



**THURBER** ENGINEERING LTD.



**FOUNDATION INVESTIGATION AND DESIGN REPORT  
RAWDON CREEK BRIDGE REPLACEMENT, HIGHWAY 62  
TOWNSHIP OF HUNTINGDON  
SITE 11-134, G.W.P. 4044-10-00  
ASSIGNMENT NUMBER: 4015-E-0015**

**GEOCRES NUMBER: 31C-256**

**SUBMITTED TO  
McINTOSH PERRY CONSULTING ENGINEERS LTD. / LEA CONSULTING LTD.  
JOINT VENTURE**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual data obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the replacement of the Highway 62 Bridge over Rawdon Creek, located within the Township of Huntingdon, Ontario. Thurber carried out the investigation as a subconsultant to McIntosh Perry Consulting Engineers – LEA Engineering Joint Venture (MPCE-LEA), under Agreement No. 4015-E-0015.

Base plan mapping and the General Arrangement (GA) drawing was provided by the Ministry of Transportation (MTO) Eastern Region Structural Office for the preparation of this report.

The purpose of this 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 11-134 is located on Highway 62, approximately 250 m south of Moira Road in the Township of Huntingdon, Ontario. The location of the structure is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

Highway 62 at this location has one through lane in each direction with narrow paved shoulders. There are concrete barriers and steel beam guide rail systems present on both sides of the highway at the creek location. It is noted that for project orientation purposes, Highway 62 will be assumed to be oriented north-south and Rawdon Creek to be oriented east-west.

Based on the GA drawing dated July 2015, the existing bridge is a single span structure, approximately 12.3 m long that carries two lanes of Highway 62 traffic over Rawdon Creek. It is proposed to construct the replacement bridge on a new alignment approximately 14.4 m west of the existing alignment.

The site is located near the boundary of three physiographic regions: the Dummer Moraine, the Peterborough Drumlin Field and the Iroquois Plain though the soil conditions on site most closely resemble those of the Iroquois Plain region. The Iroquois Plain region is characterized by the flat to undulating lake bed and beaches of the former glacial Lake Iroquois that existed during the last glacial recession. The overburden soils are comprised of glaciolacustrine sand, silt and clay deposits (though deposits of sand and gravel are also known to be present) all underlain by limestone bedrock (Chapman and Putnam, 1984).

The lands surrounding the project limits are typically agricultural with some residential properties. Storm water drainage in the area is to existing ditches and culverts. Site photographs showing the general conditions at the site are presented in Appendix E.

### 3 SITE INVESTIGATION

#### 3.1 Previous Investigations

A Preliminary Foundation Investigation for this site was carried out in 2012 (Golder Associates Report No. 12-1111-0021-1). The investigation consisted of advancing one borehole at each proposed abutment (Boreholes RC-1 and RC-2). A copy of the borehole location plan and the Record of Boreholes from the preliminary investigation are provided in Appendix C.

The stratigraphy in the area of the bridge is generally described as surficial deposits of firm to stiff silty clay (on the south side of the creek) or loose to compact sand and gravel (on the north side of the creek), overlying a deposit of loose to compact silty sand to sandy silt, underlain by a deposit of compact to very dense sand and gravel, which contains cobbles and boulders, all overlying limestone bedrock. The bedrock surface was encountered at approximately elevation 123.2 m and 122.3 m at the north and south abutments, respectively.

#### 3.2 Field Investigation

A field investigation was carried out as part of the current detailed design assignment to supplement the data from the preliminary foundation investigation.

The field investigation plan was finalized after discussion with the MTO Foundations Section. The field investigation for this site included advancing four boreholes drilled between October 3<sup>rd</sup> and 4<sup>th</sup>, 2016. The approximate locations and elevations of the boreholes are shown on Drawing No. 1 provided in Appendix A and are summarized in Table 3-1.

**Table 3-1: Borehole Summary**

Borehole	Location	Latitude (degrees)	Longitude (degrees)	Ground Surface Elevation (m)	Depth (m)
16-1	South Approach	44.33797	-77.47741	133.9	10.0
16-2	South Abutment	44.33814	-77.47754	133.1	13.6
16-3	North Abutment	44.33828	-77.47777	131.4	12.3
16-4	North Approach	44.33844	-77.47786	131.4	6.9

As a component of our standard procedures and due diligence, Thurber contacted Ontario One Call and MTO Electrical to obtain utility locates/clearances for the intended borehole locations.

The boreholes were advanced with an ATV mount CME55 drill rig equipped with hollow stem augers and HW casing. 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 during the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586-11. In-situ shear vane testing was carried out within the cohesive strata. All soil samples recovered from the boreholes were placed in moisture-proof containers and transported to Thurber's Ottawa geotechnical laboratory for further examination and testing. Bedrock was cored in Boreholes 16-2, and 16-3 with HQ size coring equipment

following ASTM Standard D6032-08. Bedrock core samples were stored in core boxes for transport.

A 50 mm inside diameter PVC monitoring well was installed in Boreholes 16-2 and 16-3 to allow for measurement of the groundwater level at the site. The monitoring well construction details are illustrated on the Record of Borehole sheet for Boreholes 16-2 and 16-3, provided in Appendix B.

The boreholes without installations were backfilled with a low-permeability combination of auger cuttings, and bentonite pellets in general accordance with the intent of Ontario MOE Regulation 903.

The as-drilled locations of the boreholes and ground surface elevations at the borehole locations were surveyed by Thurber on October 5, 2016. The vertical datum used was the benchmark (GBM) 8321 identified on the plans provided by MTO, which is located on the southeast abutment of the existing bridge. The GBM has a geodetic elevation of 134.476 m.

### **3.3 Laboratory Testing**

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all soil samples in accordance with the current MTO standards. Grain size distribution analyses, and Atterberg Limits testing were also carried out on selected samples to MTO and ASTM standards. Point Load Strength Index Testing as also carried out on selected samples of the bedrock core. 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 are provided in Appendix D.

The laboratory test results are presented on the Record of Borehole sheets in Appendix B and are illustrated on the figures in Appendix D.

## **4 DESCRIPTION OF SUBSURFACE CONDITIONS**

### **4.1 Overview / General**

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. A stratigraphic profile for the site is presented on the Drawing No. 1 in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions.

In general, the stratigraphy in the area of the boreholes is characterized by silty sand, clay and silt (at the south embankment area) or silty sand with gravel (at the north embankment area), overlying sandy silt to silty sand, overlying silty sand with gravel till, underlain by limestone bedrock. This stratigraphy is generally consistent with the stratigraphy encountered in the preliminary investigation. More detailed descriptions of the individual strata are presented below.

More detailed descriptions of the individual strata are presented below.

### **4.2 Rootmat**

A rootmat layer was encountered at the surface in all boreholes. The thickness of the rootmat ranged from 50 mm to 150 mm.

### 4.3 Silty Sand with Gravel

A silty sand layer with varying amounts of gravel was encountered below the rootmat in all four boreholes. The top of this layer ranges from elevation 131.2 m to 133.8 m and has a thickness ranging from 0.5 m to 2.2 m. The SPT 'N' values ranged from 3 to 28; indicating a very loose to compact condition. The moisture content of the samples tested ranged from 3% to 21%. A slight hydrocarbon odour was noted in a single sample in Borehole 16-4 at an approximate elevation 130.0 m.

### 4.4 Clay

A clay deposit was encountered beneath the silty sand with gravel deposit in Boreholes 16-1 and 16-2. The top of this layer ranges from elevation 132.5 m to 133.3 m and has a thickness of 1.7 m. The SPT 'N' values ranged from 12 to 21 blows; indicating stiff to very stiff consistency.

The moisture content of the samples tested ranged from 20% to 38%. The results of a grain size analysis completed on a sample of this material indicated a gravel content of 0%, sand content of 12%, silt content of 35%, and a clay content 53%. The results of the grain size analysis are illustrated on Figure 1 in Appendix D.

The results of Atterberg Limits testing completed on a sample indicated a plastic limit of 20, a liquid limit of 47, and a plasticity index of 27. Atterberg Limits analysis results are illustrated on Figure 2 in Appendix D.

### 4.5 Silt

A silt deposit was encountered beneath the clay strata in Boreholes 16-1 and 16-2. The top of this layer ranges from elevation 130.8 m to 131.6 m. The thickness of the deposit ranges from 0.6 m to 2.1 m. The SPT 'N' values ranged from 9 to 14; indicating a loose to compact condition.

The moisture content of the samples tested ranged from 25% to 33%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-1 and are illustrated on Figure 3 in Appendix D.

**Table 4-1: Gradation Results**

Soil Particles	%
Gravel	0
Sand	1 to 7
Silt	87 to 92
Clay	6 to 7

Atterberg Limits testing conducted on samples of this material indicated a non-plastic silt.

### 4.6 Sandy Silt to Silty Sand

A silt and sand deposit was encountered beneath the silt strata in Boreholes 16-1 and 16-2 and below the silty sand with gravel strata in Boreholes 16-3 and 16-4. The top of this layer ranges from elevation 129.1 m to 130.2 m. The thickness of the deposit ranges from 2.9 m to 5.9 m. The SPT 'N' values ranged from 2 to 16; indicating a very loose to compact condition.

The moisture content of the samples tested ranged from 19% to 27%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-2 and are illustrated on Figure 4 in Appendix D.

**Table 4-2: Gradation Results for Sandy Silt to Silty Sand Till**

<b>Soil Particles</b>	<b>%</b>
Gravel	0 to 6
Sand	23 to 49
Silt and Clay	48 to 77

Atterberg Limits testing conducted on samples of this material indicated that the fines portion to be non-plastic.

#### **4.7 Silty Sand with Gravel (Till)**

A glacial till deposit consisting of silt and sand with varying amounts of gravel was encountered beneath the sandy silt to silty sand materials in all boreholes. The top of this layer ranges from elevation 124.2 m to 126.2 m. Where completely penetrated, the thickness ranged from 1.4 m to 2.4 m. Boreholes 16-1 and 16-4 were terminated in this stratum.

The SPT 'N' values ranged from 46 to greater than 100; indicating a dense to very dense condition. Cobbles and boulders were noted in this stratum. The moisture content of the samples tested ranged from 7% to 11%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-3 and are illustrated on Figure 5 in Appendix D.

**Table 4-3: Gradation Results for Till**

<b>Soil Particles</b>	<b>%</b>
Gravel	22 to 41
Sand	32 to 59
Silt and Clay	15 to 27

#### **4.8 Bedrock**

The overburden materials were underlain by grey limestone bedrock. The bedrock elevation at the south abutment boreholes ranged from elevation 122.3 m to 122.9 m and from 122.6 to 123.2 at the north abutment. Photographs of the bedrock core are provided in Appendix B.

Boreholes 16-2 and 16-3 were advanced into the bedrock by coring with HQ-size core bits. The total core recovery ranged from 98% to 100%, the solid core recovery ranged from 98% to 100% and the Rock Quality Designation ranged from 81% to 100%. Based on the RQD value the bedrock is classified as good to excellent quality. Based on point load strength index testing the bedrock is classified as strong to very strong. A copy of the index test results is provided in Appendix D.



## 4.9 Groundwater

The groundwater level in the monitoring wells installed in Boreholes 16-2 and 16-3 and the existing well from the preliminary investigation was recorded on October 24, 2016 and again on April 10, 2016. The depth below ground surface and elevation of the groundwater measured is summarized in Table 4.4.

**Table 4-4: Groundwater Level Measurements**

Borehole	October 24, 2016		April 10, 2017	
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
16-2	2.6	130.5	1.5	131.6
16-3	0.95	130.5	0.01	131.4
RC-2	2.4	130.7	1.6	131.5

These observations are considered short-term readings 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.

## 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 determined the ground surface elevations based on contract drawings provided by MTO. Terex Drilling Solutions of Concord, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, and in-situ testing. The drilling, and sampling operations in the field were supervised on a full time basis by Mr. 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 report presents the interpretation of the factual data obtained from a foundation investigation conducted by Thurber for the replacement of the Highway 62 Bridge over Rawdon Creek, in the Township of Huntingdon, Ontario. Geotechnical recommendations are provided to assist the design team in designing a suitable foundation for the proposed bridge replacement.

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. The design-build contractor 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 geotechnical recommendations for the replacement of the existing overpass structure on a new alignment to the west of the existing structure. The discussions and recommendations presented in this report are based on the information provided by MPCE and MTO and on the factual data obtained during the course of this investigation.

**6.1 Proposed Structure**

It is understood that the preferred design alternative identified during the preliminary design study consists of a new single span integral abutment bridge on a permanent new alignment to the west of the existing bridge. Based on the updated GA drawing provided the by the Ministry, details regarding the proposed structure include:

- The proposed span is 22 m;
- The bridge deck will have an approximate width of 14.3 m to accommodate two, 4.15 m wide lanes, 2.5 m wide shoulders and concrete barrier walls with railings;
- Proposed top of bridge deck elevations will range from 135.403 m and 135.026 m at the south and north abutment respectively; and
- Embankment side slopes graded at 2H:1V (horizontal to vertical) are proposed.

Based on the preliminary span configuration, the proposed top of deck elevation and existing ground surface elevations measured during Thurber's field investigation, the approach fills for the new alignment will be approximately 2.3 m and 3.6 m at the south and north abutments, respectively. The existing elevations, grade raises and proposed elevations after constructing the proposed embankments are outlined in Table 6-1.

**Table 6-1: Proposed Profile Grades**

<b>Abutment</b>	<b>Existing Ground Surface (m)</b>	<b>Approximate Height of Approach Fill (m)</b>	<b>Proposed Top of Pavement (m)</b>
South	133.1	2.3	135.4
North	131.4	3.6	135.0

## 6.2 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 6-2: Bridge Structure Classification**

<b>Criteria</b>	<b>Classification</b>	<b>CHBDC Section</b>
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 1.5 m as per OPSD 3090.101.

## 6.3 Geotechnical Assessment

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

- From a geotechnical perspective, the ground conditions at the site are generally suitable for integral abutments.
- Deep foundations should be driven to bedrock, however, a thin layer of silty sand with gravel containing frequent cobbles and boulders was encountered directly above the bedrock; the tips of piles should be reinforced to reduce the risk of damage to the piles while driving through this layer.
- The loose to compact silty sand deposits will be easily disturbed when saturated or if water is permitted to seep through it. If excavations are extended below the water level or creek level, appropriate dewatering methods will be required to ensure that the base of the excavation is stable.

## **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).

### **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 abutment 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 abutments include loose to compact sandy silt to silty sand overlying a dense to very dense glacial till deposit. These deposits are not considered susceptible to liquefaction under earthquake loading using the site specific PGA value of 0.091g.

## **8 FOUNDATION DESIGN RECOMMENDATIONS**

### **8.1 Deep Foundations – Steel Piles**

The abutments may be founded on steel HP 310x110 piles end-bearing on bedrock or practical refusal within the glacial till.

It is recommended that the piles be driven to bedrock, however, due to the presence of cobbles and boulders within the till, it may not be possible to drive all piles to bedrock without damaging the piles. Therefore, for design purposes, the geotechnical resistance should be based on piles reaching practical refusal within the glacial till.

The estimated pile tip elevations, based on piles reaching the bedrock, are summarized in Table 8-1.

**Table 8-1: Estimated Pile Tip Elevations**

Foundation Element	Approximate Underside of Pile Cap Elevation (m)	Borehole	Estimated Pile Tip Elevation, Bedrock Surface (m)	Estimated Pile Length (m)
South Abutment	131.5	16-2	122.9	8.6
		RC-2	122.3	9.2
North Abutment	131.0	16-3	122.6	8.4
		RC-1	123.2	7.8

The design parameters for axial resistance of Grade 350W HP 310x110 steel piles driven to practical refusal within the glacial till deposit can be taken as:

- Factored vertical geotechnical resistances at ULS 1,400 kN
- Factored vertical geotechnical reaction at SLS 1,200 kN

The factored geotechnical resistances provided account for the short pile length and 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)
  - $\phi_{gs} = 0.8$ , SLS (static analysis; typical degree of understanding)

In light of the cohesionless makeup of the existing overburden soils at this site, downdrag on the new piles is not considered a design issue.

#### 8.1.1 Pile Lateral Resistance

##### Lateral Resistance at ULS

The ultimate geotechnical resistance of a single driven H-pile can be calculated based on the passive resistance of the adjacent soil.

For a single pile in non-cohesive soils, the passive resistance should be calculated based on the passive earth pressure over an equivalent wall area having a depth from the underside of pile cap equal to six times the pile diameter/width, and a width equal to three times the pile diameter/width.

Group effects must be considered and may be calculated based on the sum of the ultimate lateral resistance of the individual piles in the group with the individual resistances reduced by the group efficiency factors (as per Figures C6.11.3 in the CHBDC commentary).

It is noted that the static analysis methods identified above do not account for the influence of fixity or axial loads on the pile and do not consider the structural resistance.

The horizontal component of the axial resistance of battered piles (where present) also contributes to the lateral resistance.

The factored lateral resistance of piles at ULS determined based on the data and methods provided above should incorporate a resistance factor ( $\phi_{gu}$ ) of 0.5 as per Table 6.2 of the CHBDC (static analysis – typical degree of understanding).

### Lateral Resistance at SLS

The interaction between a pile and the surrounding soil may be analysed using horizontal subgrade reaction theory. For a single driven H-pile, the coefficient of horizontal subgrade reaction ( $k_h$ ) may be calculated based on the following equations (Terzaghi, 1955) in conjunction with the parameter values provided in Table 8-2.

$$k_h = \frac{n_h z}{B}$$

For cohesionless soils

$$k_h = \frac{67 S_u}{B}$$

For cohesive soils

where:

$k_h$  = coefficient of horizontal subgrade reaction

$n_h$  = coefficient related to soil density

$Z$  = depth of pile embedment (m)

$B$  = pile width perpendicular to load direction (m)

$S_u$  = undrained shear strength (kPa)

The geotechnical parameters for use in assessment of the piles are presented in Table 8-2.

**Table 8-2: Parameters for Lateral Pile Resistance**

Location	Elevation (m)	Soil	$\gamma'$ (kN/m <sup>3</sup> )	$n_h$ (kPa/m)	$S_u$ (kPa)	$K_p$
South Abutment	133.1 to 132.5	Silty Sand with gravel	20.0	3,000	-	2.7
	132.5 to 130.8	Clay	18	-	60	
	130.8 to 130.2	Silt	10.2	2,000	-	
	130.2 to 124.2	Sandy Silt to Silty Sand	10.2	2,500	-	
	124.2 to 122.9	Glacial Till	11.2	6,000	-	3.7
North Abutment	131.4 to 129.7	Silty Sand with gravel	20.0	3,000	-	2.7
	129.7 to 125.0	Sandy Silt to Silty Sand	10.2	2,500	-	2.7
	125.0 to 122.6	Glacial Till	11.2	6,000	-	3.7

The elevations provided above are based on the borehole data and current grades. Where piles are installed within CSP filled with loose sand, properties of the loose sand will govern the soil-pile interaction and the  $n_h$  value of the loose sand should be used ( $n_h = 1,300$  kPa/m,  $\gamma' = 19.5$  kN/m<sup>3</sup>, and  $K_p = 3.00$ ).

The spring constant,  $K$ , for analysis may be obtained by the expression

$$K = k_h L D$$

where:

$k_h$  = coefficient of horizontal subgrade reaction (kN/m)

$L$  = Length of pile segment (m)

$D$  = pile width (m)

The coefficient of horizontal subgrade reaction may have to be reduced to account for group effects, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.3. Intermediate values may be obtained by linear interpolation.

**Table 8-3: Subgrade Reaction Reduction Factors for Pile Spacing**

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
<b>Pile group oriented perpendicular to direction of loading</b>	4D	1.0
	1D	0.5
<b>Pile group oriented parallel to direction of loading</b>	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

The factored lateral resistance of piles at SLS determined based on the data and methods provided above should incorporate a resistance factor ( $\phi_{gs}$ ) of 0.8 as per Table 6.2 of the CHBDC (static analysis – typical degree of understanding).

#### 8.1.2 Integral Abutment

The subsurface conditions at this site are considered suitable for integral abutment design. The H-pile length below the abutment should be a minimum of 5.0 m which based on conditions encountered as outlined above, this condition should be met.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The native soils within the 3 m directly beneath the abutments generally consist of loose to compact silty sand to sandy silt (typical SPT N-value of 11). Given the relatively short span and small movement expected, the native soil is sufficiently flexible that the typical 3m long CSP filled with loose sand is not required at this site.

#### 8.1.3 Pile Installation

Driven piles must be installed in accordance with OPSS.PROV 903.

Due to the presence of cobbles and boulders within the glacial till deposit the pile tips of new piles driven 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 notes on the foundation drawings should indicate “Piles to be driven to bedrock.”.

#### 8.1.4 Frost Protection

The frost penetration depth at this site is 1.5 m as per OPSD 3090.101. Accordingly, a minimum of 1.5 m of earth cover, or equivalent insulation, must be provided above the base of the pile caps to serve as frost protection.



## 8.2 Lateral Earth Pressures

The lateral earth pressure parameters provided in Table 8-5 and 8-6 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.

### 8.2.1 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>)

$h$  = 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 8-4.

**Table 8-4: Static Lateral Earth Pressure Coefficient**

Parameter	OPSS Granular A & B Type II	Existing Fill	Native Silt & Silty Sand	Native Clay
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	21.0	20.0	20	18
Angle of Internal Friction, $\phi$	35°	30°	27°	27°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.50	0.55	0.55
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33	0.38	0.38

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.

For static analysis, passive earth resistance should be ignored, and therefore have 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.

### 8.2.2 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 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 8-5 assume the following:

- Horizontal back-slope behind the wall; and
- Seismic Site Class of D, and a PGA with a 2% probability of exceedance in 50 years of 0.091g; as outlined in Section 7.0.

**Table 8-5: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & B Type II	Existing Granular Fill	Native Silt & Silty Sand	Native Clay
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	21.0	20.0	20.0	18.0
Angle of Internal Friction, $\phi$	35°	30°	27°	27°
<b>Non-Yielding Wall</b>				
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.34	0.41	0.46	0.46
<b>Yielding Wall</b>				
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.30	0.37	0.41	0.41

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 active earth pressure coefficient

( $K_a$  for yielding walls and  $K_0$  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)

### 8.3 Approach Embankments

The proposed profile and bridge spans require approach fill heights ranging from 2.3 m to 3.6 m above the existing grade at the south and north abutments, respectively.

The embankment construction should be carried out in accordance with OPSS.PROV 206. The embankment material should consist of imported OPSS Select Subgrade (SSM) material or better. Excavated granular fill may also be reused as embankment fill provided it is unfrozen and there is no organic material in the excavated fill and there is sufficient space to stockpile on site and control the moisture content within acceptable limits for compaction.

The fill material should be placed and compacted in accordance with OPSS.PROV 501.

### 8.3.1 Assessment of Settlement

An assessment of the settlement that would result from construction of the proposed embankment construction with 2H:1V side slopes was carried out using Rocscience's Settle<sup>3D</sup> modelling software.

The estimated settlement of the approach embankments at the abutments was compared to MTO Guidelines for post construction settlement over a period of 20 years after paving are outlined below:

- 25 mm within 20 m behind bridge abutment;
- 50 mm from 20 to 50 m from the bridge abutment; and
- 100 mm for greater than 50 m from the bridge abutment.

Based on settlement analysis, if the proposed embankments are constructed as outlined above the predicted settlement is less than 25 mm and is considered to be immediate. The predicted settlement values reflect both the maximum embankment height as well as the aerial distribution of the fill.

It is noted that buried organic layers were identified beneath the new highway alignment on the north side of the structure in the pavement investigation. All buried organic layers should be removed from beneath the approach embankments.

### 8.3.2 Assessment of Global Stability

The global stability for the proposed grade raise constructed using conventional granular fill with 2H:1V side slopes was evaluated using GeoStudio 2012 Slope/W software for limit equilibrium analysis. Input parameters for undrained analysis are based on the in-situ SPT 'N' values and in-situ shear vane testing.

The following additional parameters were used in the analysis:

- A traffic surcharge load as per Section 6.12.5 of the CHBDC
- A seismic horizontal loading of 0.059, equal to ½ of the site adjusted PGA value (0.117g) was used for seismic analysis

**Table 8-6: Global Stability Analysis Results – Grade Raise Constructed with Granular Fill**

Location	Factory of Safety	
	Static Conditions	Seismic Conditions
North Abutment	1.5	1.4
South Abutment	1.5	1.4

The factor of safety does meet the target values of 1.3 and 1.0 under static and seismic conditions respectively.

## 8.4 Cement Type and Corrosion Potential

Two soil samples were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The analysis was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations to buried infrastructure. The analysis results are summarized in the Table 8-7. A copy of the test results is provided in Appendix D.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in the Table 8-7 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

**Table 8-7: Results of Chemical Analysis**

Borehole	Sample	Depth m	pH	Resistivity (Ohm-m)	Chloride (µg/g)	Sulphate (µg/g)
16-2	SS5	3.4	7.8	16.4	299	13
16-4	SS5	3.4	7.9	42.1	82	10

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater.

## 9 CONSTRUCTION CONSIDERATIONS

### 9.1 Excavations

It is anticipated that temporary excavations in the order of 2 m will be required for the installation of the new abutments.

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills and silts and sands at the site should be classified in accordance with OHSA as Type 3 above the groundwater and Type 4 below the water table unless dewatering is carried out to prevent seepage.

Subgrade preparation and construction of the abutments must be carried out in the dry.

### 9.2 Temporary Protection Systems

Temporary protection systems should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2.

Design of the temporary protection systems is the responsibility of the contractor. All shoring should be designed by a licensed engineer experienced in such designs. Lateral earth pressure coefficients for the use in the design are provided in Table 8-4. The designer of the protection system should ensure the penetration depth is sufficient to provide base fixity and incorporate traffic loading and surcharge loading due to construction equipment and operations. A suitable bracing system may also need to be incorporated into the design.

Increased difficulty with the installation of protection systems should be anticipated due to the presence of boulders within the native glacial till. Sheet piles systems may encounter obstructions in the till at this site and protection systems may consist of H-piles with timber lagging with tie back anchors. All protection systems should be designed by a Professional Engineer experienced in such designs.

### **9.3 Dewatering**

All excavations for foundations must be dewatered prior to the placement of concrete, as per OPSS 902. Design and implementation of an effective dewatering system is the responsibility of the contractor

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 any excavation at all times. Groundwater perched within the embankment fill and, surface runoff will tend to seep into, and accumulate in proposed excavations.

Dewatering design and decisions regarding dewatering, must be carried out by the Contractor. Due to the shallow excavation depths being considered and the depth to groundwater at the site it is anticipated that convention sump and pump techniques should be sufficient.

### **9.4 Erosion Control and Scour Protection**

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. 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. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

Scour and erosion protection should be provided along the creek banks in the area of the new bridge. Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS 511. The design of the scour and erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in this field.

### **9.5 Construction Concerns**

The planned construction methodology includes excavations for shallow foundations, abutments and the driving of deep foundations

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

- Damage to piles tips during driving may occur due to presence of boulders within the till and driving to bedrock, tips should be protected from damage during driving. A draft NSSP alerting the contractor to the presence of obstructions (e.g. cobbles and boulders) is provided in Appendix F.
- The existing bridge structure and pavement must not be adversely impacted during construction of the new bridge. The use of temporary protection may be required.

- Vibration monitoring of the existing bridge during pile driving for the new bridge may also be required. A draft NSSP for vibration monitoring during pile driving operations is provided in Appendix F.
- Construction should be kept above the existing creek level at the time of construction.
- Confirmation that the backfill is adequately placed and compacted to specifications.

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

## 10 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 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.



Kenton C. Power, P.Eng.  
Geotechnical Engineer



Paul Carnaffan, P.Eng.  
Principal | Senior Geotechnical Engineer



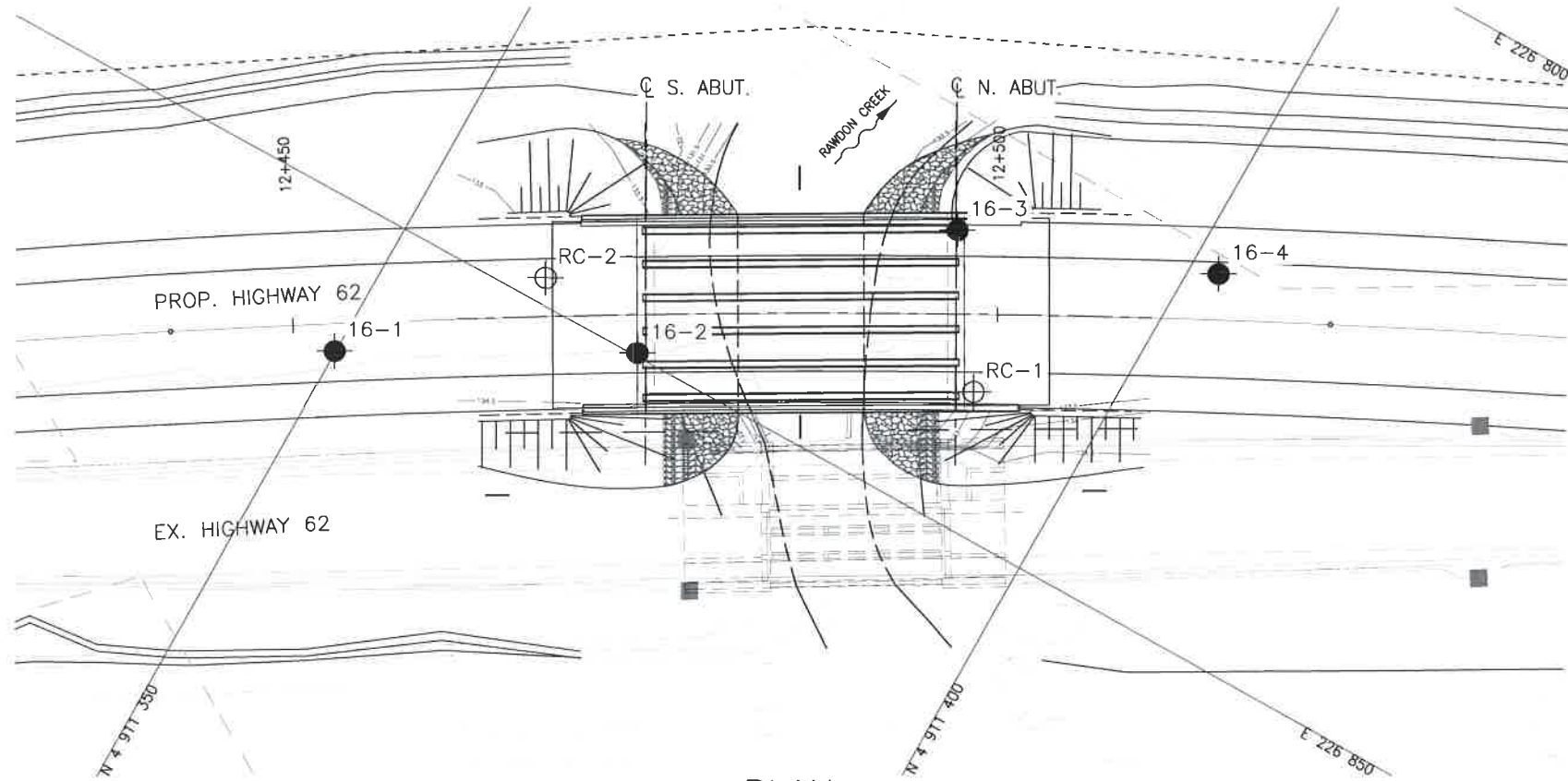
P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

**APPENDIX A**

**2016 INVESTIGATION**

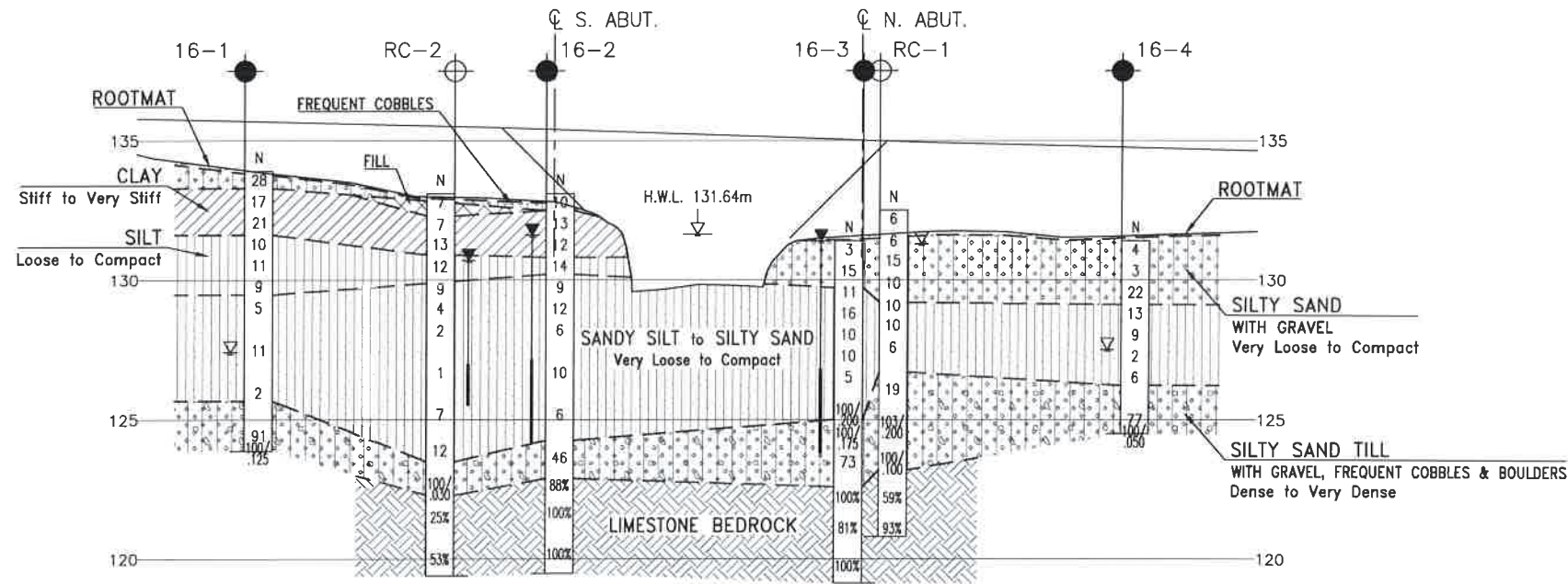
**BOREHOLE LOCATIONS AND SOIL STRATA DRAWINGS**





PLAN

SCALE 1:500



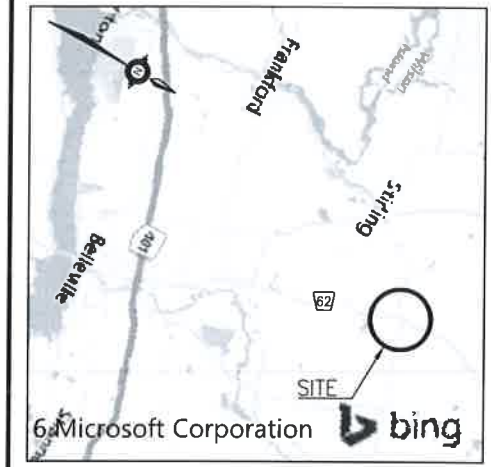
PROFILE

H 1:500  
V 1:250

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

CONT No  
GWP No 4044-10-00

HIGHWAY 62  
RAWDON CREEK  
BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

●	Borehole
⊕	Borehole and Cone
⊕	Preliminary Borehole (by Others)
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
16-1	133.9	4 911 349.9	226 860.0
16-2	133.1	4 911 368.8	226 849.7
16-3	131.4	4 911 384.3	226 831.1
16-4	131.4	4 911 402.0	226 824.8
RC-1	132.5	4 911 391.0	226 840.6
RC-2	133.1	4 911 360.5	226 848.2

-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.
- Borehole locations are shown in MTM Zone 9 coordinates.

GEOCREs No. 31C-256

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KP	CHK -	CODE
DRAWN	MFA	CHK KP	SITE 11-134
			STRUCT
			DATE APR 2017
			DWG 1



## **APPENDIX B**

### **2016 INVESTIGATION RECORD OF BOREHOLE SHEETS BEDROCK CORE PHOTOGRAPHS**

## 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):

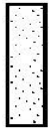
Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The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

### STRATA PLOT:

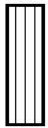
Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



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

### 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 (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.

### 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 16-1

1 OF 2

METRIC

GWP# 4044-10-00 LOCATION Highway 62 Rawdon Creek Bridge, MTM Zone 9: N 4 911 349.9 E 226 860.0 ORIGINATED BY CAM  
 HWY 62 BOREHOLE TYPE Hollow Stem Auger COMPILED BY CAM  
 DATUM Geodetic DATE 2016.10.04 - 2016.10.04 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  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 (%)	
								○ UNCONFINED	+ FIELD VANE								
								● QUICK TRIAXIAL	× LAB VANE								
133.9								20	40	60	80	100					
0.0	75 mm ROOTMAT																
0.1	Silty SAND (SM) with gravel Compact Brown		1	SS	28												
133.3																	
0.6	CLAY (Cl) Stiff to very stiff Brown																
			2	SS	17		133									0	12 35 53
			3	SS	21		132										
131.6																	
2.3	SILT (ML) Loose to compact Grey																
			4	SS	10		131										
			5	SS	11		130									0	1 92 7
			6	SS	9		130										
129.5																	
4.4	Sandy SILT (ML) to Silty SAND (SM) Very loose to compact Brown																
			7	SS	5		129										
							128										
	- slight hydro-carbon odour in sample SS8		8	SS	11		127										
			9	SS	2		126									0	49 51 (SH+CL)
125.7																	
8.2	Silty GRAVEL (GM) with sand, TILL - frequent cobbles and boulders Very dense Brown																
										</							

Continued Next Page

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

# RECORD OF BOREHOLE No 16-1

2 OF 2

METRIC

GWP# 4044-10-00 LOCATION Highway 62 Rawdon Creek Bridge, MTM Zone 9: N 4 911 349.9 E 226 860.0 ORIGINATED BY CAM  
 HWY 62 BOREHOLE TYPE Hollow Stem Auger COMPILED BY CAM  
 DATUM Geodetic DATE 2016.10.04 - 2016.10.04 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			SHEAR STRENGTH kPa									
								20	40	60	80	100					
	Continued From Previous Page																
10.0	End of Borehole Split Spoon refusal on inferred boulders Groundwater level was measured in the open borehole at 6.5 m BGS (elev. 127.4 m)		11	SS	100/125mm												

ONTMT4S RAWDON CREEK BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 25/4/17



# RECORD OF BOREHOLE No 16-2

1 OF 2

METRIC

GWP# 4044-10-00 LOCATION Highway 62 Rawdon Creek Bridge, MTM Zone 9: N 4 911 368.8 E 226 849.7 ORIGINATED BY CAM  
 HWY 62 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY CAM  
 DATUM Geodetic DATE 2016.10.04 - 2016.10.04 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					
								UNCONFINED + FIELD VANE					
								WATER CONTENT (%)					
133.1						20 40 60 80 100	20 40 60						
0.0	75 mm ROOTMAT												
0.1	Silty SAND (SM) with gravel - frequent cobbles		1	SS	10								
132.5	Compact Brown												
0.6	CLAY (Cl) Stiff Brown		2	SS	13								
			3	SS	12								
130.8													
2.3	SILT (ML) Compact Grey		4	SS	14								
130.2													
2.9	Sandy SILT (ML) to Silty SAND (SM) Loose to compact Brown		5	SS	9								
			6	SS	12								
			7	SS	6								
			8	SS	10								
			9	SS	6								
124.2	Silty SAND (SM) with gravel, TILL - frequent cobbles and occasional boulders Dense Brown		10	SS	46								
8.8													

Continued Next Page

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

20  
15  
10

(%) STRAIN AT FAILURE

ONTMT4S RAWDON CREEK BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 25/4/17

# RECORD OF BOREHOLE No 16-2

2 OF 2

METRIC

GWP# 4044-10-00 LOCATION Highway 62 Rawdon Creek Bridge, MTM Zone 9: N 4 911 368.8 E 226 849.7 ORIGINATED BY CAM  
 HWY 62 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY CAM  
 DATUM Geodetic DATE 2016.10.04 - 2016.10.04 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					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
								WATER CONTENT (%)					
	Continued From Previous Page						20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT			
122.9								W P	W	W L			
10.2	<b>BEDROCK</b> Limestone Slightly weathered Moderate bedding Strong to very strong strength Good to excellent quality Grey		1	HQ								RUN #1 TCR=100% SCR=100% RQD=88%	
			2	HQ								RUN #2 TCR=100% SCR=100% RQD=100%	
			3	HQ								RUN #3 TCR=100% SCR=100% RQD=100%	
119.5													
13.6	End of Borehole Groundwater level measured in monitoring well at 1.5 m BGS (elev. 131.6 m) on 2017/04/10												

ONTMT4S RAWDON CREEK BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 25/4/17

# RECORD OF BOREHOLE No 16-3

1 OF 2

METRIC

GWP# 4044-10-00 LOCATION Highway 62 Rawdon Creek Bridge, MTM Zone 9: N 4 911 384.3 E 226 831.1 ORIGINATED BY CAM  
HWY 62 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY CAM  
DATUM Geodetic DATE 2016.10.03 - 2016.10.03 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  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W P      W      W L				WATER CONTENT (%)
								○ UNCONFINED      + FIELD VANE						
131.4								20 40 60 80 100						
0.0	150 mm ROOTMAT							20 40 60 80 100						
0.2	Silty SAND (SM) with gravel Very loose to compact Brown		1	SS	3		131							
			2	SS	15		130							
129.7														
1.7	Sandy SILT (ML) to Silty SAND (SM) Loose to compact Brown		3	SS	11		129							
			4	SS	16		128							
			5	SS	10		127						0 28 69 3	
			6	SS	10		126						0 43 57 (SI+CL)	
			7	SS	5		125							
							124							
							123							
125.0			8	SS	100/ 200mm		122							
6.4	Silty SAND (SM) with gravel, TILL - frequent cobbles and occasional boulders Very dense Brown		9	SS	100/ 175mm		121							
							120							
							119							
							118							
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							4							
							3							
							2							
							1							
							0							

Continued Next Page



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

ONTMT4S RAWDON CREEK BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 25/4/17

## METRIC

[illegible]

## METRIC

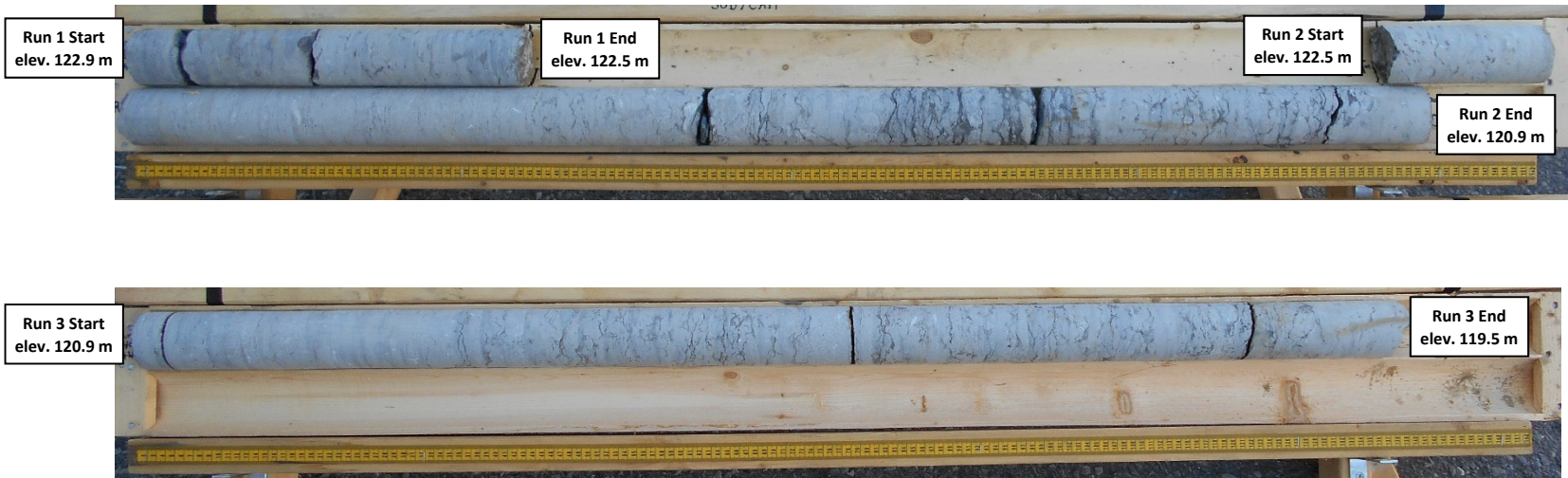
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT	UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20    40    60    80    100	W P                  W                  W L			20    40    60
								SHEAR STRENGTH kPa	WATER CONTENT (%)			
131.4												
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
							20    40    60    80    100					

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 (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE					W <sub>P</sub>	W	W <sub>L</sub>
								20 40 60 80 100						20 40 60		
131.4																
0.9																
0.1	50 mm ROOTMAT															
	Silty SAND (SM) with gravel Very loose to compact Brown to grey		1	SS	4											
			2	SS	3											
	- slight hydro-carbon odour in sample SS3		3	SS	22											
129.1																
2.3	Sandy SILT (ML) to Silty SAND (SM) Loose to compact Brown		4	SS	13											
			5	SS	9											
			6	SS	2											
			7	SS	6											
126.2																
5.2	Silty SAND (SM) with gravel, TILL - frequent cobbles and occasional boulders Very dense Brown		8	SS	77											
			9	SS	100/											
124.5																
6.9	End of Borehole Split Spoon refusal on inferred boulder Groundwater level was measured in the open borehole at 3.9 m BGS (elev. 127.5 m)				50mm											

ONTMT4S RAWDON CREEK BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 25/4/17

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

**Borehole 16-2**  
**Run 1 to 3 (of 3)**  
**Elevation 122.9 m to 119.5 m**



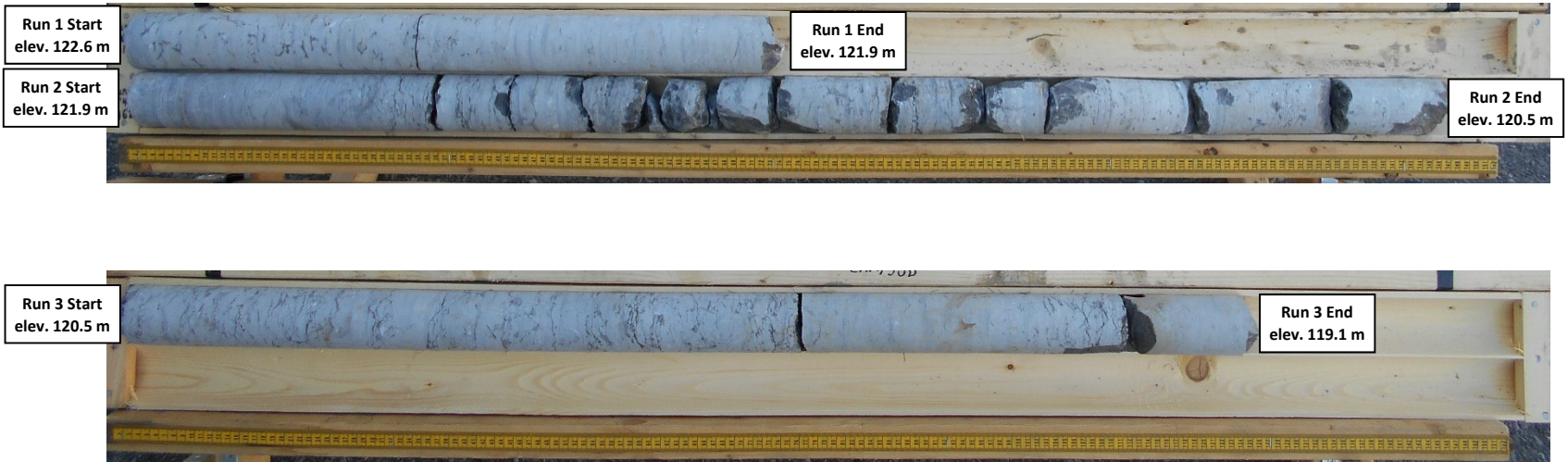
**THURBER** ENGINEERING LTD.

**Foundation Investigation**  
**Highway 62 – Rawdon Creek Bridge**  
**Site 11-134**  
**Township of Huntingdon, Ontario**

**GWP: 4044-10-00**

**Project No.: 14392**

**Borehole 16-3**  
**Run 1 to 3 (of 3)**  
**Elevation 122.6 m to 119.1 m**



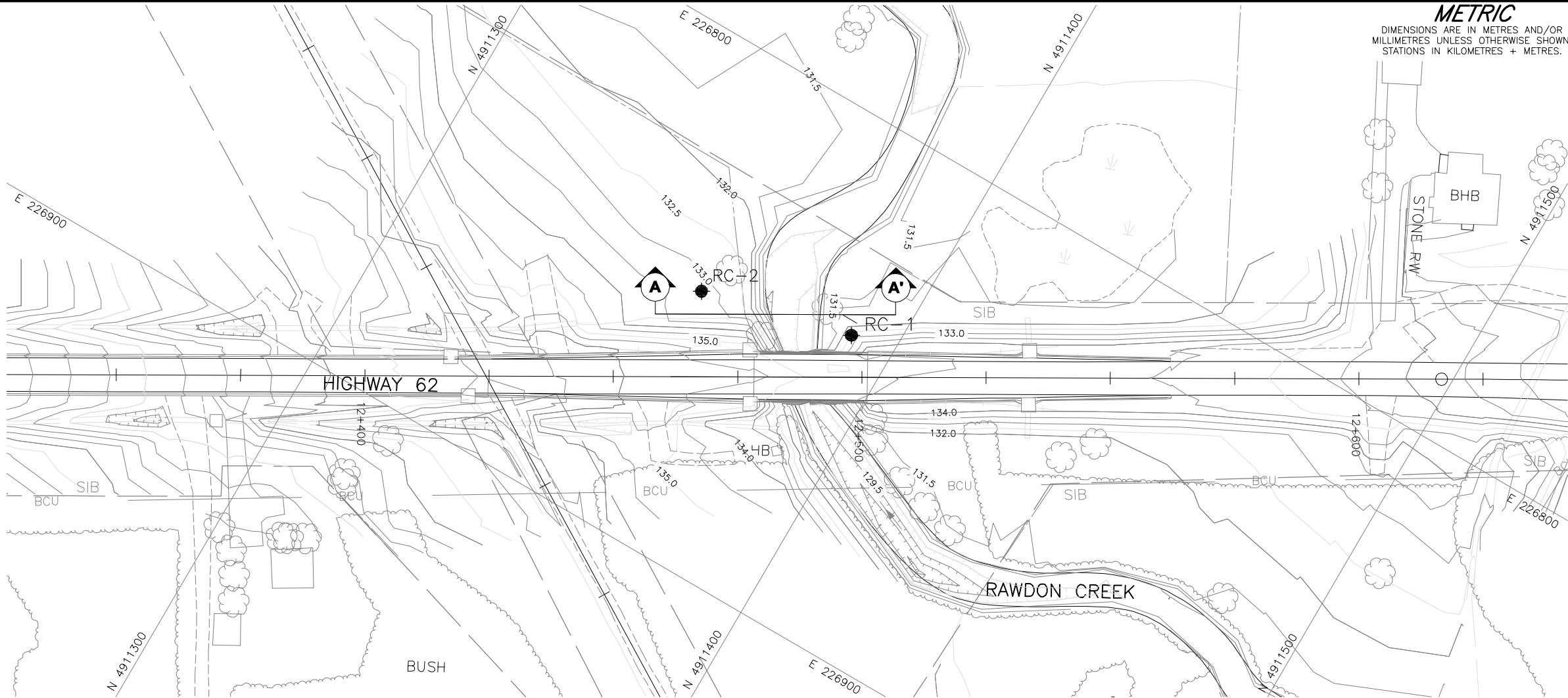
**Foundation Investigation**  
**Highway 62 – Rawdon Creek Bridge**  
**Site 11-134**  
**Township of Huntingdon, Ontario**

**GWP: 4044-10-00**  
**Project No.: 14392**

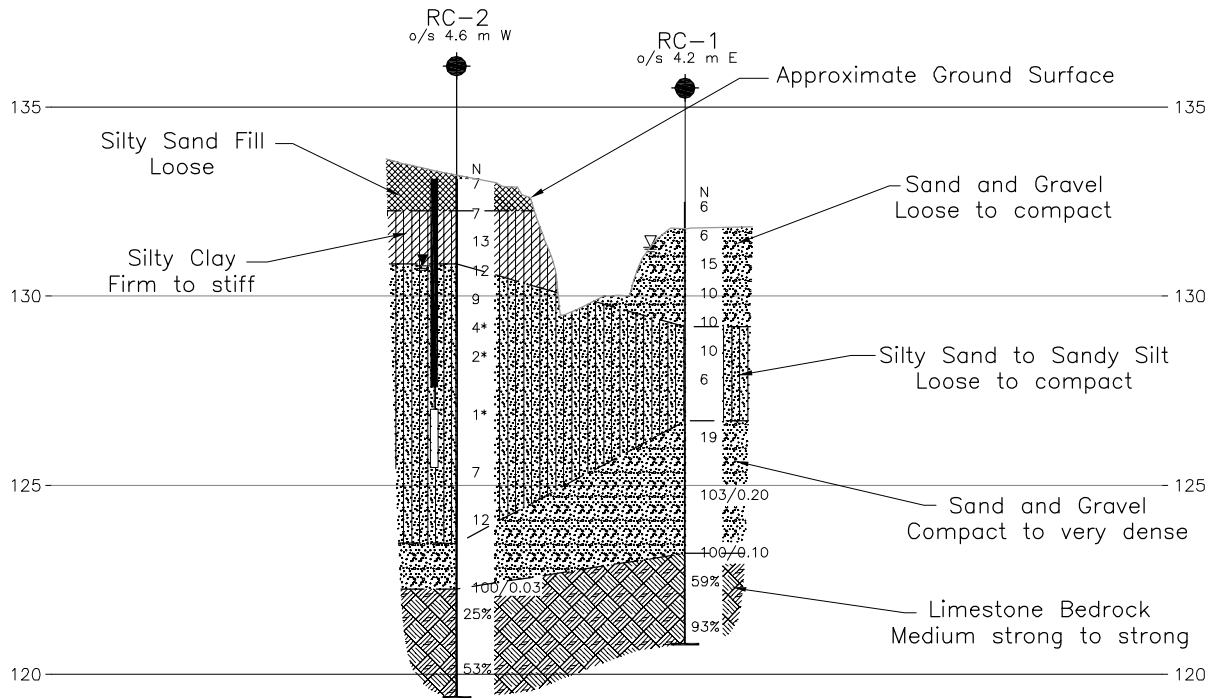
## **APPENDIX C**

### **PRELIMINARY INVESTIGATION BOREHOLE LOCATIONS AND SOIL STRATA DRAWINGS RECORD OF BOREHOLE SHEETS**

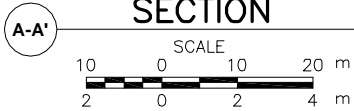




PLAN



SECTION



**METRIC**  
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.  
GWP No. 4044-10-00



HIGHWAY 62  
RAWDON CREEK BRIDGE  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN



SCALE  
0 2 4 km

LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on November 29, 2012
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
RC-1	132.5	4911391.0	226840.6
RC-2	133.1	4911360.5	226848.2

**DRAFT**

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the preliminary design configuration as shown elsewhere in the Preliminary Design Report.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file no.s ACAD-Contours\_OG-Hwy62.dwg and ACAD-X-Base\_Hwy62.dwg, received December 17, 2012.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 62		PROJECT NO. 12-1111-0021	DIST. Eastern
SUBM'D. BM	CHKD. LCC	DATE: 2/5/2013	SITE: 11-134
DRAWN: JFC	CHKD. PKS	APPD. LCC	DWG. 1

## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I. GENERAL

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

<b>(a)</b>	<b>Index Properties</b>
$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

### (a) Index Properties (continued)

w	water content
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	plasticity index $= (w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index $= (w - w_p) / I_p$
$I_C$	consistency index $= (w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_{\alpha}$	secondary compression index
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation (vertical direction)
$c_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction $= \tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_l$	sensitivity

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1  
2

$\tau = c' + \sigma' \tan \phi'$   
shear strength = (compressive strength)/2

## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils Consistency

	$c_u, s_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

#### Per cent by Weight Modifier

0 to 5	Trace
5 to 12	Trace to Some (or Little)
12 to 20	Some
20 to 30	(ey) or (y)
over 30	And (non-cohesive (cohesionless)) or With (cohesive)

#### Example

Trace sand  
Trace to some sand  
Some sand  
Sandy  
Sand and Gravel  
Silty Clay with sand / Clayey Silt with sand

---

## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

---

### WEATHERINGS STATE

**Fresh:** no visible sign of weathering

**Faintly weathered:** weathering limited to the surface of major discontinuities.

**Slightly weathered:** penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

**Moderately weathered:** weathering extends throughout the rock mass but the rock material is not friable.

**Highly weathered:** weathering extends throughout rock mass and the rock material is partly friable.

**Completely weathered:** rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

### BEDDING THICKNESS

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

### JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

### GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: \* Grains greater than 60 microns diameter are visible to the naked eye.

### CORE CONDITION

#### Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

#### Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

#### Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

### DISCONTINUITY DATA

#### Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

#### Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.






#### Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

#### Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	



PROJECT 12-1111-0021		RECORD OF BOREHOLE No RC-1		SHEET 1 OF 2		METRIC												
W.P. 4044-10-00		LOCATION N 4911391.0 ; E 226840.6		ORIGINATED BY BM														
DIST Eastern HWY 35		BOREHOLE TYPE CME-55, 108 mm I.D. Continuous Flight Hollow Stem Auger		COMPILED BY MAS														
DATUM Geodetic		DATE November 13, 2012		CHECKED BY LCC														
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 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						WATER CONTENT (%)				
132.5 0.0	GROUND SURFACE SAND and GRAVEL, trace clay, trace to some silt, containing cobbles and rootlets Loose to compact Brown to grey Moist becoming wet at approximately 1.2 m depth		1	SS	6		20	40	60	80	100							
			2	SS	6													
			3	SS	15													
			4	SS	10													
129.2 3.3	Silty SAND to Sandy SILT Compact to loose Grey Wet		5	SS	10													
			6	SS	10													
			7	SS	6													
126.7 5.8	SAND and GRAVEL, trace silt, trace clay, containing cobbles and boulders Compact to very dense Grey Wet		8	SS	19													
			9	SS	103/0.20													
123.2 9.3	Limestone (BEDROCK)  Bedrock cored from 9.3 m to 11.7 m  For bedrock coring details, refer to Record of Drillhole RC-1		10	SS	100/0.10													
			1	RC	REC 66%													
			2	RC	REC 98%													
120.8 11.7	END OF BOREHOLE  NOTE:  1. Water encountered at a depth of approximately 1.2 m (Elev. 131.3 m) during drilling.																	

GTA-MTO 001 12-1111-0021.GPJ GAL-GTA.GDT 4/28/14 DD

PROJECT: 12-1111-0021

## RECORD OF DRILLHOLE: RC-1

SHEET 2 OF 2

LOCATION: N 4911391.0 ; E 226840.6

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: Strong Soil Search

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH	COLOUR	% RETURN	RECOVERY		FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -C/ AVG.	NOTES WATER LEVELS INSTRUMENTATION			
										TOTAL CORE %	SOLID CORE %		R.O.D. %	DIP w.r.t. CORE AXIS		TYPE AND SURFACE DESCRIPTION					Jr	Ja	Jn
										JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage		PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock								
		Continued from Record of Borehole RC-1		123.20																			
10		Limestone (BEDROCK) Slightly weathered to fresh Thinly laminated Grey Medium strong to very strong		9.30																			
11																					(Axial)		
12		END OF DRILLHOLE		120.80																			
13																							
14																							
15																							
16																							
17																							
18																							
19																							

DEPTH SCALE

1 : 50



LOGGED: BM

CHECKED: LCC

GTA-RCK 004 12-1111-0021.GPJ GAL-MISS.GDT 4/28/14 DD

<b>PROJECT</b> 12-1111-0021		<b>RECORD OF BOREHOLE No RC-2</b>		SHEET 1 OF 3	<b>METRIC</b>
W.P. 4044-10-00		LOCATION N 4911360.5 ; E 226848.2		ORIGINATED BY BM	
DIST Eastern HWY 35		BOREHOLE TYPE CME-55, 108 mm I.D. Continuous Flight Hollow Stem Auger		COMPILED BY MAS	
DATUM Geodetic		DATE November 13, 2012		CHECKED BY LCC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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 (%)				
								UNCONFINED ● QUICK TRIAXIAL	FIELD VANE + REMOULDED X	W <sub>p</sub>	W	W <sub>L</sub>		
133.1	GROUND SURFACE													
0.0	Silty sand, trace clay, trace gravel (FILL) Loose Brown Moist		1	SS	7									
132.3														
0.8	SILTY CLAY with sand, containing rootlets Firm to stiff Brown Moist		2	SS	7									0 25 45 30
			3	SS	13									
130.9														
2.2	Silty SAND to Sandy SILT Loose to compact Brown becoming grey at approximately 3.7 m depth Wet		4	SS	12									
			5	SS	9									0 24 76 0
			6	SS	4*									
			7	SS	2*									
			8	SS	1*									0 72 27 1
			9	SS	7									
123.5			10	SS	12									
9.6	SAND and GRAVEL, containing cobbles and boulders Very dense Grey Wet													
122.3			11	SS	00/0.00									
10.8	Limestone (BEDROCK)  Bedrock cored from 10.8 m to 13.7 m  For bedrock coring details, refer to Record of Drillhole RC-2		1	RC	REC 58%									RQD = 25%
			2	RC	REC 84%									RQD = 53%
119.4														
13.7														

GTA-MTO 001 12-1111-0021.GPJ GAL-GTA.GDT 4/28/14 DO

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>12-1111-0021</u>		<b>RECORD OF BOREHOLE No RC-2</b>		SHEET 2 OF 3		<b>METRIC</b>	
W.P. <u>4044-10-00</u>		LOCATION <u>N 4911360.5 ; E 226848.2</u>		ORIGINATED BY <u>BM</u>			
DIST <u>Eastern</u> HWY <u>35</u>		BOREHOLE TYPE <u>CME-55, 108 mm I.D. Continuous Flight Hollow Stem Auger</u>		COMPILED BY <u>MAS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 13, 2012</u>		CHECKED BY <u>LCC</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			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			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
	--- CONTINUED FROM PREVIOUS PAGE ---													
	END OF BOREHOLE													
	NOTES:  * SPT "N" values considered to be affected by sample disturbance due to groundwater inflow in borehole.  1. Water encountered at a depth of approximately 2.3 m (Elev. 130.8 m) during drilling.  2. Water level in piezometer measured as follows Date      Depth (m)      Elev. (m) 11/13/12      2.3      130.8 11/29/12      2.4      130.7													

GTA-MTO 001 12-1111-0021.GPJ GAL-GTA.GDT 4/28/14 DD



PROJECT: 12-1111-0021

## RECORD OF DRILLHOLE: RC-2

SHEET 3 OF 3

LOCATION: N 4911360.5 ;E 226848.2

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: Strong Soil Search

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE mm/min	FLUSH	COLOUR	% RETURN	RECOVERY				FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load		RMC -Q- AVG.	NOTES WATER LEVELS INSTRUMENTATION			
										TOTAL CORE %	SOLID CORE %	R.Q.D. %	B Angle		DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	K <sub>i</sub> cm/sec	1	2	3	4	5			6	7	8
										JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break		BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.													
		Continued from Record of Borehole RC-2		122.30																									
11		Limestone (BEDROCK) Slightly weathered to fresh Thinly laminated Grey Medium strong to very strong		10.80																					(Axial)				
12																													
13																									(Axial)				
14		END OF DRILLHOLE		119.40																									
15				13.70																									
16																													
17																													
18																													
19																													
20																													

DEPTH SCALE

1 : 50



LOGGED: BM

CHECKED: LCC

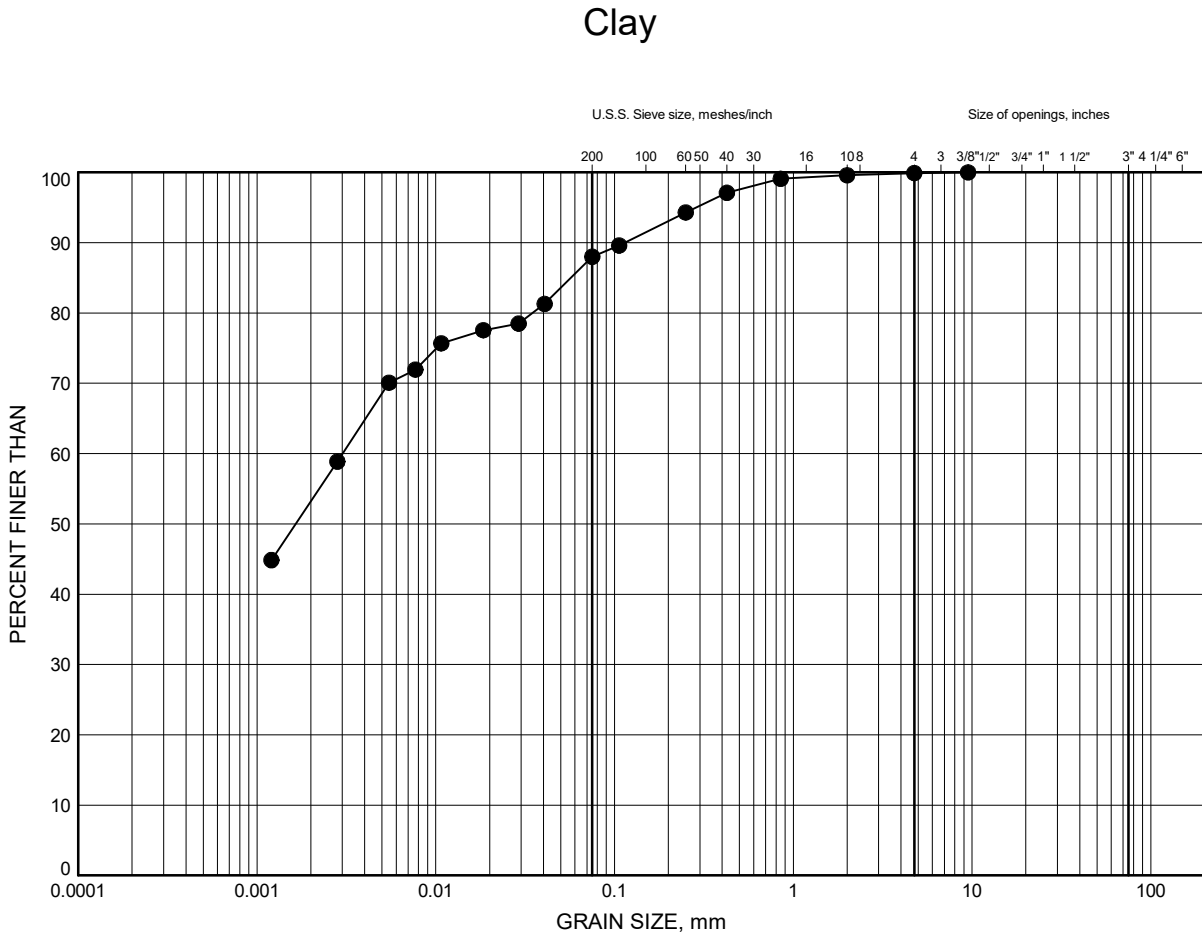
GTA-RCK 004 12-1111-0021.GPJ GAL-MISS.GDT 4/28/14 DD

**APPENDIX D**  
**LABORATORY TEST RESULTS**

Site 11-134 - Highway 62 Rawdon Creek Bridge

# GRAIN SIZE DISTRIBUTION

FIGURE 1



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-1	1.07	132.82

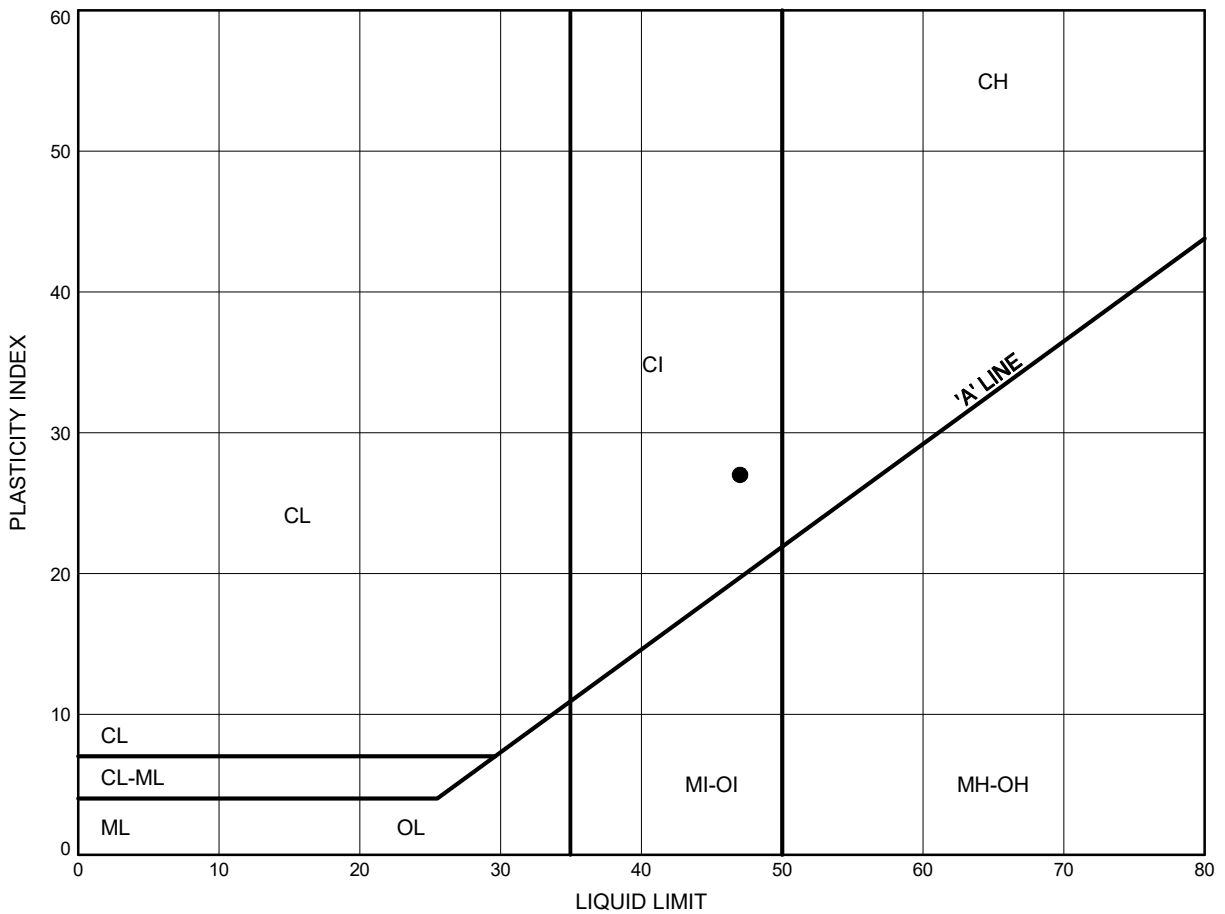
Date November 2016  
GWP# 4044-10-00



Prep'd KCP  
Chkd. PC

Site 11-134 - Highway 62 Rawdon Creek Bridge  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE 2



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-1	1.07	132.82

Date November 2016  
 GWP# 4044-10-00

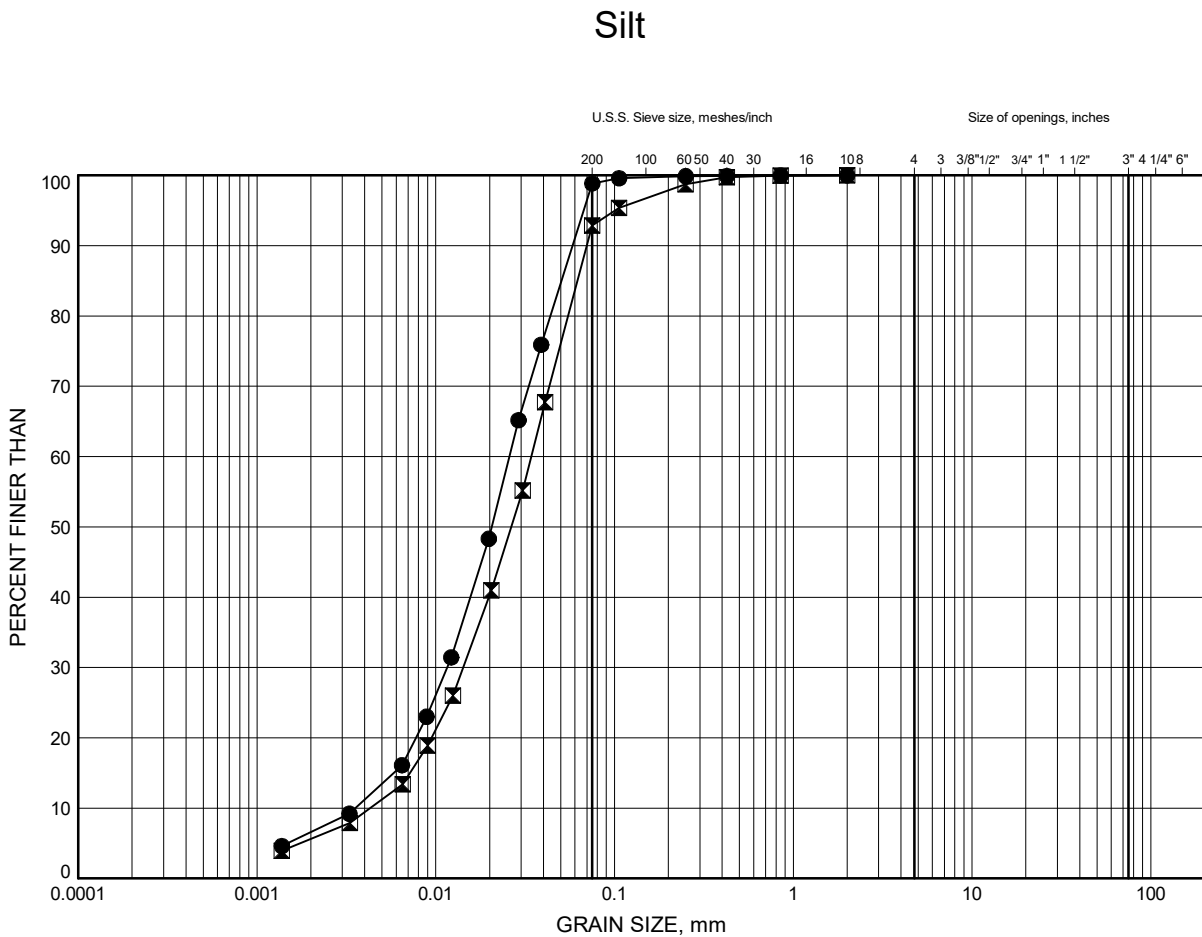


Prep'd KCP  
 Chkd. PC

Site 11-134 - Highway 62 Rawdon Creek Bridge

# GRAIN SIZE DISTRIBUTION

FIGURE 3



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-1	3.35	130.54
⊠	16-2	2.59	130.48

Date November 2016

GWP# 4044-10-00



Prep'd KCP

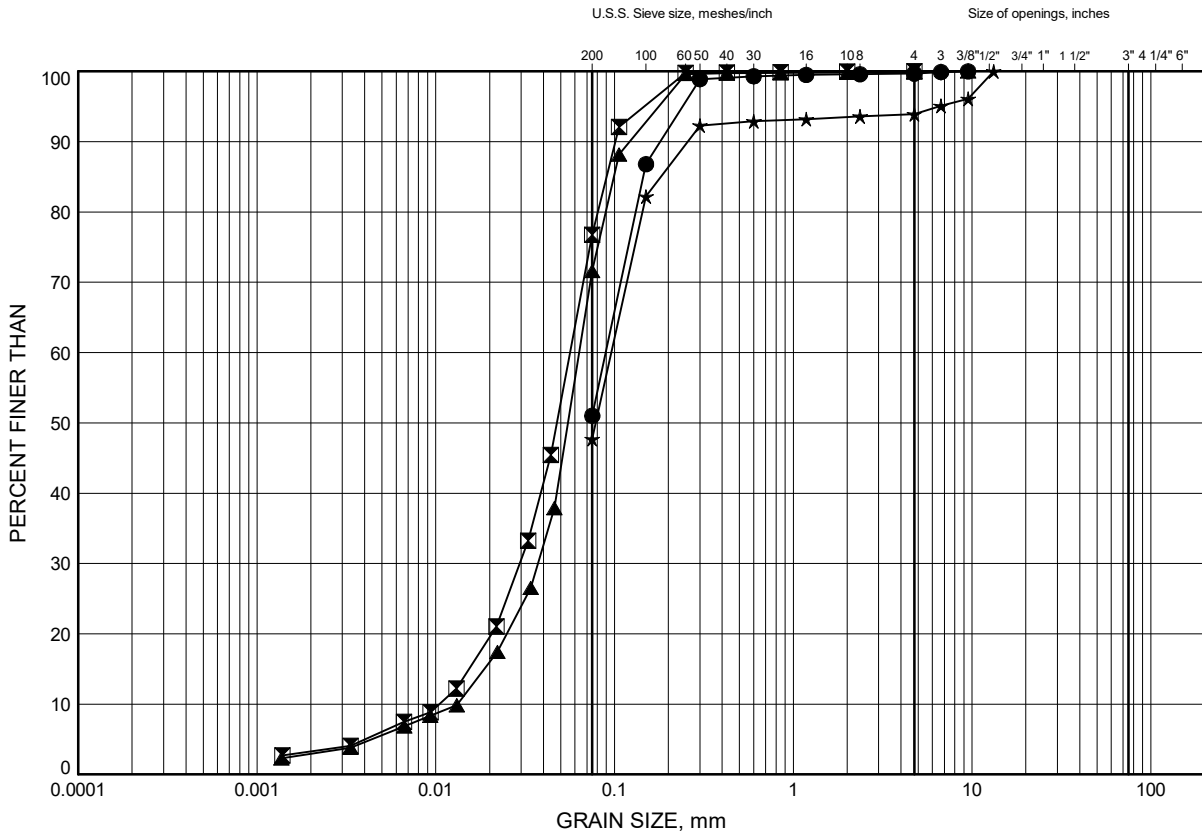
Chkd. PC

Site 11-134 - Highway 62 Rawdon Creek Bridge

# GRAIN SIZE DISTRIBUTION

FIGURE 4

## Sandy Silt to Silty Sand



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-1	7.92	125.96
⊠	16-2	4.88	128.19
▲	16-3	3.35	128.04
★	16-4	2.59	128.84

Date November 2016

GWP# 4044-10-00



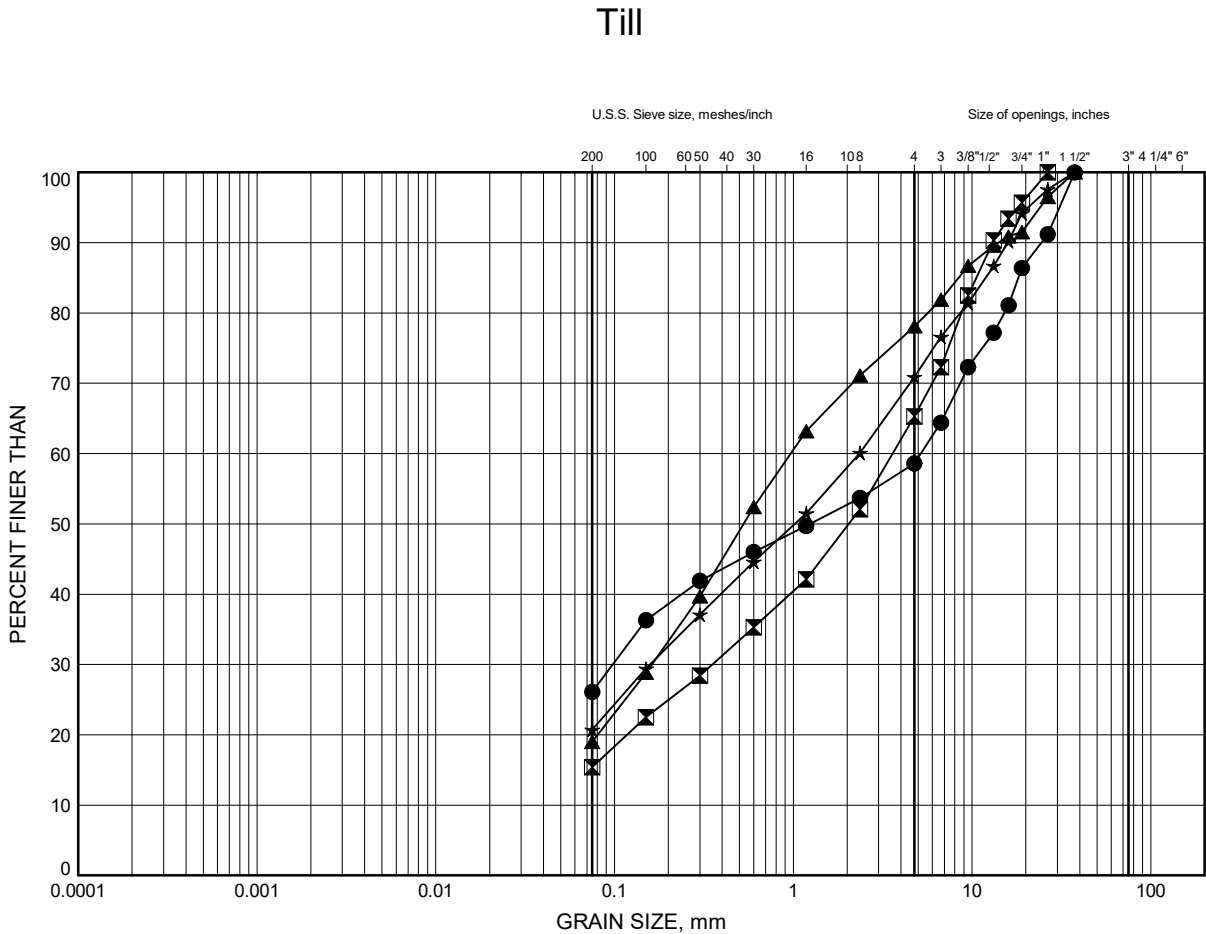
Prep'd KCP

Chkd. PC

Site 11-134 - Highway 62 Rawdon Creek Bridge

# GRAIN SIZE DISTRIBUTION

FIGURE 5



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-1	9.45	124.44
⊠	16-2	9.45	123.62
▲	16-3	7.92	123.47
★	16-4	6.40	125.03

Date November 2016  
GWP# 4044-10-00



Prep'd KCP  
Chkd. PC



## Determination of the Point Load Strength Index

**Project Name:** Site 11-134 - Highway 62 Rawdon Creek Bridge  
**Project No:** GWP: 4044-10-00  
**Borehole No:** 16-2  
**Core Size:** HQ

**Client:** MPCE-LEA  
**Date Sampled:** October 4, 2016  
**Date Tested:** November 22, 2016  
**Lab Technician:** CAM

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	10.6	D	12.48	63.5	85.0	78.4	Limestone	Strong
2	1	10.6	A	27.84	63.5	70.0	134.5	Limestone	Very Strong
3	2	11.6	A	21.37	63.5	70.0	103.2	Limestone	Very Strong
4	2	11.4	D	9.78	63.5	85.0	61.5	Limestone	Strong
5	3	12.3	A	27.96	63.5	70.0	135.1	Limestone	Very Strong
6	3	12.4	D	24.15	63.5	95.0	151.7	Limestone	Very Strong

\* It is ideal to perform axial test on core specimens with D/L ratio of  $1.1 \pm 0.1$

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

\* Diametral Test should have  $0.7 \times D$  on either side of test point.



**APPENDIX E**  
**SITE PHOTOGRAPHS**



**Figure 1: Looking north toward Rawdon Creek along the new alignment for Highway 62 from the south approach**



**Figure 2: Looking north along the new alignment for Highway 62 from Borehole 16-3**





**Figure 3: Looking south towards Rawdon Creek**



**Figure 4: Looking west from existing bridge towards proposed Rawdon Creek crossing**

## **APPENDIX F**

**GSC SEISMIC HAZARD CALCULATION  
SLOPE STABILITY ANALYSIS RESULTS  
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

November 16, 2015

Site: 44.3379 N, 77.4774 W User File Reference: Rawdon Creek

Requested by: , Thurber Engineering LTD.

**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>	<b>PGA (g)</b>	<b>PGV (m/s)</b>
0.121	0.160	<b>0.150</b>	0.125	<b>0.101</b>	<b>0.060</b>	<b>0.031</b>	<b>0.0082</b>	<b>0.0034</b>	<b>0.091</b>	<b>0.086</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)	0.016	0.046	0.071
Sa(0.1)	0.025	0.066	0.099
Sa(0.2)	0.026	0.067	0.097
Sa(0.3)	0.023	0.058	0.083
Sa(0.5)	0.018	0.047	0.068
Sa(1.0)	0.0092	0.027	0.040
Sa(2.0)	0.0038	0.013	0.020
Sa(5.0)	0.0008	0.0031	0.0048
Sa(10.0)	0.0005	0.0013	0.0021
PGA	0.014	0.037	0.057
PGV	0.011	0.035	0.054

## References

**National Building Code of Canada 2015 NRCC no. 58190;**  
**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**



Silt	20 kN/m <sup>3</sup>	0 kPa	27 °
Sandy Silt	20 kN/m <sup>3</sup>	0 kPa	32 °
Till	20 kN/m <sup>3</sup>	0 kPa	32 °
Clay	18 kN/m <sup>3</sup>	100 kPa	0 °
Bedrock			
Embankment fill	21 kN/m <sup>3</sup>	0 kPa	30 °

Reviewed By: \_\_\_\_\_  
 Tool Version: 8.15.5.11777  
 Last Solved Date: 11/24/2016, 10:07:02 AM  
 Directory: H:\Projects\10000 to 20000\14392 - Hwy 62 Rawdon Creek\Foundations\Analysis\slope\Rawdon Creek.gsz

Title: Replacement of Highway 62 overpass at Rawdon Creek  
Comments: Proposed Embankments  
Name: Seismic

Method: Morgenstern-Price, Half-Sine  
Minimum Slip Surface Depth: 1.52 m  
PWP Conditions Source: Piezometric Line  
Seismic: H1 0.059 V1 0  
Slip Surface Center: (-20.154918, 139.78796) w/ Radius: 3.6471904 m  
FoS Contours: 1.4 to 2.4, ++0.1

Silt	20 kN/m <sup>3</sup>	0 kPa	27 °
Sandy Silt	20 kN/m <sup>3</sup>	0 kPa	32 °
Till	20 kN/m <sup>3</sup>	0 kPa	32 °
Clay	18 kN/m <sup>3</sup>	100 kPa	0 °
Bedrock			
Embankment fill	21 kN/m <sup>3</sup>	0 kPa	30 °

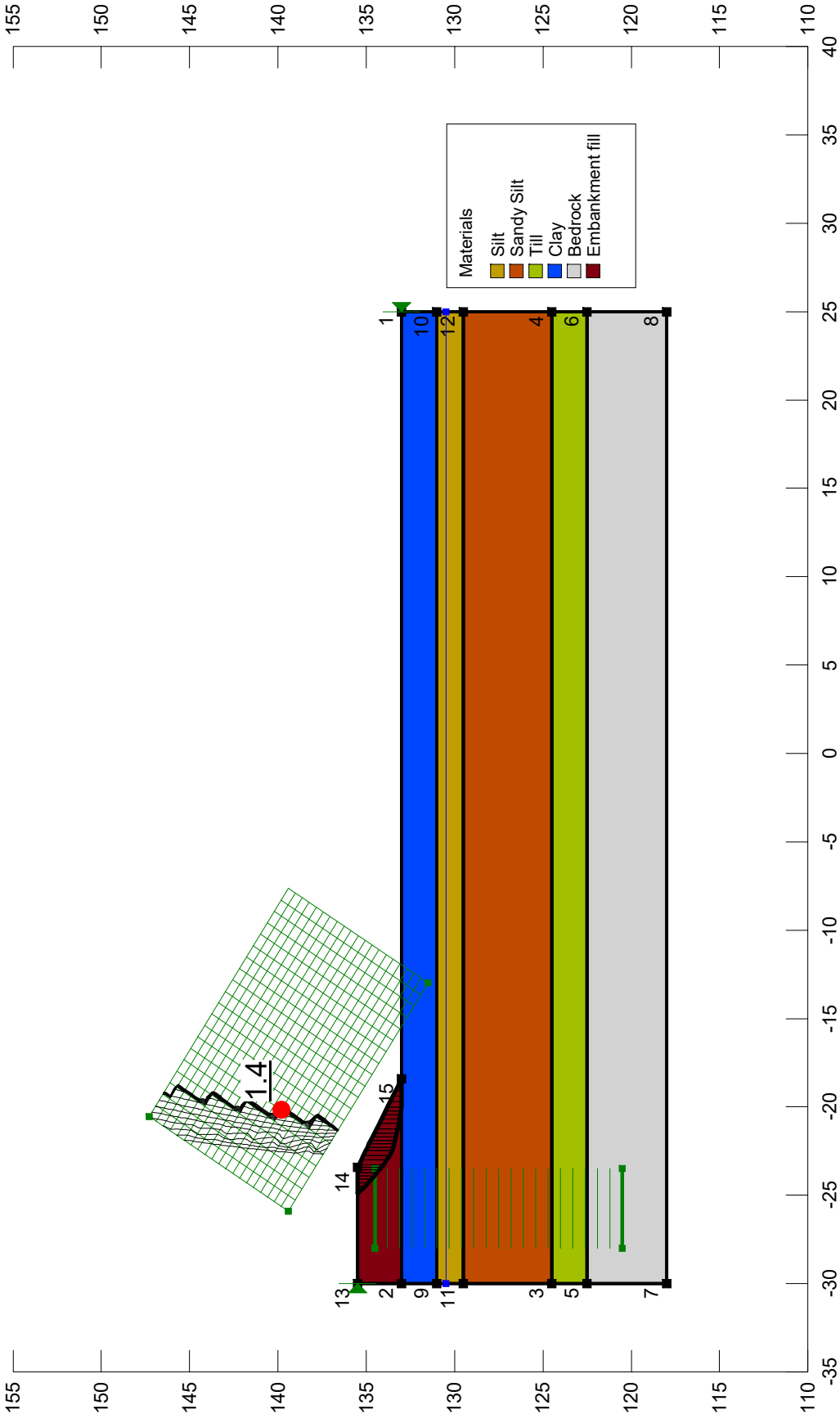


Figure 2

## **NON-STANDARD SPECIAL PROVISIONS**

### **NSSP – 903.07.02.01 PILE DRIVING REQUIREMENTS AND RESTRICTIONS**

Subsection 903.07.02.01 of OPSS.PROV 903 is amended by the addition of the following:

The soil stratum directly above the bedrock contains cobbles and boulders. Piles driven at this site may meet refusal on the boulders and cobbles within this layer.

Reference can be made to the Foundation Investigation Report Rawdon Creek Bridge Replacement, Highway 62 prepared by Thurber Engineering Ltd. for further details on likely subsurface conditions at the pile locations.

### **NSSP – VIBRATION MONITORING**

#### Scope

This special provision describes requirements for vibration monitoring during pile installation for the Rawdon Creek Bridge replacement, Site 11-134.

#### References

The subsurface conditions at the site are described in the following Foundation Investigation Report for G.W.P. 4044-10-00: Foundation Investigation Report, Rawdon Creek Bridge Replacement, Highway 62 prepared by Thurber Engineering Ltd.

#### Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to this contract. The QVE shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificates of conformance.

#### Submission Requirements

The Contractor/QVE shall submit details of the vibration monitoring plan to the Contract Administrator for review at least three (3) weeks in advance of commencement of the work activity requiring vibration monitoring as specified herein. All submissions shall bear the signature and seal of Professional Engineer licensed to practice in the Province of Ontario. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
- Qualifications of vibration monitoring specialist
- Details regarding proposed instrumentation.
- Proposed location of instruments on the existing Rawdon Creek Bridge.
- Method of monitoring, proposed frequency of readings and proposed frequency of submission of readings to Contract Administrator.
- Action plan to be taken to adjust bedrock excavation methods or deep foundation installation methods if readings show vibrations exceeding tolerable levels.



### Monitoring

The vibration monitoring equipment shall be placed on the existing Rawdon Creek Bridge. The Contractor shall take readings on the existing structure throughout the pile driving operations, as applicable at this site and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on the existing bridge structures shall not exceed 100 mm/s (peak particle velocity).

If the readings are not within the limits stated above, the Contractor must alter deep foundation installation procedures until the vibrations at the existing bridge structures, commercial/industrial buildings and/or rail tracks, as applicable, are within acceptable levels.

### Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

## **LIST OF REFERENCED SPECIFICATIONS**

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, And Granular Sheeting
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