

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
PAYS PLAT CREEK BRIDGE REPLACEMENT  
HIGHWAY 17, DISTRICT OF THUNDER BAY, ONTARIO  
G.W.P. 6071-09-00, SITE #48C-19**

**Geocres Number: 42D-37**

**Report to**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation conducted at the site of the proposed replacement of the Pays Plat Creek Bridge on Highway 17, in the Thunder Bay District, Ontario.

The purpose of the investigation was to explore the subsurface conditions at the site, and based on the data obtained, to provide a borehole location plan, record of borehole sheets, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited (MMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0011.

**2 SITE DESCRIPTION**

The bridge site is located on Highway 17 approximately 55 km southeast of Nipigon or 6.0 km northwest of Rosspoint. The existing bridge is a single span structure with a total span length of 19.8 m between abutments and a width of 11.6 m, as indicated on the contract drawings prepared for rehabilitation of the bridge in July 1996. The existing approach embankments vary in height from 4 m at the abutments to 2 m in a distance of approximately 15 m away from the abutments.

The Pays Plat Creek is a tributary to the Pays Plat River, which flows southerly into the Lake Superior. Both creek and the river flow through a broad flat valley. The land surrounding the site is treed with residences located to the east of the bridge. At this location, Highway 17 travels in a predominantly northwest to southeast direction along the north edge of Lake Superior. Photographs of the bridge and surrounding area are presented in Appendix C.

The site lies within the physiographic region known as the Wawa Subprovince of the Superior Province of the Canadian Shield. Based on Ontario Geological Survey (OGS) Map 2518, titled "Surficial Geology of Northern Ontario", dated 1987, the site is located in an area of "the mainly glaciofluvial deposits, including shallow water, glaciolacustrine and glaciomarine deposits". Based on OGS Map

2545, titled “Bedrock Geology of Ontario”, dated 1991, the bedrock is of the Archean age and consists of intrusive rocks, mainly massive to foliated granodiorite and granite.

### 3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing at this site were carried between June 23 and July 4, 2013. A total of four boreholes, denoted as PPC-01 to PPC-04 were advanced to depths ranging from 15.8 m to 30.7 m below the existing highway embankment. Two dynamic cone penetration tests were advanced to 2.4 m and 19.2 m depth, respectively, to supplement the sampled borehole information. Details of the borehole locations, drilling depths and completion details are summarized in Table 3.1 below.

**Table 3.1 – Details of Boreholes**

<b>Location</b>	<b>Boreholes</b>	<b>Drilling and Coring Depth/ Base of Hole Elevation (m)</b>	<b>Completion Details</b>
West Approach	PPC-01	15.8 / 171.0	Borehole backfilled with bentonite holeplug to 0.6 m, concrete mix to 0.1 m then asphalt to surface.
West Abutment	PPC-02	30.7 / 156.1	Standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 3 m slotted screen installed. After final water level reading, piezometer was decommissioned.
East Abutment	PPC-03	27.6 / 159.2	Standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 3 m slotted screen installed. After final water level reading, piezometer was decommissioned.
East Approach	PPC-04	15.8 / 171.0	Borehole backfilled with bentonite holeplug and cuttings to 0.6 m, concrete to 0.1 m then asphalt cold patch to surface.

The locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing included in Appendix F.

All boreholes were advanced using a CME55 truck-mounted drill rig in combination with hollow stem augers and NW casing/coring methods. Samples of the encountered soils were obtained from the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths using MTO Standard “N” size vane and a calibrated torque wrench.

A member of Thurber’s technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber’s laboratory for further examination and testing. The ground surface elevations at the borehole locations were obtained from the drawings provided by MMM.

Groundwater conditions in the open boreholes were observed during the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipe with a slotted screen were installed in Boreholes PPC-02 and PPC-03. Following the final water level reading, the piezometers were decommissioned in general accordance with MOE Regulation 903.

#### **4 LABORATORY TESTING**

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets included in Appendix A. Selected samples were also subjected to grain size analysis and Atterberg Limits testing, and the results of this testing program are summarized on the Record of Borehole sheets in Appendix A, and are shown on the figures included in Appendix B.

#### **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

Reference is made to the Record of Borehole sheets in Appendix A consisting of details of the encountered soils. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations during drilling; therefore they represent transitions between soil types rather than exact geological boundaries. The subsurface conditions may vary between and beyond the borehole locations. The model of the soil stratigraphy is illustrated on the “Borehole Locations and Soil Strata” drawing in Appendix F.

An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

The subsurface stratigraphy below the existing embankment fill encountered at the site generally consists of cohesionless glaciofluvial and cohesive glaciolacustrine deposits to the depths investigated. The embankment fill is underlain by a loose to compact cohesionless deposit consisting of sand to sandy silt, which grades at approximately 8.7 m depth to a varved silty clay and clay. This varved silty clay/clay deposit is typically soft to firm in consistency and extends to as much as 20.4 m depth. The compressible layer is underlain by a lower cohesionless deposit grading with depth from silt to gravelly sand. The cohesionless deposit is compact to between 24.4 m and 26.8 m depths, below which it becomes coarser and very dense to depths investigated in the boreholes. Descriptions of the individual strata are presented below.

##### **5.1 Asphalt and Concrete**

Asphalt pavement was encountered in all boreholes and dynamic cone penetration holes. The thickness of the asphalt ranged from 65 to 125 mm. Boreholes PPC-02 and PPC-03 were advanced through the approach slabs and encountered 325 and 430 mm thick concrete.

##### **5.2 Embankment Fill**

Embankment fill was encountered below the asphalt and the approach slabs in the boreholes. In Boreholes PPC-02 and PPC-03, located in the immediate vicinity of the abutments, the fill extended to 4.1 m depth (Elev. 182.7), and in Boreholes PPC-01 and PPC-04, located

approximately 15 m away from the abutments, the fill extended to 2.2 m depth (Elev. 184.6). The thickness of the fill ranged from 2.1 m to 3.6 m.

The fill contains various proportions of sand and gravel and in general, the material can be classified as sand to sand and gravel. Varying content of fine fractions (silt and clay) and occasional cobbles were observed in the fill. The lower zones of the fill in Boreholes PPC-02 and PPC-03 were typically coarser.

SPT 'N' values recorded in the embankment fill ranged from 9 to 89 blows per 0.3 m penetration, indicating a loose to very dense relative density. The higher SPT 'N' values are probably indicative of the presence of cobbles.

Moisture contents of the fill materials ranged from 4 to 22%.

The results of grain size analyses conducted on fill samples are provided on the Record of Borehole sheets in Appendix A, and are illustrated in Figure B1 of Appendix B. The results of grain size analyses for fill materials are summarized below:

Gravel %	0 to 59
Sand %	39 to 97
Silt and Clay %	2 to 9

### 5.3 Upper Sand to Sandy Silt

A deposit of sand to sandy silt was encountered beneath the fill materials in all boreholes. The thickness of the deposit ranged from 4.6 m to 6.5 m with the lower boundary of the deposit at 8.7 m depth (Elev. 178.1). The composition of the deposit varies with depth and across the site.

SPT 'N' values recorded in this deposit varied between 3 and 19 blows per 0.3 m of penetration indicating a loose to compact relative density. The values of 14 to 19 blows per 0.3 m of penetration were obtained within the sandier zones of the deposit in Boreholes PPC-01 and PPC-04.

The results of grain size analyses conducted on samples of the upper sand to sandy silt are provided on the Record of Borehole sheets in Appendix A, and are illustrated in Figure B2 and B3 of Appendix B. The results of the grain size analyses are summarized below:

	Sand and Silt /Sandy Silt	Sand
Gravel %	0	0
Sand %	20 to 38	82 to 86
Silt %	54 to 65	14 to 18
Clay %	8 to 15	

Natural moisture contents of the deposit ranged from 22 to 48%.

#### 5.4 Varved Silty Clay to Clay

A layer of grey, varved silty clay to clay underlies the upper cohesionless deposit in all boreholes. The deposit, where fully penetrated, was between 9.0 m and 11.7 m thick with the lower boundary encountered between 17.7 m and 20.4 m depth (Elev. 169.1 and 166.4). Boreholes PPC-01 and PPC-04 were terminated in this deposit at 15.8 m depth (Elev. 171.0). The upper zone, approximately 1.5 m to 2 m thick, reflects transition from the upper sand and silt deposit to the varved silty clay/clay, and contains more silt.

SPT 'N' values recorded in the silty clay/clay varied between zero blows per 0.3 m penetration (Weight of Rod to Weight of Hammer) to 5 blows per 0.3 m of penetration. Field vane shear tests (VST) measured undrained shear strengths ranging from 21 to 35 kPa. Based on the SPT and VST data, the consistency of the deposit varied from soft to firm.

The sensitivity of the deposit, calculated as a ratio of undisturbed strength to remoulded strength, ranged from 2 to 8, however typically being 3 to 4, suggesting that the silty clay/clay is of normal sensitivity.

The results of grain size analyses conducted on samples of the clay are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B4 of Appendix B. The results are summarized as follows:

Gravel %	0
Sand %	0
Silt %	17 to 34
Clay %	66 to 83

The results of the grain size analysis for one sample of the silty clay collected from the upper/transition zone of the deposit presented in Figure B5 and are as follows:

Gravel %	0
Sand %	6
Silt %	77
Clay %	17

The results of Atterberg Limits testing conducted on samples of the silty clay/clay are provided on the Record of Borehole sheets in Appendix A and are illustrated on the Plasticity Charts (Figures B8 and B9). The results indicated that the clay deposit has liquid limits ranging from 55% to 60% and plasticity indices ranging from 32% to 38%, indicating high plasticity of the deposit. In the upper zone, a liquid limit of 35% and plasticity index of 14% were obtained, indicating low to medium plasticity of the deposit.

Natural moisture contents of the silty clay ranged from 39% to 58%.



### 5.5 Lower Silt to Sandy Silt

In Boreholes PPC-02 and PPC-03, a silt with various proportions of sand and trace clay was encountered underlying the varved silty clay/clay below the depth of 20.4 m and 17.7 m. The deposit was grey and varied in thickness from 6.4 m to 6.7 m with the lower boundary between 26.8 m and 24.4 m depth (Elev. 160.0 and 162.4).

SPT 'N' values recorded in the silt layer ranged from 12 to 29 blows per 0.3 m penetration, indicating a compact relative density. Natural moisture contents were measured to be between 16 and 23%.

The results of grain size analyses conducted on samples of the deposit are provided on the Record of Borehole sheets in Appendix A and are plotted in Figure B6 of Appendix B. The results are presented below:

Gravel %	0
Sand %	1 to 26
Silt %	67 to 90
Clay %	7 to 9

### 5.6 Sand to Gravelly Sand

A layer of brown sand with varying proportion of gravel and trace to some silt was encountered in Boreholes PPC-02 and PPC-03 underlying the lower silt/sandy silt below depth of 26.8 m and 24.4 m (Elev. 160.0 and 162.4), respectively. The deposit was classified as sand to gravelly sand, and contained occasional cobbles and boulders. Rock coring technique was used to advance Borehole PPC-02 through the boulders below 28.3 m depth.

Boreholes PPC-02 and PPC-03 were advanced into this deposit for 3.9 m and 3.2 m, respectively, and terminated at 30.7 m and 27.6 m depth (Elev. 156.1 and 159.2).

The deposit was very dense as indicated by SPT 'N' values of more than 100 blows per 0.3 m penetration.

Natural moisture contents of 13% to 30% were measured on samples of this deposit.

The results of grain size analyses conducted on two samples of the sand/gravelly sand are provided on the Record of Borehole sheets in Appendix A and are plotted in Figure B7 of Appendix B. The results are summarized as follows:

Gravel %	0 to 27
Sand %	65 to 86
Silt & Clay %	8 to 14

## 5.7 Water Levels

Water levels in the boreholes were measured upon completion of drilling operations. Water was used during drilling and coring through boulders, therefore the measured water levels on completion of drilling may not reflect prevailing groundwater levels at the site.

Standpipe piezometers were installed in Boreholes PPC-02 and PPC-03 to monitor groundwater levels after drilling. The water levels measured in the open boreholes and in the piezometers are summarized in Table 5.1.

**Table 5.1: Water Level Measurements**

<b>Borehole Number</b>	<b>Date</b>	<b>Water Level (Depth/Elev.) in metres</b>	<b>Comments</b>
PPC -01	June 23, 2013	4.7 / 182.1	Water level in open borehole on completion of drilling. Borehole open to 15.8 m depth.
PPC-02	July 3, 2013	2.9 / 183.9	Water level in open borehole on completion of drilling.
	May 2, 2014	3.9 / 182.9	Water level in piezometer; piezometer sealed at 24.6 m depth.
PPC-03	July 4, 2013	0.7 / 186.1	Water level in open borehole on completion of drilling.
	May 2, 2014	3.8 / 183.0	Water level in piezometer; piezometer sealed at 21.5 m depth.
PPC-04	June 24, 2013	4.2 / 182.6	Water level in open borehole on completion of drilling. Borehole open to 15.8 m depth.

The preliminary General Arrangement drawing indicates the following water levels in Pays Plats Creek:

Elev. 182.6 – April 2011, and  
Elev. 183.8 - high water level.

The water level in the creek and groundwater levels are expected to fluctuate seasonally and are subject to precipitation patterns, and may vary from the levels presented above.

## 6 MISCELLANEOUS

Eastern Ontario Diamond Drilling supplied the drill rig and conducted the drilling, sampling and in-situ testing operations. A truck-mounted CME #55 drill rig was used for the duration of the investigation.

The drilling and sampling operations were supervised in the field by Ms. Eckie Siu and George Azzopardi of Thurber. Mr. Mark Farrant, P.Eng. directed the field operations.

The report was prepared by Ms. Anna Piascik, P.Eng., and reviewed by Mr. Murray Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects.

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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**7 GENERAL**

This report presents interpretation of the geotechnical data in the factual report and provides geotechnical recommendations to assist the design team in selecting and designing a suitable foundation system for the proposed replacement bridge.

At present, Highway 17 crosses the Pays Plat Creek on a single span structure with a span length of 19.8 m and a width of 11.6 m. Based on the archive design drawings dated 1958, each bridge abutment is supported on two rows of five 12 ½" (0.3 m) O.D. steel pipe piles driven closed-end to elevation 166.1 and filled with concrete. The piles in the front row are battered at 1H:3V. An additional pile is installed at the back end of each wingwall. The pile cut-off elevation is 182.3 at the abutments and 184.9 at the wingwalls.

The approach embankments vary in height from approximately 4 m at the abutments to 2 m in a distance of approximately 15 m away from the abutments. The existing road grade at the bridge is at approximate Elev. 186.8.

The preliminary General Arrangement drawing indicates that the replacement bridge will be a single span structure with a span of 27.0 m and width of 15.8 m. The new abutments will be installed behind the existing abutments. The existing road grade will be raised by up to 200 mm and the embankment will be widened to accommodate the wider bridge including a sidewalk on the north side.

The discussion and recommendations presented in this report are based on information provided by MMM and on the factual data obtained in the course of this investigation. The existing MTO foundation report (Geocres No. 42D-008, dated 1958) prepared for this bridge site was reviewed to supplement the factual information obtained during current investigation.

**8 STRUCTURE FOUNDATIONS**

In general, the site is underlain by cohesionless glaciofluvial and cohesive glaciolacustrine deposits extending to at least the 30.7 m depth of investigation. In summary, the embankment fill is underlain

by a loose to compact sand to sandy silt deposit, which transitions at approximately 8.7 m depth to a varved silty clay and clay. The varved silty clay/clay is typically soft to firm, between 9 m and 11.7 m thick and extends to as much as 20.4 m depth (Elev.166.4). This compressible layer is underlain by a lower cohesionless deposit, which grades with depth from silt to gravelly sand. The lower cohesionless deposit is compact to depths ranging from 24.4 m to 26.8 m, below which it becomes coarser and very dense to the depths of 27.6 m and 30.7 m (Elev.159.2 and 156.1) investigated in the boreholes.

The normal water level in Pays Plat Creek was indicated on the General Arrangement drawing at Elev. 182.6, and the high water level at Elev. 183.8. Groundwater levels in the piezometers installed in Boreholes PPC-02 and PPC-03 on each side of the creek were measured at 3.9 m and 3.8 m depth (Elev. 182.9 and 183.0).

Based on the subsurface conditions, several foundation options were considered for this bridge, namely

- spread footings placed on native soil or engineered fill,
- driven steel H-piles, and
- augered caissons.

A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix D.

Recommendations for design of the feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation alternative from a geotechnical perspective is recommended.

## **8.1 Spread Footings on Native Soil or Engineered Fill**

The use of spread footings to support the abutments is not recommended given the relatively low geotechnical resistance available in the native soils and the potential for large consolidation settlement in the underlying soft to firm cohesive deposits.

## **8.2 Driven H-Pile Foundations**

### **8.2.1 Axial Resistance**

The ground conditions at the site are considered to be suitable for the use of steel H-piles driven to practical refusal in the very dense sand/gravelly sand encountered below Elev. 160.0 on the west side of the creek and Elev. 162.4 on the east side of the creek.

The recommended geotechnical resistance and anticipated pile tip elevations for H-piles driven to refusal in the very dense sand/gravelly sand are presented in Table 8.1.

The actual pile tip elevations may vary during installation and will be controlled as described in Section 8.2.3 Pile Installation.

**Table 8.1 – Recommended Geotechnical Resistance and Reaction for HP310x110**

Foundation Element	Pile Tip Depth/Elevation (m)	Factored Geotechnical Resistance at ULS (kN) per pile	Geotechnical Reaction at SLS (kN) per pile
West Abutment	27.8 / 159.0	1600	1400
East Abutment	25.4 / 161.4	1600	1400

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used for any new fill through which the piles will be driven.

### 8.2.2 Pile Tips

Pile tip protection is recommended for driven H-piles to prevent pile damage when setting the piles in the very dense founding strata, where cobbles and boulders were encountered. The tips of all driven H-piles at the abutments should be fitted with pile tip protection from an approved manufacturer such as Skyline Steel, Titus Steel (Standard H-point) or similar.

### 8.2.3 Pile Installation

Pile installation should be in accordance with OPSS 903.

Pile driving should be controlled in accordance with Standard Drawing SS103-11 (Hiley Formula) and an ultimate pile resistance to be specified by the designer. The Hiley formula need not be used until the piles are within 2.0 m of the design pile tip elevation. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile. “R” should have a minimum value of twice the design load at ULS.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerance, a driving template or other means may be required to achieve the specified maximum deviation.

The new piles will be installed behind the existing bridge abutments and in close proximity to the existing pile caps. The upper loose to very loose cohesionless soils and underlying soft cohesive soils are sensitive to disturbance; therefore it is important that the existing piles and pile caps are left in place.

### 8.2.4 Pile Lateral Resistance

The geotechnical lateral resistance acting on a pile in cohesionless soils may be calculated using a value for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) as follows:

$$k_s = n_h z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \gamma' z K_p \quad (\text{kPa})$$

where

$z$  = depth of embedment of pile (m)

$D$  = pile width or diameter (m)

$n_h$  = coefficient related to soil density ( $\text{kN/m}^3$ )

$\gamma'$  = effective unit weight ( $\text{kN/m}^3$ )

$K_p$  = passive earth pressure coefficient

The geotechnical lateral resistance acting on a pile in cohesive soils may be calculated using a value for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) as follows:

$$k_s = 67 S_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 S_u \quad (\text{kPa})$$

Where

$S_u$  = undrained shear strength (kPa)

$D$  = pile width or diameter in metres

The above equations and recommended parameters in Table 8.2 below may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

**Table 8.2 – Soil Parameters for Lateral Pile Resistance**

Soil Unit	Elevation (m)		$\gamma'$ (kN/m <sup>3</sup> )*	$n_h$ (kN/m <sup>3</sup> )	$K_p$	$S_u$ (kPa)
	Top	Bottom				
West Abutment						
Sand and Gravel Fill	GS	182.7	20	4,000	3.0	-
Upper Sandy Silt	182.7	178.1	8	2,000	3.0	-
Silty Clay to Clay	178.1	166.4	7	-	-	25
Lower Sandy Silt	166.4	160.0	9	2,500	3.0	-
Sand	160.0	156.1	12	8,000	3.9	-
East Abutment						
Sand Fill	GS	182.7	20	4,000	3.0	-
Upper Sand and Silt	182.7	178.1	8	2,500	3.0	-
Silty Clay to Clay	178.1	169.1	7	-	-	22
Lower Silt	169.1	162.4	9	2,500	3.0	-
Gravelly Sand	162.4	159.2	12	8,000	3.9	-

\* Effective unit weight to be used in the evaluation of the lateral resistance of piles.

For analysis, the spring constant,  $K_s$ , may be obtained from the expression:

$$K_s = k_s L D \text{ (kN/m)},$$

where  $k_s$  = coefficient of horizontal subgrade reaction ( $\text{kN/m}^3$ ),  
 $D$  = pile width (m), and  
 $L$  = length (m) of the pile segment or element used in the analysis.

The ultimate lateral resistance,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} L D$ . This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

According to CHBDC Clause 6.8.7.1, Table C 6.4, the lateral resistance of an HP310x110 pile driven in those soil conditions should be limited to 110 kN at ULS and 40 kN at SLS.

The modulus of subgrade reaction may have to be reduced, 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	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

In the case of conventional abutments, i.e. not integral type, horizontal loads may be resisted by means of battered piles.

### 8.3 Downdrag

The weight of the additional fill that is proposed to raise the grade of the approach embankments will induce consolidation of the underlying varved silty clay/clay deposit. As a result, downdrag forces will develop along the length of abutment piles embedded in this deposit.

For design purposes, an unfactored downdrag load of 300 kN per pile should be used to evaluate the impact of downdrag on the abutment piles.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C6.8.4 to obtain a factored downdrag load. In accordance with Section 6.8.4 of the CHBDC and Clause C6.8.4 of the Commentary, in the structural design of a pile, the factored



downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile.

In geotechnical analysis of downdrag, the effect of live load should not be considered.

The location of the neutral plane for a pile or group of piles should be determined by using unfactored loads and unfactored geotechnical parameters.

#### **8.4 Caissons / Drilled Shafts**

Caisson installation at this site would extend through loose cohesionless soils below the groundwater table, a soft cohesive clay layer, and into very dense sand. Such means as drilling mud and/or a permanent liner would be required to support the caisson sidewalls and prevent boiling at the caisson base. Inspection of the caisson base to confirm the caisson capacity would not be possible in these conditions. The use of caissons is therefore not recommended and this alternative has not been developed herein.

#### **8.5 Recommended Foundation**

From a geotechnical perspective and based on the subsurface conditions, steel H-piles driven into the very dense sand/gravelly sand is the preferred foundation option at this site.

#### **8.6 Frost Cover**

The depth of frost penetration at this site is approximately 2.3 m. The base of pile caps, if employed, should be provided with a minimum of 2.3 m of earth cover as protection against frost action.

#### **8.7 Impact on Existing Foundations**

Piles for the replacement bridge will be driven adjacent to the existing piles supporting the existing abutments and wing walls. The new foundation units should be positioned to avoid encountering the existing foundations during the installation of new piles.

Archive documents indicate that the pipe piles supporting the existing bridge were driven to Elev. 166.1, through the upper loose sand and silt, soft to firm clay and 0.5 m to 3 m into the lower compact silt. In these conditions, driving of new piles in close proximity to the existing piles may result in disturbance and settlement of the existing bridge foundations. Therefore, it is recommended that a monitoring program (including establishment of adequate benchmarks outside the zone of potential influence and acquirement of baseline readings in advance of construction) be implemented for the duration of foundation construction to identify any movement of the existing structure.

Appropriate monitoring points and tolerable levels of movement should be specified by the structural designer. If movements exceed tolerable levels, the Contractor must be prepared to implement remedial measures for the existing bridge structure. Suggested wording for an

NSSP for monitoring of the existing structure during pile driving has been included in Appendix E.

## 9 SHEET PILE WALLS

The current design proposes the installation of steel sheet pile walls adjacent to the pile foundations in lieu of conventional abutment walls. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill. The alignment of the proposed sheet pile walls should be carefully selected to avoid existing foundations.

Lateral stability of the sheet pile walls should be checked by the wall designer using the parameters presented in Table 9.1. The coefficients of passive earth pressure ( $K_p$ ) are provided for horizontal ground surface in front of the sheet pile wall. For sloping ground in front of the sheet pile wall, the recommended values for the coefficients of passive earth pressure ( $K_p$ ) should be reduced. The possibility of material loss due to creek erosion in front of the sheet piles should also be considered in the check of the lateral earth pressure balance.

**Table 9.1 – Soil Parameters for Sheet Pile Analysis**

Foundation Element	Soil Unit	Elevation (m)		$\gamma'$ (kN/m <sup>3</sup> )	$K_a$	$K_o$	$K_p$
		Top	Bottom				
West Abutment	Fill	184.5 *	182.7	21	0.33	0.50	3.0
	Sandy Silt	182.7	178.1	8	0.35	0.52	2.9
	Silty Clay	178.1	166.4	7	0.38	0.56	2.6
East Abutment	Fill	184.5 *	182.7	20	0.33	0.50	3.0
	Sand and Silt	182.7	178.1	8	0.35	0.52	2.9
	Silty Clay	178.1	169.1	7	0.38	0.56	2.6

\* Top of sheet pile elevation varies.

The depth of sheet piles will also be governed by temporary construction conditions such as a heavy crane loading on the approach embankments during pile driving or girder lifting. A preliminary analysis of a typical crane loading indicates that the sheet piles should be driven to a minimum tip elevation of 177.0 (a depth of penetration of approximately 9.5 m) to maintain stability of the approaches under the temporary crane loading. It is suggested that this minimum tip elevation be adopted for design. Although, the depth of penetration may need to be greater to provide lateral stability.

In general, backfill to the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A or Granular B Type II or Granular B Type III material. All granular materials should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

Driving of the sheet piles through the existing approach fill may encounter cobbles. Removal of any such obstructions may be required to install the sheeting. Any visible obstructions such as boulders and rock protection along the sides of the embankment should be removed prior to driving the sheet

piles. Suggested wording for an NSSP in this regards is provided in Appendix E. Tip protection is recommended for the sheet piles.

In light of the sensitive sand and silt deposit and the underlying soft to firm clay below the water level, vibratory methods should not be used at this site to install sheet piles.

Design of the permanent sheet pile walls should consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure. The native soils in front of the sheet piles should be protected from creek erosion so that the sheet piles do not lose lateral support.

## **10 APPROACH EMBANKMENTS**

Based on the preliminary GA drawing and information from the designer, the existing road grade will be raised by 200 mm at the east abutment rising to 400 mm towards the east. The embankment will also be widened to accommodate the wider bridge with a sidewalk.

The foundation soils governing stability of the approach embankments consist of loose to compact sand and silt extending to 8.7 m depth underlain by soft to firm silty clay/clay extending to as much as 20.4 m depth.

No evidence of the embankment slope instability was observed during the field investigation. The existing embankment inclinations are relatively flat and covered by vegetation (grass and occasional shrubs). An increase of the grade by up to 400 mm is not expected to affect the embankment stability.

Placement of additional fill to widen the embankment is expected to result in consolidation settlement of the silty clay/clay deposits underlying the site. Based on computations using Terzaghi one-dimensional consolidation theory, the primary consolidation settlements are estimated to be in the order of 30 mm for a widening of 2 m and a grade raise of up to 400 mm. The noted settlement will largely occur as differential settlement between the existing embankment and the new widened section.

It is expected that some portion of this settlement will be induced immediately after the fill placement, however, long term deformation between the existing and the new part of the embankment should be anticipated and may require long-term maintenance after construction. To minimize differential settlements after construction, consideration could be given to scheduling widening at the beginning of the contract, nominal surcharging of the widened portion of the embankment, and delaying paving for as long as practical.

Construction of the embankment widening should be in accordance with OPSS.PROV 206. Any topsoil/organic deposit encountered within the footprint of the widening should be stripped from the plan limits of the proposed works and the subgrade soils should be proof-rolled. In order to minimize the differential settlement between the existing embankment slope and the newly placed embankment fill, the new fill should be keyed into the existing embankment side slope per the requirements of OPSD 208.010 (Benching the Earth Slopes). The slopes for the earth fill embankment should be no steeper than 2 horizontal to 1 vertical (2H:1V).

All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

## 11 SCOUR AND EROSION PROTECTION

Erosion protection, such as rock protection as per the requirements of OPSS 511, should be provided along any soil surfaces that may be in contact with the creek flow, to at least 0.5 m above the design high water level. In particular, erosion protection must be provided in front of the sheet pile walls to maintain lateral support of the walls.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

## 12 LATERAL EARTH PRESSURES

Earth pressures acting on the structure may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures 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 at depth  $h$  (kPa)

$K$  = coefficient of lateral earth pressure

$\gamma$  = unit weight of retained soil

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

$q$  = value of any surcharge (kPa).

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 12.1.

**Table 12.1 – Coefficients of Lateral Earth Pressure (K)**

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

\* For wing walls.

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

The factors in Table 12.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code (CHBDC).

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or III, or at a depth of 1.7 m for Granular A or Granular B Type II.

### 13 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.00
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.00
- Peak Ground Acceleration 0.011 g

The soil profile type at this site has been classified as Type III. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient *S* of 1.5 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects of earthquake loading. For the design of retaining walls under seismic loading, the coefficients of horizontal earth pressure in Table 13.1 may be used:

**Table 13.1 – Earth Pressure Coefficient for Earthquake Loading**

Earth Pressure Coefficient (K) for Earthquake Loading				
Loading Condition	Granular A or Granular B Type II $\phi = 35^\circ$ ; $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ$ ; $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active ( $K_{AE}$ )*	0.28	0.40	0.31	0.48
Passive ( $K_{PE}$ )	3.7	-	3.2	-
At Rest ( $K_{OE}$ )**	0.44	-	0.49	-

\* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

\*\* After Woods (1973).

Based on soil types and field test data, the upper and lower silts at the site are susceptible to liquefaction or cyclic mobility (CFEM, 4<sup>th</sup> Edition). However, considering the low seismic activity in the area (acceleration related seismic zone of zero), liquefaction of the foundation soils is not a concern.

#### **14 EXCAVATION AND DEWATERING**

The proposed abutments will be installed behind and in close proximity to the existing bridge abutments. Excavation for removal of the existing abutments and wing walls is expected to be carried out within the existing approach fills. Provided the work is not carried out during a period of unusually high water levels, the excavation is expected to remain above the creek water level.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill above the water table may be classified as Type 3 soil and as Type 4 soil below the water level. Flatter slopes may be required at locations where water seepage affects surficial stability.

It is recommended that removal of the existing abutment walls not extend below the creek water level to avoid the need for cofferdams and dewatering during excavation and the possible environmental impacts on the creek water quality.

The excavation and backfilling for foundations should be carried out in accordance with OPSS 902.

The selection of the method of excavation and equipment is the responsibility of the Contractor. Provision should be made for handling of pavement materials, potential obstructions in the fill, and cobbles/boulders. Special equipment may be required for removal of the existing abutments to the required levels.

Roadway protection will be required to facilitate staged construction at this site. The temporary excavation support system should be designed and constructed in accordance with OPSS 539. Sheet piles or soldier pile and lagging walls are considered appropriate for roadway protection. The Contractor should select the wall type and design taking into account the soil conditions encountered in the boreholes.

#### **15 CONSTRUCTION CONCERNS**

Potential construction concerns include, but are not limited to:

- Driving of H-piles and sheet piles for the replacement bridge may potentially cause settlement of the existing bridge during staged construction. It is recommended that settlement monitoring of the existing bridge be carried out for the duration of pile driving. The Contractor should be prepared with appropriate equipment on site to maintain the grade of the existing bridge within acceptable tolerance.
- Installation of the sheet piles may encounter resistance in the fill due to the presence of cobbles. The Contractor must allow for removal of any such obstructions. Vibratory methods must not be used to install sheet piles at this site.

- The existing piles and pile caps should be left in place to minimize disturbance to the loose to compact, cohesionless sand to sandy silt deposit and soft clay/silty clay underlying the site.
- The sequence of H-pile and sheet pile installation should be carefully considered to avoid pile alignment problems.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the subgrade soils to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support). Site conditions may limit the type of equipment suitable for use. The design and safety of any temporary works is the responsibility of the Contractor. Recommended wording for an NSSP addressing this issue is provided in Appendix E.

## 16 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Ms. Anna Piascik, P.Eng. The report was reviewed by Mr. Murray Anderson, P.Eng. and Dr. P. K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

### THURBER ENGINEERING LTD.

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## **Appendix A**

### **Record of Borehole Sheets**



## SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

### 1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

### 5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level

$C_{pen}$  Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

## EXPLANATION OF ROCK LOGGING TERMS


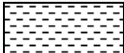



### ROCK WEATHERING CLASSIFICATION

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

### DISCONTINUITY SPACING

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

### SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

### STRENGTH CLASSIFICATION

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

### TERMS

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

# UNIFIED SOILS CLASSIFICATION

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

# RECORD OF BOREHOLE No PPC-01

1 OF 2

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 129.2 E 263 656.0 ORIGINATED BY GA  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN  
 DATUM Geodetic DATE 2013.06.23 - 2013.06.23 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  <b>γ</b>  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						
186.8	GROUND SURFACE													
0.0	ASPHALT:(100mm)													
0.1	SAND, trace silt, trace gravel Compact to Very Dense Brown Wet (FILL)  Occasional cobbles		1	SS	27		186							0 91 9 (SI+CL)
			2	SS	60									
			3	SS	17		185							
184.6														
2.2	SAND, some silt Compact Grey Wet		4	SS	14		184							
			5	SS	19									0 86 14 (SI+CL)
							183							
182.7														
4.1	Sandy SILT, some clay Loose Grey Wet		6	SS	4		182							
							181							
			7	SS	4									0 20 65 15
							180							
			8	SS	4		179							
178.1							178							
8.7	Silty CLAY, grading to clay, trace sand, varved Soft to Firm Grey Wet		9	SS	2									
							177							

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PPC-01

2 OF 2

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 129.2 E 263 656.0 ORIGINATED BY GA  
HWY 17 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN  
DATUM Geodetic DATE 2013.06.23 - 2013.06.23 CHECKED BY KS

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											
	Continued From Previous Page							20	40	60	80	100	20	40	60				
			10	SS	2		176							○					
								4.0											
			11	SS	2		175							○					
								3.0											
			12	SS	2		174												
							173												
							172												
			13	SS	4									○					
171.0							171												
15.8	END OF BOREHOLE AT 15.8m. BOREHOLE OPEN TO 15.8m AND WATER LEVEL AT 4.7m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, CONCRETE TO 0.1m THEN ASPHALT COLD PATCH TO SURFACE.																		

ONTMT4S 1197.GPJ 2015TEMPLATE(MTO).GDT 4/2/15

# RECORD OF BOREHOLE No PPC-02

1 OF 4

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 133.6 E 263 668.7 ORIGINATED BY GA  
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2013.06.26 - 2013.07.03 CHECKED BY KS

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						
186.8	GROUND SURFACE							20 40 60 80 100						
0.0	ASPHALT:(125mm)							20 40 60 80 100						
0.1	CONCRETE:(325mm)							20 40 60 80 100						
186.3								20 40 60 80 100						
0.5	SAND, occasional gravel Compact Brown Wet (FILL)		1	SS	24		186							
185.3														
1.5	SAND and GRAVEL, occasional cobbles Very Dense to Dense Brown Wet (FILL)		2	SS	86		185							
			3	SS	89		184							
			4	SS	28		183							59 39 2 (SI+CL)
182.7														
4.1	Sandy SILT, some clay, occasional roots and wood fibres Very Loose to Loose Grey Wet to Moist		5	SS	6		182							0 31 57 12
			6	SS	3		181							
							180							
			7	SS	4		179							
178.1														
8.7	Silty CLAY, grading to clay, trace sand, varved Soft to Firm Grey Wet		8	SS	5		178							0 6 77 17
							177							

Continued Next Page

+ 3, x 3 : Numbers refer to  
Sensitivity 20 15 10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No PPC-02

2 OF 4

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 133.6 E 263 668.7 ORIGINATED BY GA  
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2013.06.26 - 2013.07.03 CHECKED BY KS

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 ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE									WATER CONTENT (%)
	Continued From Previous Page							20	40	60	80	100					
	Silty <b>CLAY</b> , grading to clay, trace sand, varved Soft to Firm Grey Wet																
			9	SS	4			176									
								175									
			10	SS	2			174									
									3.0								
			1	TW				173									
									2.0								
			11	SS	2			171									
			12	SS	0			170									
								2	TW		169						
						168											
							3.0										
						167											

Continued Next Page

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

# RECORD OF BOREHOLE No PPC-02

3 OF 4

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 133.6 E 263 668.7 ORIGINATED BY GA  
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2013.06.26 - 2013.07.03 CHECKED BY KS

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							WATER CONTENT (%)			
Continued From Previous Page																		
166.4	Sandy <b>SILT</b> , trace clay Compact Grey Wet		13	SS	29		166											
20.4							165											
							164											
							163											
			14	SS	12		162							0 26 67 7				
160.0	<b>SAND</b> , some silt, trace gravel, occasional cobbles and boulders Very dense Brown Wet Spoon bouncing, cored through cobbles		15	SS	111/ 0.20		160											
26.8							159											
							158											
							157											
	Cored through cobbles		16	SS	100/ 0.125													

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No PPC-02

4 OF 4

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 133.6 E 263 668.7 ORIGINATED BY GA  
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2013.06.26 - 2013.07.03 CHECKED BY KS

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									
	Continued From Previous Page																
156.1	SAND, some silt, trace gravel, occasional cobbles and boulders Very dense Brown Wet		17	SS	122/												
30.7	END OF BOREHOLE AT 30.7 m. WATER LEVEL AT 2.9 mbgs IN OPEN BOREHOLE UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.  WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m)  May 02/14 3.9 182.9				0.250												

# RECORD OF BOREHOLE No PPC-03

1 OF 3

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 132.4 E 263 701.3 ORIGINATED BY ES  
HWY 17 BOREHOLE TYPE NW Casing COMPILED BY AN  
DATUM Geodetic DATE 2013.07.04 - 2013.07.04 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
186.8	GROUND SURFACE							20   40   60   80   100		w <sub>P</sub> w                      w <sub>L</sub>				
0.0	ASPHALT:(65mm)							○ UNCONFINED      + FIELD VANE						
0.1	CONCRETE WITH REBAR(430mm)							● QUICK TRIAXIAL      × LAB VANE						
186.3								20   40   60   80   100		20   40   60				
0.5	SAND, some gravel to gravelly, trace silt Loose to Dense Brown Moist (FILL)  Occasional cobbles		1	SS	9		186							
			2	SS	31		185							
			3	SS	20		184						28   70   2 (SI+CL)	
			4	SS	11		183							
182.7														
4.1	SAND and SILT, trace clay Loose Grey Wet to Moist          No recovery		5	SS	4		182						0   38   54   8	
			6	SS	9		181							
			7	SS	5		179							
178.1														
8.7	Silty CLAY, grading to clay, trace sand Soft to Firm Grey Wet		8	SS	1		178							
							177							

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity  
20  
15 10 5 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No PPC-03

2 OF 3

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 132.4 E 263 701.3 ORIGINATED BY ES  
HWY 17 BOREHOLE TYPE NW Casing COMPILED BY AN  
DATUM Geodetic DATE 2013.07.04 - 2013.07.04 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT		UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub> W      W <sub>L</sub>	WATER CONTENT (%)		
	Continued From Previous Page							20   40   60   80   100	○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE				
	Silty <b>CLAY</b> , grading to clay, trace sand Soft to Firm Grey Wet		1	TW			176	4.0					
							175	3.0					
			9	SS	0		174						
							173	4.0					
			10	SS	0		172	3.0					0   0   17   83
							171						
			11	SS	0		170	8.0					
			12	SS	0		169						
169.1							168						
17.7	<b>SILT</b> , trace clay, trace sand Compact Grey Wet		13	SS	18		167						0   1   90   9

Continued Next Page

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

## METRIC

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	ELEVATION SCALE
Continued From Previous Page					
162.4 24.4	<b>SILT</b> , trace clay, trace sand Compact Grey Wet  Occasional coarse sand seam				
			14	SS	19
	Occasional cobbles				
Gravelly <b>SAND</b> , trace silt, occasional cobbles Very dense Brown Wet	15	SS	100/ 0.05		
	16	SS	134/ 0.175		
159.2 27.6	END OF BOREHOLE AT 27.6 m. WATER LEVEL AT 0.68 mbgs IN OPEN BOREHOLE UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.		17	SS	100/ 0.125
WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m)  May 02/14      3.8      183.0					
DYNAMIC CONE PENETRATION RESISTANCE PLOT					
SHEAR STRENGTH kPa					
○ UNCONFINED + FIELD VANE					
● QUICK TRIAXIAL × LAB VANE					
WATER CONTENT (%)					
PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT					
w <sub>p</sub> w w <sub>L</sub>					
UNIT WEIGHT γ					
kN/m <sup>3</sup>					
REMARKS & GRAIN SIZE DISTRIBUTION (%)					
GR SA SI CL					
27 65 8 (SI+CL)					

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

# RECORD OF BOREHOLE No PPC-04

1 OF 2

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 137.0 E 263 716.0 ORIGINATED BY GA  
 HWY 17 BOREHOLE TYPE NW Casing COMPILED BY AN  
 DATUM Geodetic DATE 2013.06.24 - 2013.06.24 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE				WATER CONTENT (%) w <sub>P</sub> w      w <sub>L</sub>				GR	SA	SI	CL				
186.8	GROUND SURFACE						186									0   97   3 (SI+CL)							
0.0	ASPHALT:(100mm)																						
0.1	SAND, trace silt Compact Brown Wet (FILL) Cobble (100mm)		1	SS	22																		
			2	SS	17																		
			3	SS	12																		
184.6									185														
2.2	SAND, some silt, occasional wood fibres Loose to Compact Grey Wet		4	SS	6																		
			5	SS	8																		
									183														
			6	SS	12																		
							182																
							181																
							180																
			7	SS	11																		
							179																
			8	SS	14																		
							178																
							177																
178.1																							
8.7	Silty CLAY, grading to clay, trace sand, varved Soft to Firm Grey Moist		9	SS	2																		

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No PPC-04

2 OF 2

METRIC

WP# 6071-09-00 LOCATION Pays Plat Creek N 5 416 137.0 E 263 716.0 ORIGINATED BY GA  
 HWY 17 BOREHOLE TYPE NW Casing COMPILED BY AN  
 DATUM Geodetic DATE 2013.06.24 - 2013.06.24 CHECKED BY KS

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												
	Continued From Previous Page							20 40 60 80 100					20 40 60					
	Silty <b>CLAY</b> , grading to clay, trace sand, varved Soft to Firm Grey Moist							3.0 +										
			10	SS	2		176							○				
							175											
			11	SS	2		174						-----○-----			0 0 34 66		
								4.0 +										
			12	SS	2		173							○				
							172											
			13	SS	3								-----○-----			0 0 25 75		
171.0							171											
15.8	END OF BOREHOLE AT 15.8m. BOREHOLE OPEN TO 15.8m AND WATER LEVEL AT 4.2m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, CONCRETE TO 0.1m THEN ASPHALT COLD PATCH TO SURFACE.																	

ONTMT4S 1197.GPJ 2015TEMPLATE(MTO).GDT 4/2/15

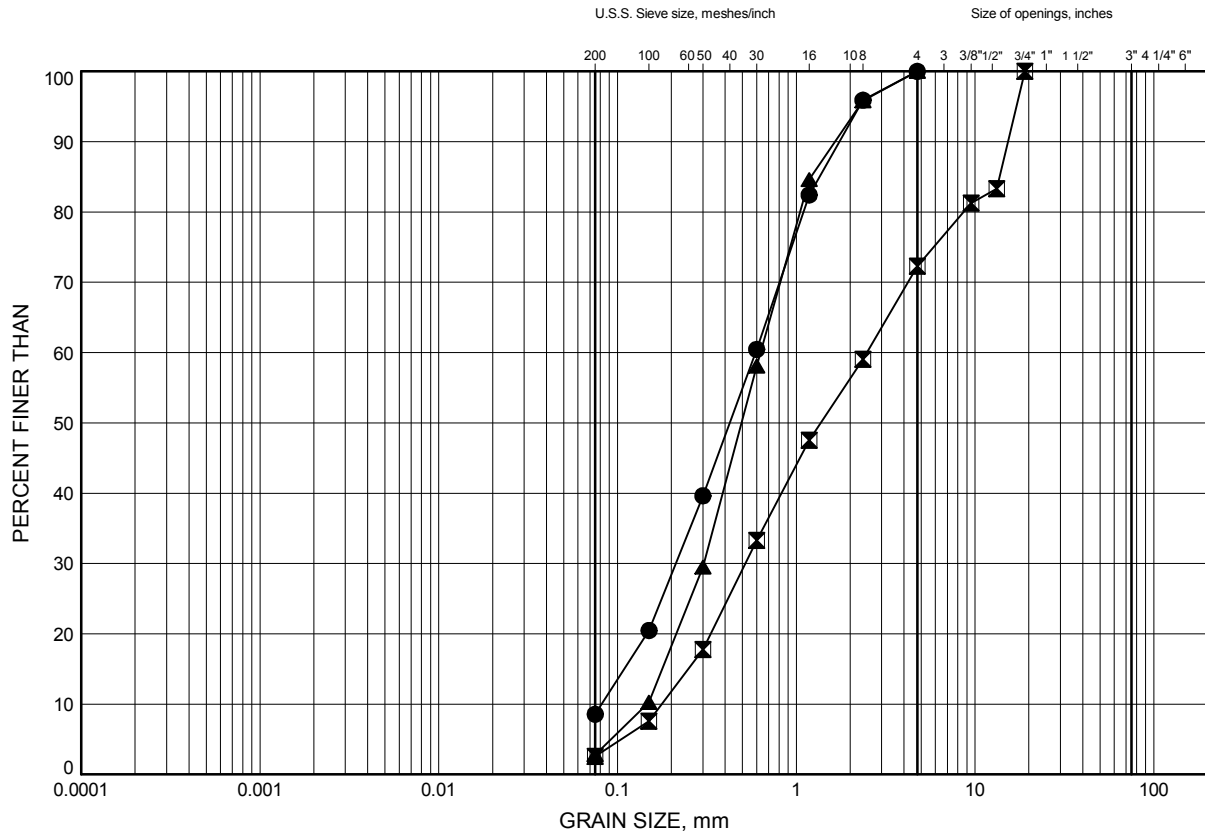
## **Appendix B**

### **Laboratory Test Results**

# Pays Plat Creek GRAIN SIZE DISTRIBUTION

FIGURE B1

## SAND/GRAVELLY SAND FILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-01	0.38	186.42
⊠	PPC-03	2.59	184.21
▲	PPC-04	1.07	185.73

Date January 2015  
WP# 6071-09-00



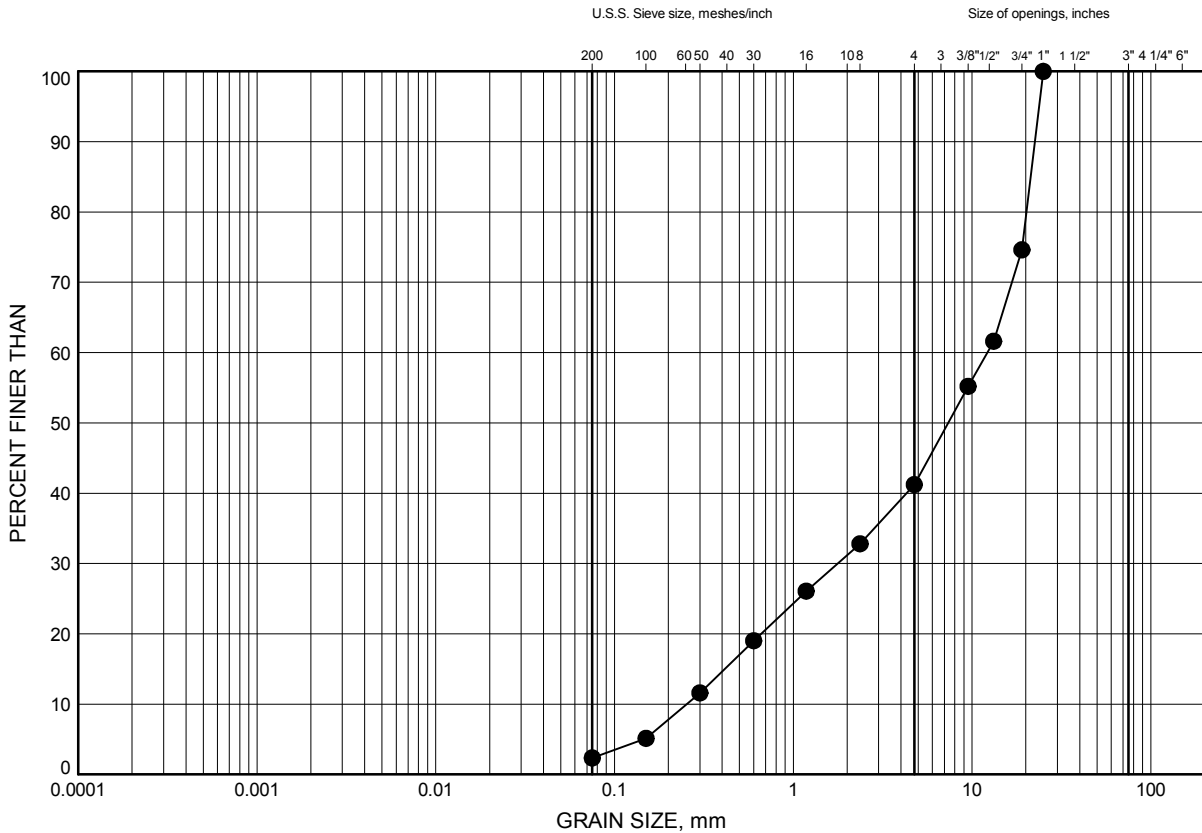
Prep'd AN  
Chkd. AP



# Pays Plat Creek GRAIN SIZE DISTRIBUTION

FIGURE B2

## SAND & GRAVEL FILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-02	3.35	183.45

Date January 2015  
WP# 6071-09-00

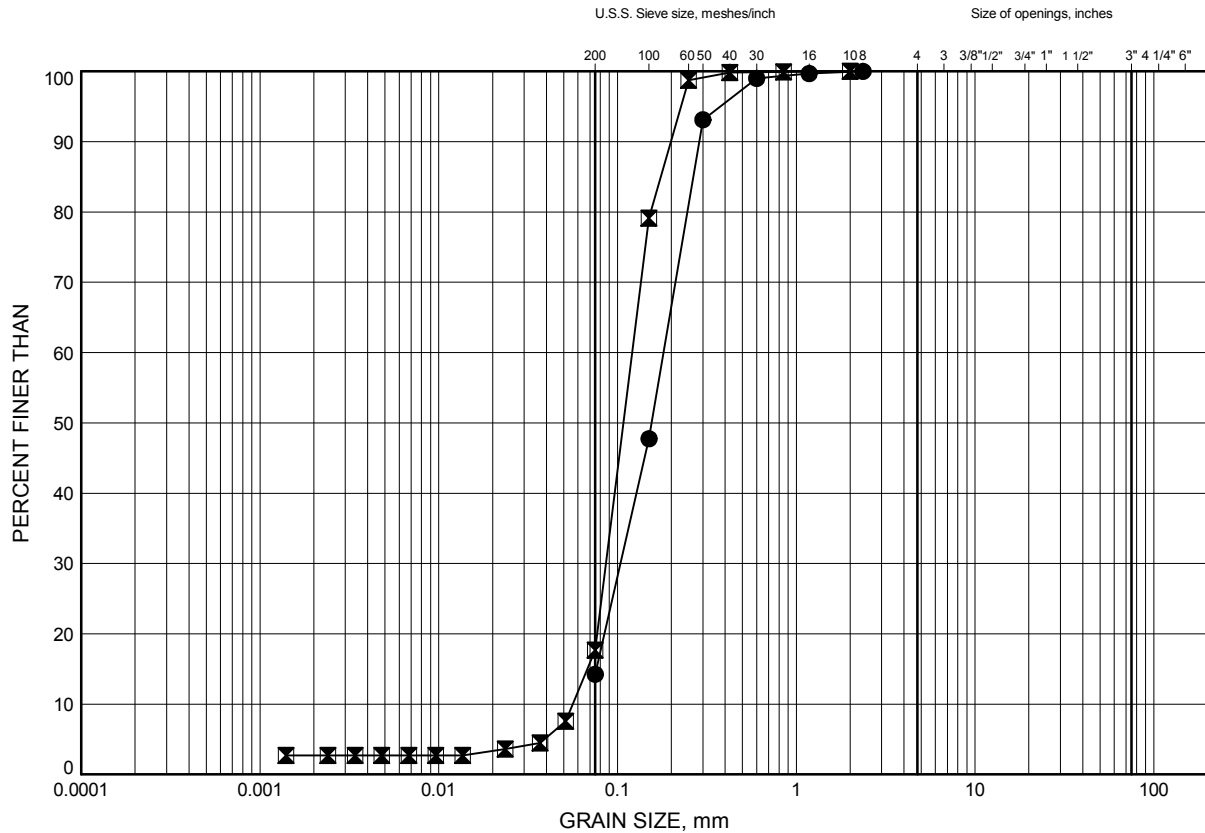


Prep'd AN  
Chkd. AP

# Pays Plat Creek GRAIN SIZE DISTRIBUTION

FIGURE B3

## Upper SAND



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-01	3.35	183.45
⊠	PPC-04	7.92	178.88

Date January 2015  
WP# 6071-09-00

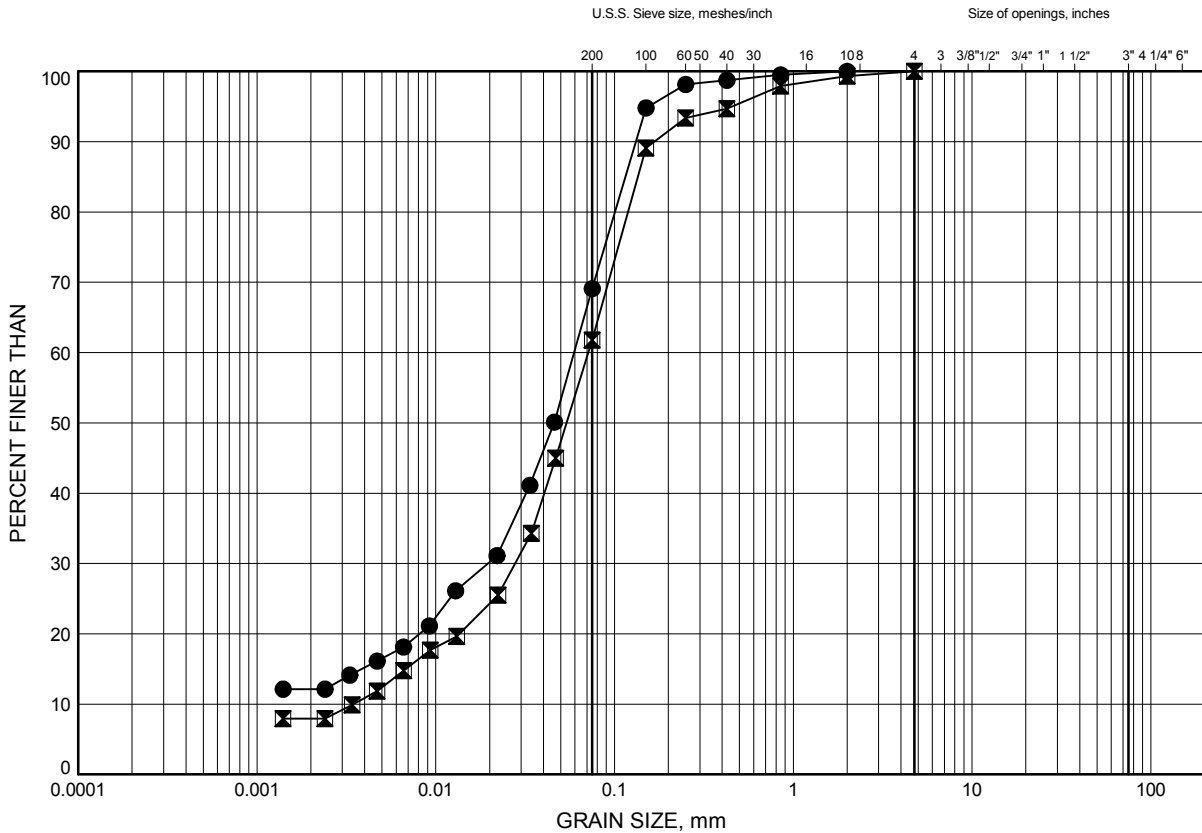


Prep'd AN  
Chkd. AP

# Pays Plat Creek GRAIN SIZE DISTRIBUTION

FIGURE B4

## Upper SAND & SILT/SANDY SILT



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-02	4.88	181.92
⊠	PPC-03	4.88	181.92

Date January 2015  
WP# 6071-09-00

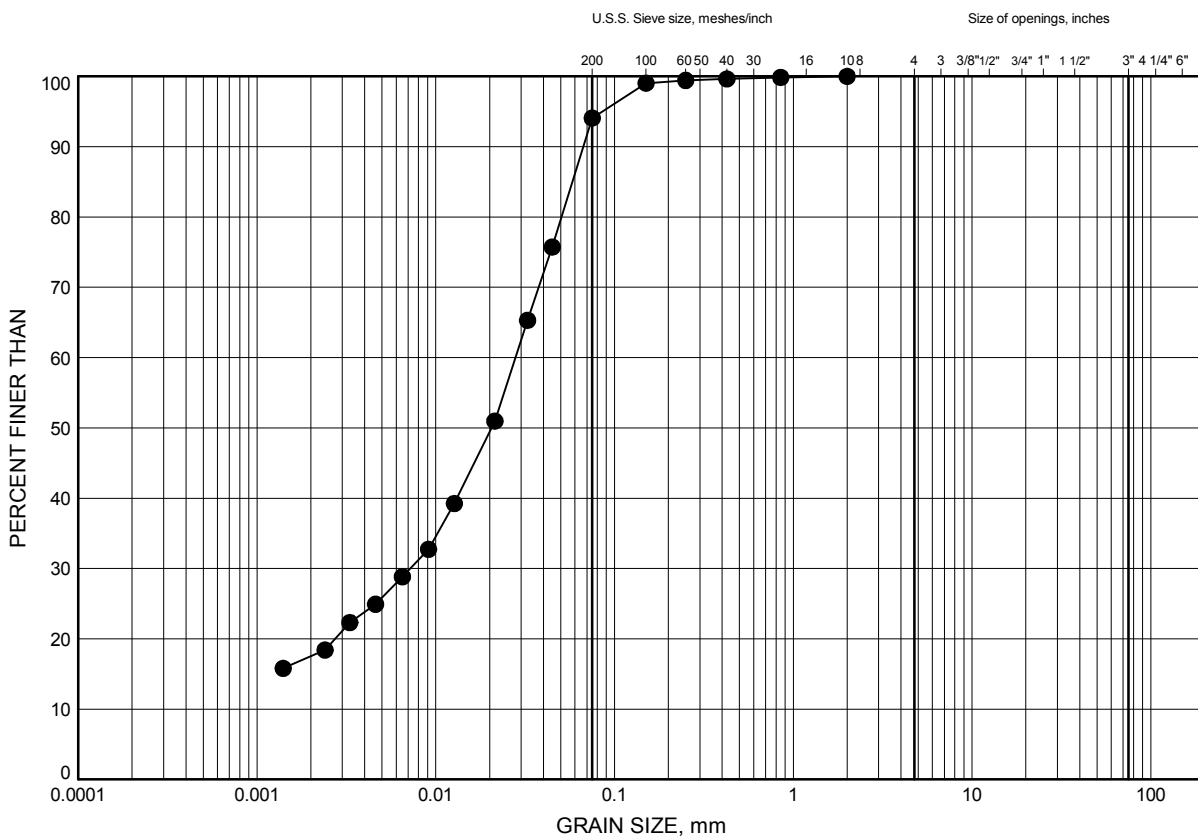


Prep'd AN  
Chkd. AP

# Pays Plat Creek GRAIN SIZE DISTRIBUTION

FIGURE B5

## SILTY CLAY



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-02	9.45	177.35

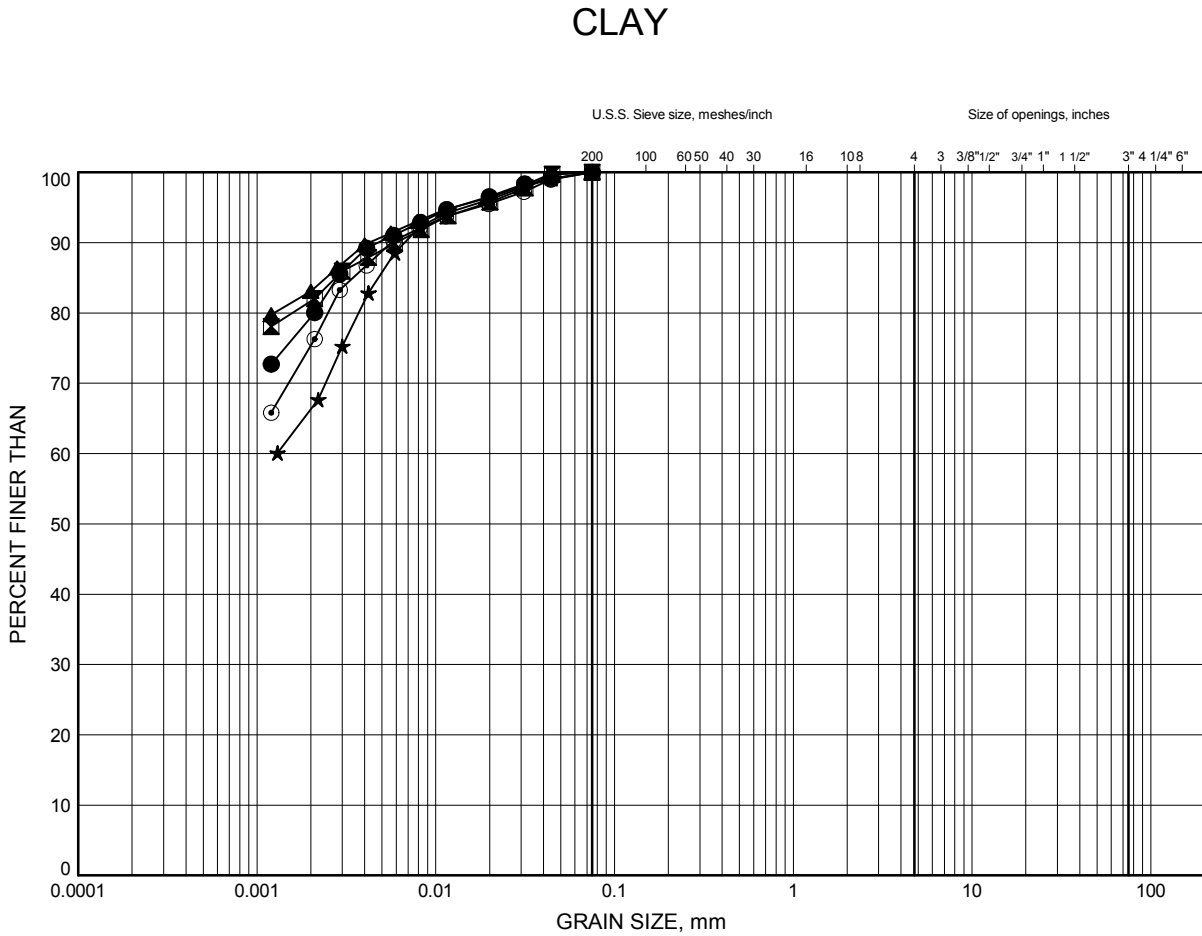
Date January 2015  
WP# 6071-09-00



Prep'd AN  
Chkd. AP

# Pays Plat Creek GRAIN SIZE DISTRIBUTION

FIGURE B6



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-01	14.02	172.78
⊠	PPC-02	15.54	171.26
▲	PPC-03	14.02	172.78
★	PPC-04	12.50	174.30
⊙	PPC-04	15.54	171.26

Date January 2015  
WP# 6071-09-00

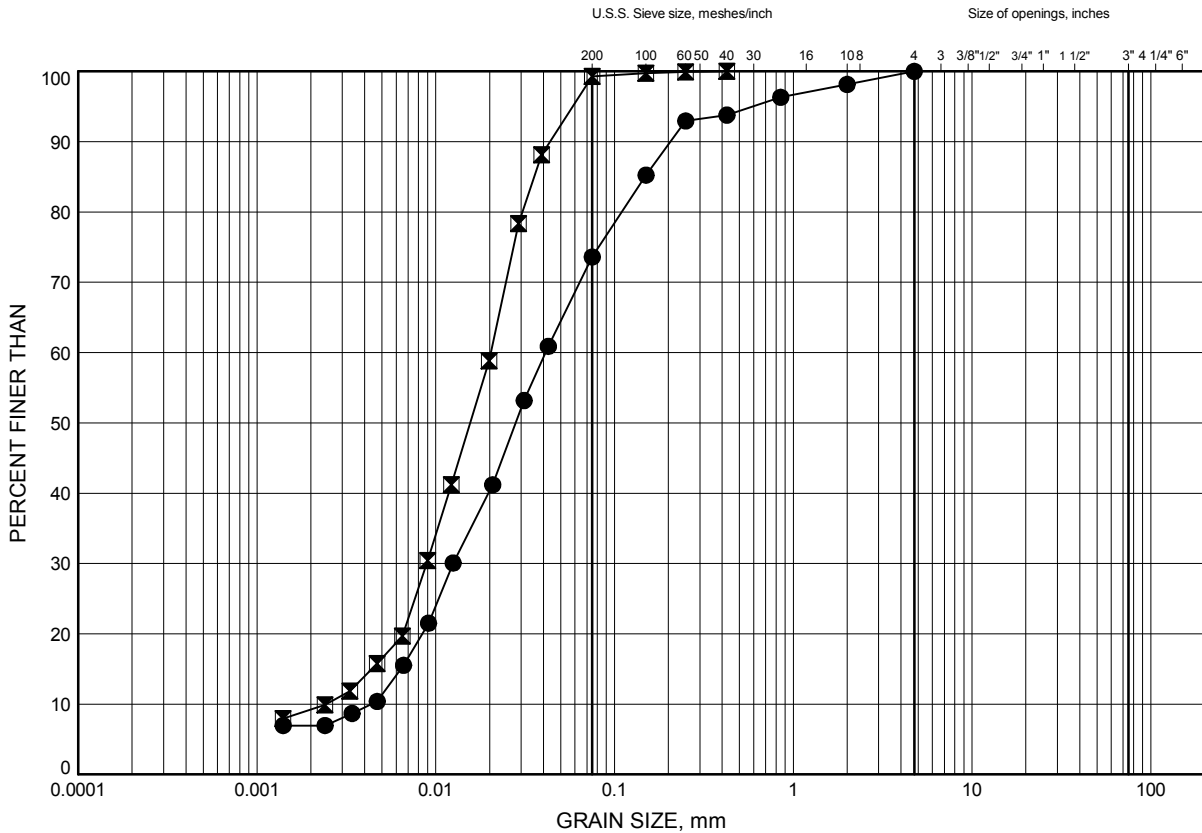


Prep'd AN  
Chkd. AP

# Pays Plat Creek GRAIN SIZE DISTRIBUTION

FIGURE B7

## Lower SANDY SILT to SILT



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-02	24.69	162.11
⊠	PPC-03	18.59	168.21

Date January 2015  
WP# 6071-09-00

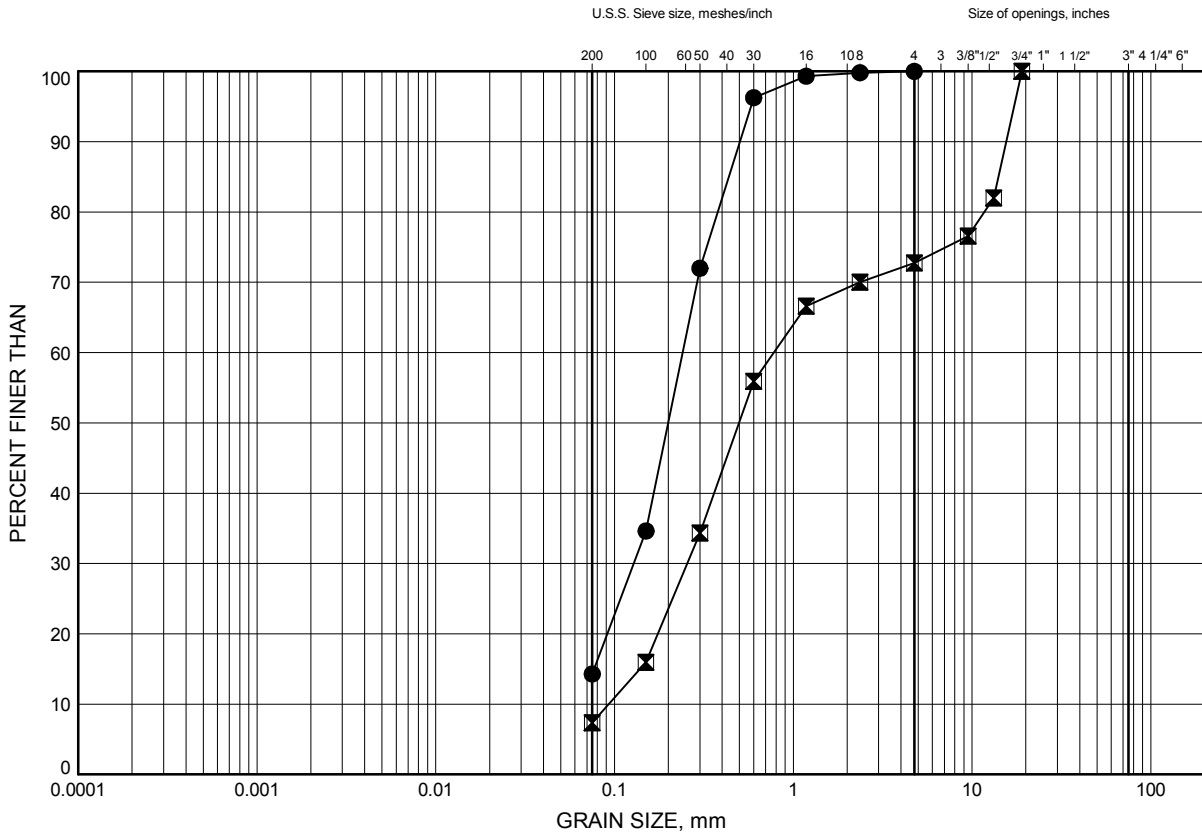


Prep'd AN  
Chkd. AP

Pays Plat Creek  
GRAIN SIZE DISTRIBUTION

FIGURE B8

Lower SAND to GRAVELLY SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-02	27.61	159.19
⊠	PPC-03	26.14	160.66

Date January 2015  
WP# 6071-09-00

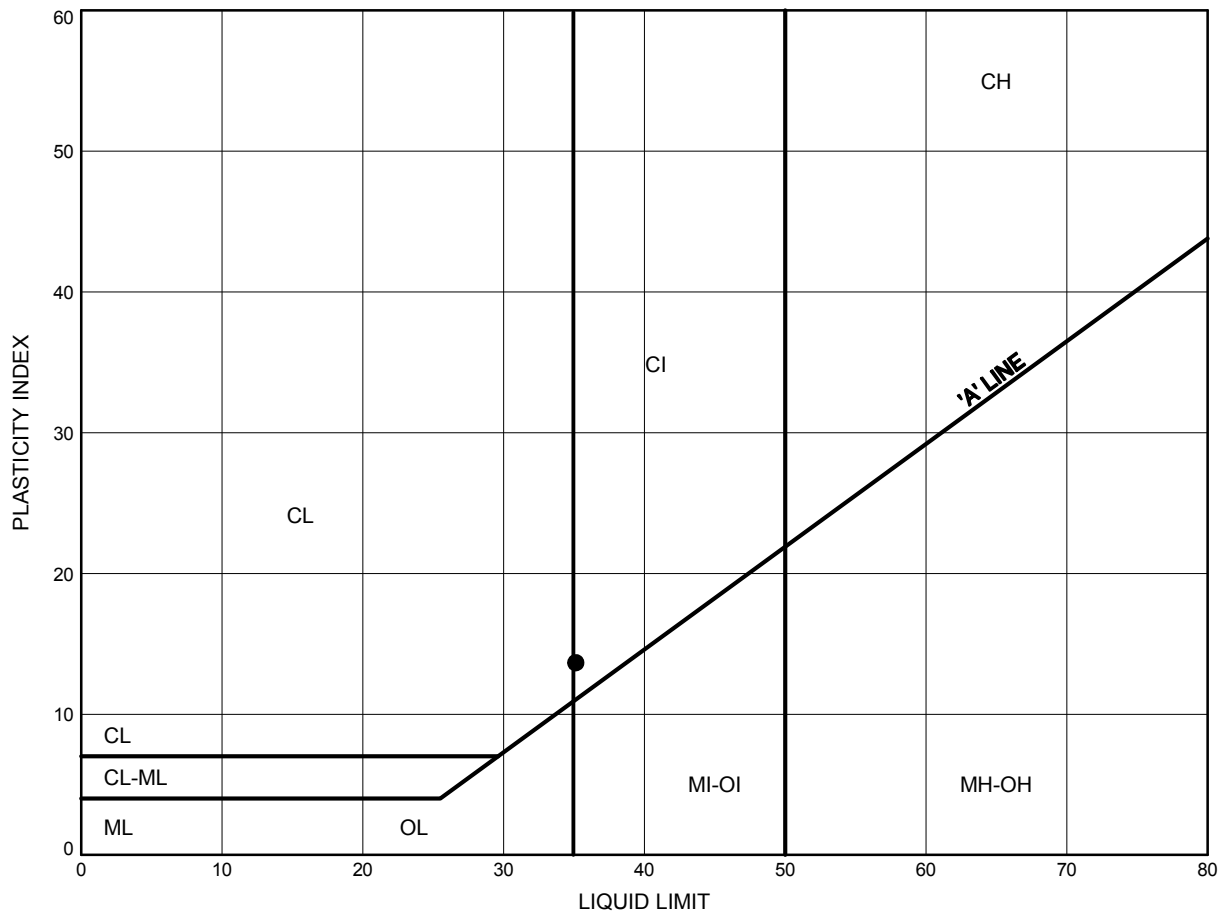


Prep'd AN  
Chkd. AP

Pays Plat Creek  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B9

**SILTY CLAY**



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-02	9.45	177.35

Date January 2015  
 WP# 6071-09-00



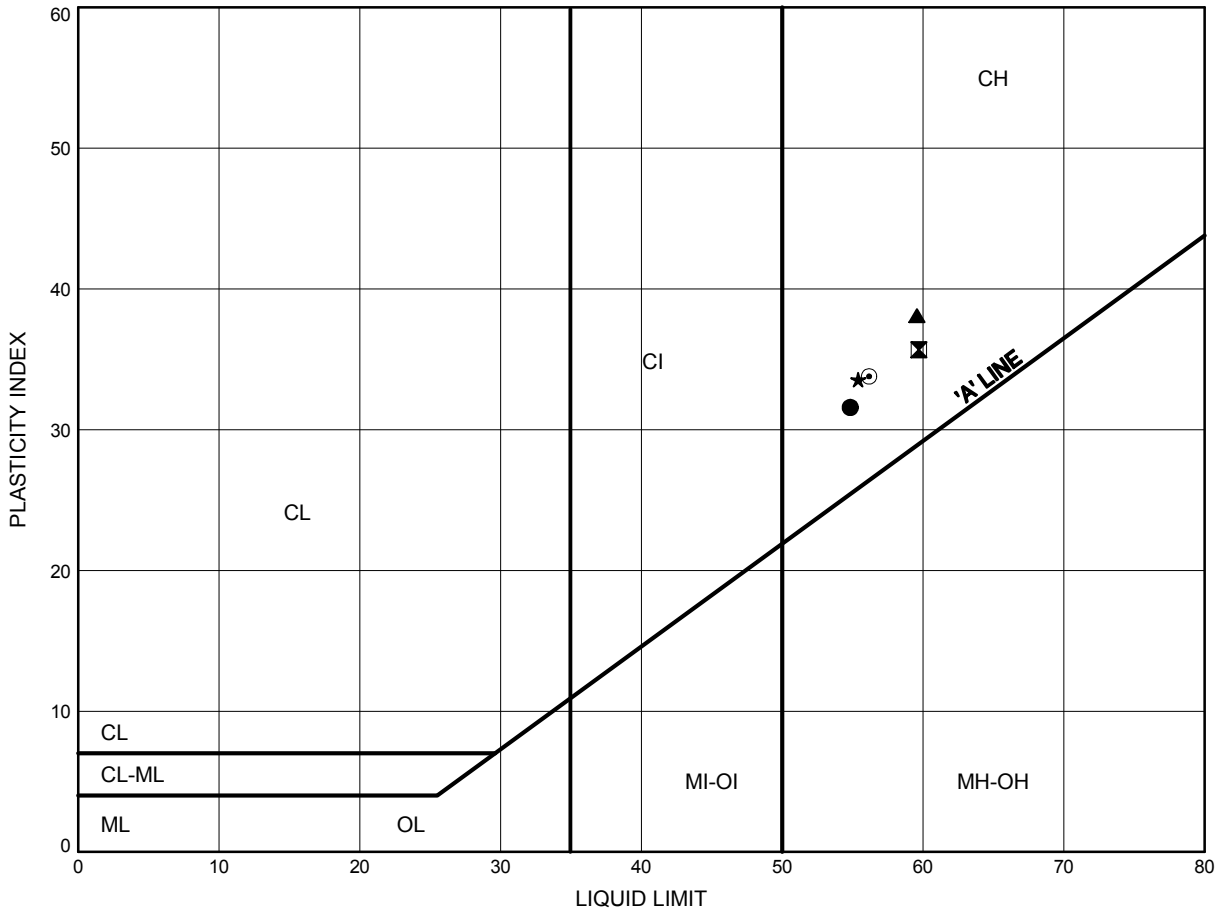
Prep'd AN  
 Chkd. AP



Pays Plat Creek  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B10

CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PPC-01	14.02	172.78
⊠	PPC-02	15.54	171.26
▲	PPC-03	14.02	172.78
★	PPC-04	12.50	174.30
⊙	PPC-04	15.54	171.26

Date January 2015  
 WP# 6071-09-00



Prep'd AN  
 Chkd. AP

## **Appendix C**

### **Site Photographs**



**Photograph 1 – Pays Plat Creek Bridge Looking East**



**Photograph 2 – Pays Plat Creek Bridge Looking West**





**Photograph 3 - North Bridge Elevation - Looking West**



**Photograph 4 - North Bridge Elevation - Looking East**

## **Appendix D**

### **Comparison of Foundation Alternatives**

### COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Engineered Fill	Driven H-Piles	Caissons
<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• Ease of construction.</li> <li>• Lower cost than deep foundations.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• Generally less costly construction than deep foundation elements.</li> <li>• Allows use of perched abutments.</li> <li>• Higher geotechnical resistance than for spread footings placed directly on native soil.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• Piles will develop high geotechnical resistance on very dense soils.</li> <li>• Installation of piles could continue in freezing weather.</li> <li>• Allows integral abutment design.</li> <li>• Requires less excavation than spread footings.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>• Higher resistances may be available for caissons than for spread footings founded in native soils.</li> <li>• Construction of caissons could continue in freezing weather.</li> </ul>
<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Low geotechnical resistance available in native soils</li> <li>• Potential for significant consolidation settlement in silty clay/clay not acceptable for the structure.</li> <li>• Dewatering may be required, depending on depth of excavation.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Cost of engineered fill placement.</li> <li>• Potential for significant consolidation settlement in silty clay/clay not acceptable for the structure.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Higher unit costs than for spread footings.</li> <li>• Possibility that cobbles and/or boulders may be encountered in the fill and native deposits, obstructing pile installation.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>• Relatively low capacities in native soils.</li> <li>• High cost of construction, as caissons would need to be relatively deep.</li> <li>• Possibility of encountering cobbles and boulders during installation.</li> <li>• Liners and/or drilling mud required to support caisson sidewalls and prevent base heave.</li> <li>• Difficulty in cleaning and inspecting bases.</li> </ul>
<b>NOT RECOMMENDED</b>	<b>NOT RECOMMENDED</b>	<b>RECOMMENDED</b>	<b>NOT RECOMMENDED</b>

## **Appendix E**

### **List of Standard Specifications and Special Provisions**

- 1) The following Standard Specifications and Special Provisions are referenced in this report:

OPSS 501

OPSS 511

OPSS 539

OPSS 804

OPSS 902

OPSS 903

OPSS.PROV 1010

OPSS.PROV 206

OPSD 208.010

SS103-11 (Hiley Formula)

- 2) Suggested wording for “NSSP – Use of Heavy Construction Equipment”

The use of heavy construction equipment and in particular heavy lift cranes may be required during removal of the existing bridge and erection of the new bridge. The impact of the heavy equipment loads on the underlying sensitive soils, creek banks and existing bridge foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO’s RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – High Complexity.

The assessment shall include, but not be limited to, the following:

- Determining appropriate setbacks for heavy equipment from the creek banks and existing foundations;
- Evaluating the need for preventing heavy equipment from travelling or operating on the areas adjacent to the creek, possibly requiring restriction of heavy loads to the existing highway embankment platform;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation failure.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.



3) Suggested wording for “NSSP – Monitoring of Existing Structure”

The Contractor shall ensure that the existing structure remains stable while in use during construction.

A monitoring program is required to confirm that any movements of the existing structure remain within tolerable levels. As a minimum, the monitoring program should require the Contractor to establish reference points over each abutment of the existing structure and to monitor movement of these points relative to known, fixed reference points on a regular basis. The suggested frequency is:

- Three readings on separate days prior to construction to establish a baseline;
- Twice daily while any foundation construction or other subsurface construction is in progress;
- Daily for one week after completion of foundation construction.

The vertical and horizontal accuracy of readings should be  $\pm 2$  mm. All readings must be reported to the Contract Administrator within 24 hours and immediately if any movement exceeds limits set by the structural designer.

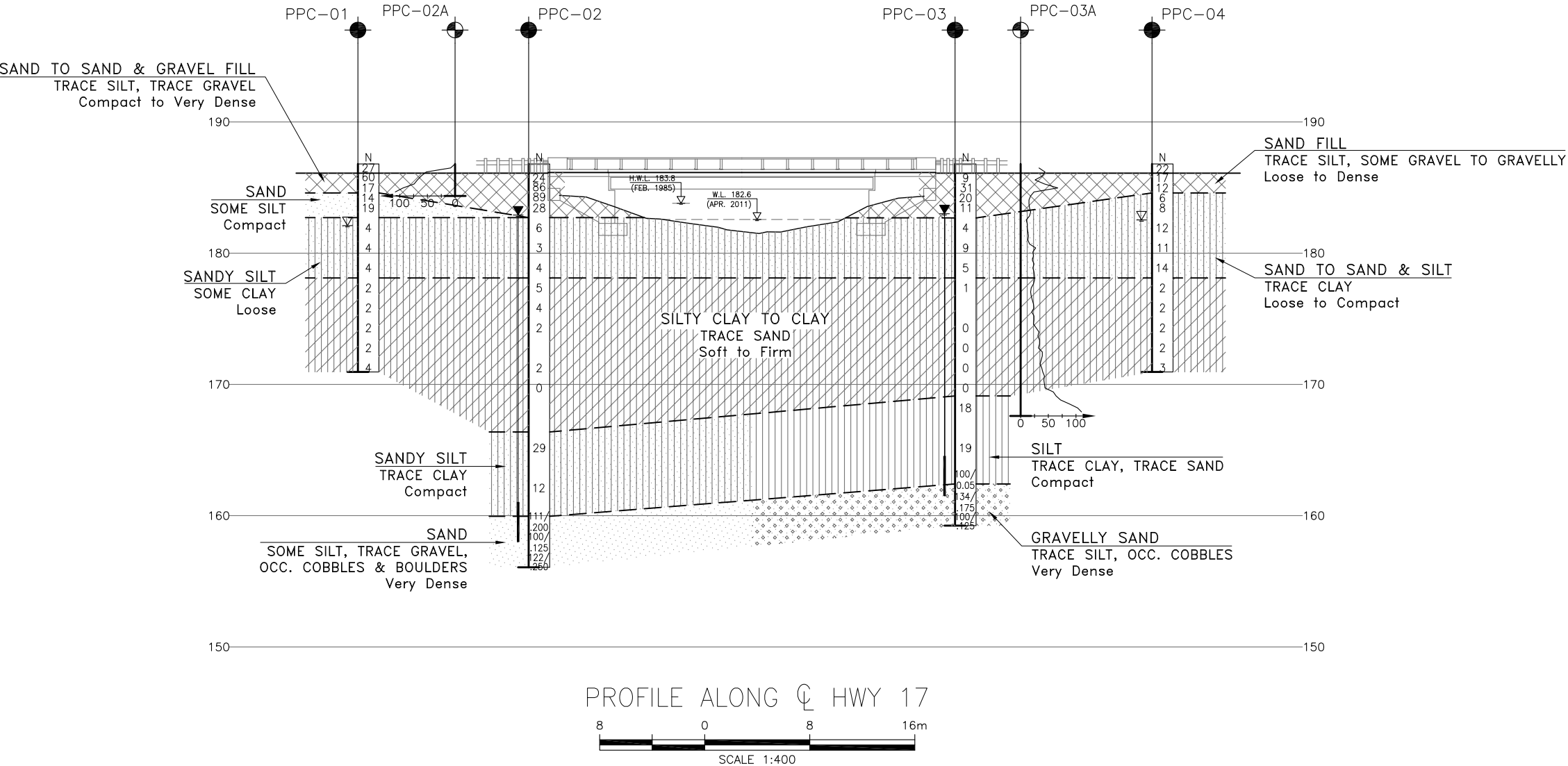
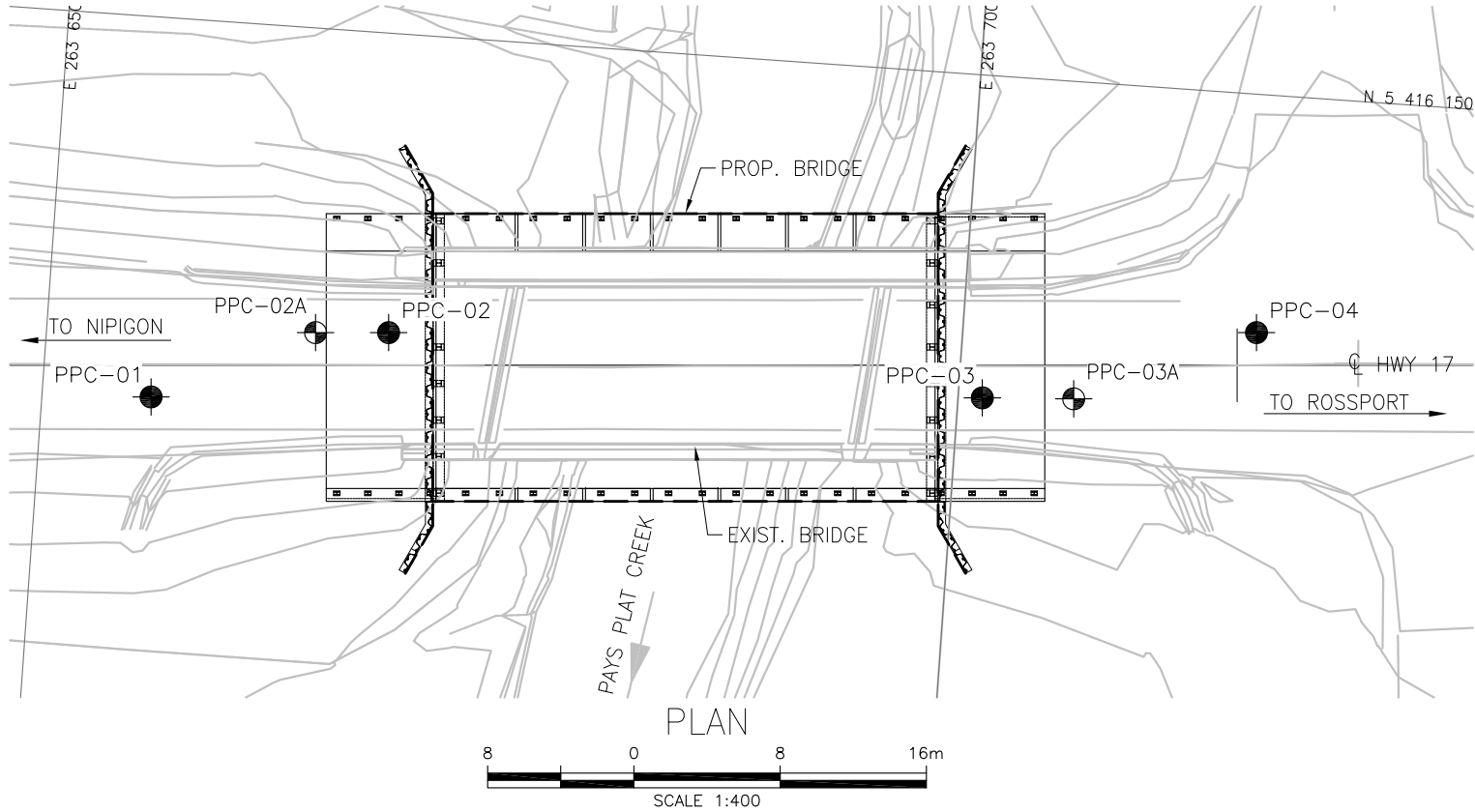
The Contract Administrator must be advised of the importance of monitoring and be required to advise the Ministry immediately if the vertical and horizontal movements exceed the specified limits.

4) Suggested wording for “NSSP – Presence of Cobbles and Boulders”

Cobbles and boulders are present within the existing embankment fill and in the sand deposit underlying the silts and clays on site. The cobbles and boulders may interfere with H-pile and sheet pile installation. The Contractor must be prepared to remove, dislodge or otherwise penetrate these obstructions to advance the piles to the specified tip elevation/resistance while meeting the specified deflection tolerances.

## **Appendix F**

### **Borehole Locations and Soil Strata Drawing**



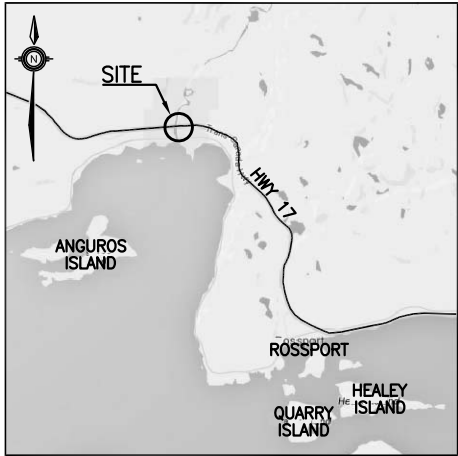
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN



CONT No WP No 6072-09-01	
HIGHWAY 17 PAYS PLAT CREEK BRIDGE BOREHOLE LOCATIONS AND SOIL STRATA	
	SHEET



THURBER ENGINEERING LTD.



### KEYPLAN

### LEGEND

	Borehole
	DCPT (Dynamic Cone Penetration Test)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level In Open Borehole
	Water Level In Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
PPC-01	186.8	5 416 129.2	263 656.0
PPC-02	186.8	5 416 133.6	263 668.7
PPC-02A	186.8	5 416 133.4	263 664.7
PPC-03	186.8	5 416 132.4	263 701.3
PPC-03A	186.8	5 416 132.7	263 706.3
PPC-04	186.8	5 416 137.0	263 716.0

### -NOTES-

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

GEOCRES No. 42D-37

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MRA	CHK MRA	CODE
DRAWN	AN	CHK SKP	SITE 48C-19 STRUCT DWG 2