	0.204	<b>0.178</b>	0.140	<b>0.105</b>	<b>0.059</b>	<b>0.029</b>	<b>0.0074</b>	<b>0.0031</b>	<b>0.112</b>	<b>0.087</b>

**Notes.** Spectral ( $S_a(T)$ , where  $T$  is the period in seconds) and peak ground acceleration (PGA) values are given in units of  $g$  ( $9.81 \text{ m/s}^2$ ). Peak ground velocity is given in  $\text{m/s}$ . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity  $450 \text{ m/s}$ ). NBCC2015 and CSAS6-14 values are specified in **bold** font. 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.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.014	0.049	0.086
Sa(0.1)	0.021	0.069	0.114
Sa(0.2)	0.023	0.066	0.104
Sa(0.3)	0.020	0.056	0.085
Sa(0.5)	0.016	0.044	0.066
Sa(1.0)	0.0080	0.025	0.038
Sa(2.0)	0.0034	0.012	0.019
Sa(5.0)	0.0007	0.0027	0.0044
Sa(10.0)	0.0005	0.0012	0.0018
PGA	0.012	0.038	0.063
PGV	0.0100	0.033	0.052

## References

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

**User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx** (in preparation)  
**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](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information

Aussi disponible en français



Natural Resources  
Canada

Ressources naturelles  
Canada

Canada

### **SUGGESTED TEXT FOR “NSSP – PRESENCE OF EXISTING PILES”**

The proposed piles are to be advanced in close proximity to the piles supporting the existing piers and abutments. Although the pile layout on the structural drawings has been selected to avoid conflict with piles supporting the existing bridge piers and abutments the potential for conflict still exists.

Should the new piles encounter the existing piles the Contractor shall report the conflict to Contract Administrator to determine if adjustment to the pile driving program is required.

### **LIST OF REFERENCED SPECIFICATIONS**

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material
NSSP FOUN0003	Amendment to OPSS 902, Dewatering Structure Excavations