



**THURBER** ENGINEERING LTD.



**FOUNDATION INVESTIGATION AND DESIGN REPORT  
DUCHESNAY CREEK BRIDGE REPLACEMENT  
HIGHWAY 17B, NORTH BAY, ONTARIO  
G.W.P. 5120-07-00, SITE #43-067**

**GEOCRES No. 31L-197**

**Report**

to

**LEA Consulting Ltd.**

Date: November 27, 2017  
File: 19-3948-5

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**GEOGRES No. 41H-166**

**PART 1: FACTUAL INFORMATION**

**1. INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation conducted for the proposed replacement of the Duchesnay Creek Bridge on Highway 17B, located in the City of North Bay, 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 and cross-section, 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 LEA Consulting Limited, under the Ministry of Transportation Ontario (MTO) Agreement Number 5014-E-0017.

A preliminary foundation investigation was carried out at this site for the replacement bridge (Preliminary Foundation Investigation and Design Report, Duchesnay Creek Bridge Replacement, Highway 17B, Ontario; prepared by Stantec Consulting Ltd.; dated June 2014; Geocres No. 31L-179). The information presented in the above report was reviewed and incorporated in the current investigation, and the borehole logs and location plan are provided in Appendix C for information purposes.

**2. SITE DESCRIPTION**

The bridge site is located on Highway 17B approximately 1.1 km south east of the Highway 17/17B intersection in the City of North Bay, Ontario. The Duchesnay Creek flows from north to south at the bridge location into Lake Nipissing. The bridge site is located approximately 70 m north of the creek mouth. A CPR Rail line is located about 30 m south of the bridge location.

Duchesnay Creek flows from north to south at the bridge location. The creek banks are well vegetated with tall grass and shrubs and frequent trees. The local topography is of low relief with no visible bedrock outcrops. Local slope instability has been observed on the northeast bank of the river. Photographs in Appendix D show the general nature of the site and the existing bridge.



The site lies within the physiographical area of Canadian Shield, which is characterized by Pre-Cambrian igneous and metamorphic bedrock typically occurring as rounded knobs and ridges where exposed. According to Ontario Geological Survey (OGS) data, the bedrock at this site generally consists of Mesoproterozoic magmatic rock and gneisses of the Grenville Province's Central Gneiss Belt. The bedrock is overlain by a discontinuous cover of Pleistocene sands and gravels (glaciofluvial outwash) and silts and sands (glaciofluvial outwash) overlain by silt and clay (glaciolacustrine deposit).

### 3. SITE INVESTIGATION AND FIELD TESTING

The preliminary foundation investigation for the replacement bridge consisted of advancing two sampled boreholes (Boreholes 13-3 and 13-4) in 2013 through the approach embankment behind the west and east bridge abutments, respectively. The borehole depths ranged from 22.8 to 29.7 m. Bedrock was encountered in both boreholes and proved by coring 3 m.

The current site investigation and field testing program for this project was carried out in two segments. The first between December 14 and December 21, 2015 and the second between January 20 and February 2, 2016. A total of eight boreholes, identified as DCB-01 to DCB-08, were advanced to depths ranging from 7.3 to 26.2 m below the existing ground surface. Details of the borehole locations, drilling depths and completion details are summarized in Table 3.1 below.

**Table 3.1 – Borehole Summary**

Location	Boreholes	Drilling & Coring Depth/Elev. (m)	Completion Details
West Approach	DCB-01	9.8/195.0	Borehole backfilled with bentonite holeplug and cuttings to 0.1 m, and asphalt coldpatch to ground surface.
West Abutment	DCB-02	20.3/184.1	Standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen installed. Bottom of screen located at 20.1 m, sand filter to 16.6 m and bentonite holeplug to 3.7 m, bentonite holeplug mixed with cuttings to 0.6 m, sand to 0.3 m and then concrete to ground surface.
West Pier	DCB-05	7.3/188.9	Standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen installed. Bottom of screen located at 7.2 m, sand filter to 3.8 m and bentonite holeplug mixed with cuttings to the ground surface.
West Pier	DCB-06	11.7/186.1	Borehole backfilled with bentonite holeplug and cuttings to ground surface.

Location	Boreholes	Drilling & Coring Depth/Elev. (m)	Completion Details
East Pier	DCB-07	18.7/182.0	Standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen installed. Bottom of screen located at 15.2 m, sand filter to 11.9 m and bentonite holeplug mixed with cuttings to 0.3 m, and cuttings to the ground surface.
East Pier	DCB-08	11.9/185.8	Borehole backfilled with bentonite holeplug and cuttings to ground surface.
East Abutment	DCB-03	26.2/180.1	Standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen installed. Bottom of screen located at 26.1 m, sand filter to 23.2 m and bentonite holeplug to 1.5 m, bentonite holeplug mixed with cuttings to 0.6 m, sand to 0.3 m and then concrete to ground surface.
East Approach	DCB-04	9.8/196.8	Borehole backfilled with bentonite holeplug and cuttings to 0.2 m and asphalt coldpatch to ground surface.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawings included in Appendix H.

Borehole DCB-04 was advanced using a track mounted Diedrich D-50 drill rig in combination with hollow stem augers to advance the borehole in the overburden. Borehole DCB-08 was advanced using a portable tripod drilling rig in combination with NW wash boring methods to advance the borehole in the overburden. The remaining boreholes were advanced using a Diedrich D-25 drill rig in combination with hollow stem augers and NW casing methods to advance the boreholes in the overburden. Samples of the overburden 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 using an MTO "N" size vane were carried out in very soft to soft cohesive soils.

Core samples of the underlying bedrock were recovered from Boreholes DCB-06 and DCB-07 using NQ rock coring equipment. All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

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 DCB-02, DCB-03, DCB-05 and DCB-07. Following the final water level reading, the piezometers were decommissioned in general accordance with MOE Regulation 903.

#### **4. LABORATORY TESTING**

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

Point load tests (PLT) were performed on selected intact rock core samples. Unconfined compressive strengths (UCS) of the rock cores correlated from the PLT results are shown on PLT sheets included in Appendix B.

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

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered stratigraphy are presented in this appendix and on the “Borehole Locations and Soil Strata” drawings in Appendix H. Boreholes 13-3 to 13-4 from the preliminary investigation (referenced hereafter as previous boreholes) were considered in the current investigation. The borehole logs from the previous investigation are presented in Appendix C. 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 a deposit of sandy silt to silty sand underlain by a firm to stiff silty clay that overlies a discontinuous layer of clayey silt till or silt over granite bedrock. A sand deposit was found underlying the embankment fill in boreholes on the east approach. More detailed descriptions of the individual strata are presented below.

##### **5.1 Asphalt**

Asphalt pavement was encountered in Boreholes DCB-01 to DCB-04. The thickness of the asphalt ranged from 100 to 175 mm in the boreholes.

##### **5.2 Topsoil**

Topsoil was encountered in Borehole DCB-07. The thickness of the topsoil was 50 mm at the borehole location. Topsoil thickness may vary in other areas of the site.

##### **5.3 Sand and Gravel Fill and Rockfill**

Existing embankment fill was encountered below the asphalt pavement in Boreholes DCB-01 to DCB-04 and immediately at the ground surface in Boreholes DCB-05 to DCB-08. The brown fill

is heterogeneously composed of gravelly sand and sand, and contained trace to some silt and organics. Occasional cobbles were present in the fill deposit. The thickness of the fill ranged from 0.9 to 2.1 m, with the base of the fill at Elev. 205.2 to 194.7.

Rockfill was encountered in Borehole DCB-03 underlying the sand fill. The thickness of the rockfill was 3.9 m with the base of the layer at Elev. 201.3.

A layer of sand and gravel fill was inferred below the rockfill in Borehole DCB-03. The thickness of the inferred sand and gravel fill was 1.1 m with the base at Elev. 200.2.

SPT 'N' values recorded in the fill ranged from 3 to 46 blows per 0.3 m penetration, indicating a loose to very dense relative density. One SPT 'N' value of 77 blows per 0.3 m penetration was recorded at the surface in Borehole DCB-04. Moisture contents of the fill ranged from 2 to 28%.

Fill thickness in previous Boreholes 13-3 and 13-4 ranged from 3.6 to 4.3 m. Rockfill was noted from 1.4 to 4.6 m in Borehole 13-4. Coring was required to get through the rockfill in Borehole DCB-03 and 13-4.

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

Soil Particle	Percentage (%)	
	Gravelly Sand	Sand
Gravel	30-34	1-17
Sand	58-63	72-95
Silt & Clay	8-7	4-11

## 5.4 Sand

A layer of sand was encountered below the fill in Boreholes DCB-03 and DCB-04. The sand contains trace to some gravel and silt. Occasional cobbles are present in the deposit. The thickness of the sand layer ranged from 1.1 to 5.6 m, and the lower boundary was encountered at Elev. 199.4 to 199.1.

SPT 'N' values recorded in the sand layer ranged from 15 to 23 blows per 0.3 m penetration, indicating a compact relative density. Natural moisture contents of the sand ranged from 13 to 22%.

The results of grain size analyses conducted on a sample of the sand are provided on the Record of Borehole sheets in Appendix A, and plotted in Figure B2 of Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	0
Sand	80
Silt	17
Clay	3

### 5.5 Sandy Silt to Silty Sand

A deposit of sandy silt to silty sand was encountered below the fill in Boreholes DBC-01, DCB-02, DCB-05, DCB-06 and DCB-07, and below the sand deposit in Boreholes DCB-03 and DCB-04. This deposit is brown to grey in colour and contains trace to some clay and trace organic material. In Borehole DCB-05, this layer contained wood pieces and organics between 2.3 and 3.5 m depth. Where fully penetrated, the thickness of the layer ranged from 0.8 to 5.9 m with the lower boundary encountered at Elev. 200.7 to 188.9. Borehole DBC-04 was terminated in the deposit at a depth of 9.8 m or Elev. 196.8.

SPT 'N' values recorded in the sandy silt to silty and layer ranged from 3 to 20 blows per 0.3 m penetration, indicating a very loose to compact relative density. Natural moisture contents of the deposit ranged from 16 to 30%. Two moisture contents, 61 and 71%, were recorded in Borehole DCB-05 at depths of 2.3 and 3.0 m respectively. The high moisture contents are representative of organic material found in the samples.

The results of grain size analyses conducted on samples of the sandy silt to silty sand are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figures B3 and B4 of Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)		
	Sandy Silt	Silt & Sand	Silty Sand
Gravel	0	0	0
Sand	20-34	35-47	72
Silt	57-66	46-53	24
Clay	9-22	7-12	4

### 5.6 Silty Clay

A layer of grey silty clay with trace sand was encountered below the sandy silt to silty sand in Boreholes DCB-01, DCB-02, DCB-03, DCB-06 and DCB-07, and below the fill in Borehole DCB-08.

Where fully penetrated, the thickness of the layer ranged from 5.6 to 15.3 m with the lower boundary encountered at Elev. 189.9 to 180.8. Borehole DCB-01 was terminated in the silty clay deposit at a depth of 9.8 m or Elev. 195.0.

The silty clay transitioned to reddish brown with silty sand interbeds in Boreholes DCB-02, DCB-03, DCB-06 and DCB-07 at depths ranging from 6.7 to 20.1 m or Elev. 194.2 to 186.2.

SPT 'N' values recorded in the silty clay typically ranged from 1 to 27 blows per 0.3 m penetration with typical values between 4 and 14 blows per 0.3 m penetration. Field vane shear tests (VST) measured undrained shear strengths ranging from 32 to 118 kPa with typical values between 34 and 85 kPa. Based on the SPT and VST data, the consistency of the silty clay varies from firm to stiff. Very stiff zones were identified near the top of the silty clay layer in Boreholes DCB-01, DCB-02 and DCB-03. Sensitivity of the silty clay, calculated as a ratio of undisturbed strength to remoulded strength, ranged from 3 to 7, suggesting that the silty clay is low to medium sensitive. Natural moisture contents of the silty clay ranged from 18 to 60%.

The results of grain size analyses conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figures B5, B6 and B7 of Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	0
Sand	0-8
Silt	42-83
Clay	17-57

The results of the Atterberg Limits tests conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A and illustrated in Figures B10 and B11 of Appendix B. The results indicated that the liquid limits ranged from 29 to 51% and the plasticity index ranged from 9 to 28%. This indicates the silty clay plasticity ranges from low to high.

## 5.7 Silt

A deposit of grey silt was encountered below the silty clay in Boreholes DCB-02 and DCB-06. The silt contains trace to some sand and clay. Where fully penetrated in DCB-06, the thickness of the silt layer was 0.3 m, and the lower boundary was encountered at Elev. 189.6. DCB-02 was terminated upon auger refusal on probable bedrock below the silt at a depth of 20.3 m or Elev. 184.1. Possible cobbles and boulders were encountered at 19.1 m depth (Elev. 185.3) in Borehole DCB-02 as indicated by the rate and resistance of auger advancement and grinding.

SPT 'N' values recorded in the silt deposit ranged from 6 to 37 blows per 0.3 m penetration, indicating a loose to dense relative density. Natural moisture contents of the silt ranged from 18 to 26%.

The results of grain size analyses conducted on silt samples are provided on the Record of Borehole sheets in Appendix A, and plotted in Figure B8 of Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	0
Sand	0-6
Silt	85-87
Clay	7-15

### 5.8 Clayey Silt with Sand Till

A layer of grey clayey silt with sand till was encountered below the silty clay deposit in Borehole DCB-03. The till contains trace gravel. Borehole DCB-03 was terminated on auger refusal on probable bedrock below the till giving an inferred thickness of 0.7 m and a lower boundary of Elev. 180.1.

One SPT 'N' value recorded in the till was 50 blows per 0.28 m penetration, indicating a hard relative consistency. Measured natural moisture contents ranged from 14 to 24%.

The results of grain size analyses conducted on a sample of the till deposit are provided on the Record of Borehole sheets in Appendix A and plotted in Figure B9 of Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	6
Sand	35
Silt	39
Clay	20

### 5.9 Bedrock

Granite bedrock was encountered in Boreholes DCB-06 and DCB-07 below the silt and silty clay deposits respectively, and was proven by coring. Boreholes DCB-02, DCB-03, DCB-05 and DCB-08 were terminated upon auger refusal on probable bedrock. Bedrock was proven by coring at Borehole 13-3 and 13-4 in the previous investigation. Table 5.1 summarizes the depth to bedrock and the bedrock surface elevations determined in the boreholes.

**Table 5.1: Depth to Bedrock at Borehole Locations**

Foundation Element	Borehole	Depth to Bedrock (m)	Top of Bedrock Elevation (m)	Comment
West Abutment	DCB-02	20.3	184.1	Auger Refusal Core 3m
	13-3	19.6	184.9	
West Pier	DCB-05	7.3	188.9	Auger Refusal Core 3 m
	DCB-06	8.2	189.6	



Foundation Element	Borehole	Depth to Bedrock (m)	Top of Bedrock Elevation (m)	Comment
East Pier	DCB-07	15.2	185.5	Core 3 m Auger Refusal
	DCB-08	11.9	185.8	
East Abutment	DCB-03	26.2	180.1	Auger Refusal Core 3m
	13-4	26.7	179.7	

The bedrock elevation decreases from the east and west piers to the respective abutments. The stratigraphic profile drawing in Appendix H indicates sloping bedrock from the piers to the abutments.

The bedrock is generally described as slightly weathered and grey to pinkish grey in colour. Total Core Recovery (TCR) in the bedrock ranged from 51 to 100%. The Rock Quality Designation (RQD) determined from the recovered cores generally ranged from 0 to 96%, indicating very poor to excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to greater than 10.

The unconfined compressive strength (UCS) of the rock, estimated from the results of point load tests (PLT) conducted on the rock core samples, ranges between 64 and 200 MPa using a conversion factor of 24, indicating a strong to very strong rock. One UCS value of 19.7 MPa was recorded in Borehole DCB-06 at a depth of 11.6 indicating weak rock. The point load test results are included in Appendix B.

## 5.10 Water Levels

The water levels in select boreholes were measured upon completion. However, water was used during the wash-boring and coring operations and therefore the measured water levels may not reflect prevailing groundwater levels at the site. Standpipe piezometers were installed in Boreholes DCB-02, DCB-03, DCB-05 and DCB-07 to monitor groundwater levels after drilling. The water levels measured in the open boreholes upon completion of drilling and in the piezometers are summarized in Table 5.2. Water level measurements in standpipe piezometers installed in the preliminary Boreholes 13-3 and 13-4 are also included.

**Table 5.2: Water Level Measurements**

Borehole	Date	Water Level (m)		Remark
		Depth	Elevation	
13-3	Oct. 1, 2013	2.4	202.1	In VWP (shallow)
	Oct. 1, 2013	3.1	201.4	In VWP (deep)
13-4	Oct. 1, 2013	6.4	200.0	In VWP
DCB-01	Dec. 14, 2015	8.0	196.8	In open borehole
DCB-02	Dec. 16, 2015	7.0	197.4	In piezometer
	Dec. 18, 2015	7.2	197.2	



Borehole	Date	Water Level (m)		Remark
		Depth	Elevation	
DCB-03	Dec. 18, 2015	4.0	202.3	In piezometer
	Feb. 2, 2016	4.5	201.8	
DCB-05	Dec. 20, 2015	0.0	196.2	In piezometer
	Feb. 2, 2016	0.5	195.6	
DCB-07	Dec. 22, 2015	0.3	200.4	In piezometer
	Feb. 2, 2016	1.6	199.1	

Note: VWP = Vibrating Wire Piezometer

The approximate water level in the creek shown on the GA drawing is at Elev. 195.5 m in December 2012. The groundwater levels measured in the piezometers in the current boreholes are up to 6.8 m above the high water level in the creek. The creek and groundwater levels are expected to fluctuate seasonally and subject to precipitation patterns, and may vary from the levels presented above.

## 6. MISCELLANEOUS

Walker Drilling Ltd. of Utopia, Ontario supplied the drilling equipment and conducted the drilling, sampling and in-situ testing operations. A track mounted Diedrich D-50 drill rig was used to drill the east embankment borehole. A portable tripod drill rig was used to drill the southeast pier borehole. A track-mounted Diedrich D-25 drill rig was used for the remaining boreholes.

The drilling and sampling operations were supervised in the field by Ms. Paige Maddock of Thurber. Geotechnical laboratory testing was carried out by Thurber in its MTO-approved laboratory.

Overall supervision of the field program was carried out by Mr. Stephane Loranger, CET. The report was prepared by Ms. Deanna Pizycki, EIT and Mr. Keli Shi, P.Eng.

The report was reviewed by Mr. Alastair Gorman, 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 17B crosses the Duchesnay Creek on a ten-span structure with a total length of 74 m and an overall width of 11 m accommodating two lanes of traffic. The 32 m long main span or the 6th span from the west abutment is a timber truss supported on concrete piers. The existing road grade on the bridge is at approximate Elev. 205 to 206 m.

The existing bridge, which is a heritage structure, was constructed in 1937, and the abutments and piers are founded on timber piles. According to the preliminary foundation investigation and design report, a structural inspection of the bridge indicated that the east abutment has settled as much as 80 mm and several of the piles have moved laterally as much as 250 mm. Wood bracing was installed beneath the bridge to stabilize the structure. MTO maintenance have noted movement in the east creek slopes to the north of the bridge. Immediately south of the bridge is a CP Rail bridge crossing of the Duchesnay Creek.

The General Arrangement drawing indicates that the existing bridge will be replaced on the same horizontal alignment by a three-span structure with a total length of 83 m and an overall width of 13 m. The replacement bridge will use timber glulam girders and conventional abutments founded on tube piles with reinforced concrete socketed into bedrock. Approximately 1 to 1.5 m platform widening will be required on each side of the approach embankments to accommodate the new bridge deck which will be approximately 2 m wider than the existing deck.

Reinforced soil system (RSS) retaining walls are proposed on the south side of the approach embankments. The finished road grade will be approximately 0.3 m above the existing grade at the west abutment and 0.3 m below the existing grade at the east abutment. The valley slopes will be flattened to a maximum inclination of 3H:1V and approximately 2 to 3 m of existing embankment fill will be removed between the existing abutment and the new abutment at the west and east abutments, respectively.

The discussion and recommendations presented in this report are based on the information provided by LEA Consulting Limited, on the subsurface conditions presented in the Geocres report and on the factual data obtained in the course of this investigation.

## **8. STRUCTURE FOUNDATIONS**

In general, the soil stratigraphy below the existing approach embankment fill consists of a layer of silty sand to sandy silt underlain by a firm to very stiff silty clay deposit which overlies a discontinuous layer of silt to clayey silt till over the granitic bedrock. Bedrock was encountered and cored at depths of 8.2 m and 15.2 m (Elev. 189.6 and 185.5) near the proposed west and east piers, respectively. Bedrock was encountered and cored near the proposed abutment locations in the previous investigation at depths of 19.6 m and 26.7 m (Elev. 184.9 and 179.7).

The river level in the preliminary GA was shown at Elev. 195.5 in December 2012. Groundwater levels measured in the previous investigation were at approximately Elev. 202 at the west abutment and Elev. 200 at the east abutment. Groundwater levels measured in the current investigation ranged approximately from Elev. 197.2 to 202.3 near the abutments and Elev. 195.6 to 200.4 near the piers.

Based on the subsurface conditions, initial consideration was given to supporting the replacement bridge on spread footings on native soil or engineered fill, steel H-piles driven to bedrock, drilled-in steel pipe piles socketed into bedrock, and augered caissons (drilled shafts). A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix E.

Recommendations for design of the feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme 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 silty clay deposits. Construction of engineered fill to support the footings is not feasible in view of the depth of loose sand/silt and silty clay to be excavated.

Similarly, supporting the piers on spread footings is not feasible due to the low geotechnical resistance available in the underlying silty clay. Footings will also have to be protected from scour and erosion.

Spread footings are not recommended to support the new structure.

### **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 refusal on bedrock.

Steel HP310x110 piles founded on the bedrock should be designed for a factored axial geotechnical resistance at ULS of 2,000 kN per pile. For an HP360x132 steel pile driven to bedrock, the factored geotechnical resistance at ULS is 2,400 kN. The geotechnical resistance at SLS does not govern for steel piles driven to bedrock.

The estimated tip elevations of piles driven to the bedrock surface, are presented in Table 8.1.

**Table 8.1 – Estimated Pile Tip Elevation for HP310x110**

Foundation Element	Borehole	Estimated Pile Tip Elevation (m)
West Abutment	DCB-02	184.1 *
	BH13-3	184.9
West Pier	DCB-05	188.9 *
	DCB-06	189.6
East Pier	DCB-07	185.5
	DCB-08	185.8 *
East Abutment	DCB-03	180.1 *
	BH13-4	179.7

Note: \* Auger or casing refusal on probable bedrock.

Rock fill containing cobbles and boulders was encountered and cored through in the east abutment boreholes (DCB-03 and 13-4). Cobbles were encountered in the existing fill at the east pier location. The rock fill noted from Elev. 205.2 to 201.3 at the east abutment could potentially interfere with pile installation. It is recommended that the rock fill be removed prior to driving piles and it is anticipated that this will be achieved by excavation for abutment construction.

## 8.2.2 Pile Tips

Pile tip protection is recommended for driven H-piles to prevent pile damage when setting the piles on bedrock or if cobbles or boulders are encountered. The use of rock points (Titus Rock Injector or equivalent) is recommended for driving piles to the sloping bedrock surface.

## 8.2.3 Pile Installation

The layout of the H-piles must take account of the existing foundation timber piles and concrete pier caps. Consideration could be given to removing existing foundation elements where conflicts occur.

Pile installation must be in accordance with OPSS 903 and taking note of the requirement to seat on bedrock. The appropriate pile driving note is “Piles to be driven to bedrock”.

Cobbles, boulders and/or rock fill may be encountered when driving piles through the existing fill. Probable cobbles and boulders were encountered in the silt at 19.1 m depth in Borehole DCB-02. The Contract Documents should contain an NSSP alerting bidders to the presence of the cobbles, boulders and rock fill within the existing embankment and in the foundation soil. Suggested wording for an NSSP addressing this issue is included in Appendix F.

## 8.3 Drilled-in Steel Pipe Pile

### 8.3.1 Axial Resistance

The replacement bridge may be supported on drilled-in steel pipe piles socketed into the granite bedrock and filled with concrete. It is recommended that the pipe piles be advanced a minimum 1.0 m into the bedrock to penetrate any fractured rock and cobbles/boulders at the bedrock surface.

The axial capacity of concrete-filled pipe piles socketed into bedrock will be governed by the structural resistance of the composite pile section and not by the geotechnical resistance of the bedrock. The factored resistances recommended for various pipe pile sections are presented in Table 8.2.

**Table 8.2 – Factored Resistance of Drilled-in Pipe Pile**

Pipe Pile Section		Factored Resistance (kN)
Outside Diameter (mm)	Wall Thickness (mm)	
356	12.7	2,900
406	12.7	3,500
508	12.7	4,900
610	12.7	6,500

The resistance values presented above assume a steel yield strength of 310 MPa and a concrete compressive strength of 35 MPa. If the soil-structure interaction analysis indicates that fixity is not being achieved within the depth of the soil, Thurber should be provided with the moments in the piles to allow them to assist in determining the depth of fixity in the bedrock.

### 8.3.2 Drilled-in Pipe Pile Installation

Installation of pipe piles must be in accordance with OPSS.PROV 903.

Drilled-in pipe piles must be installed in rock sockets that are clean and free of drilling cuttings or other debris. The method of installation of the pipe piles in order to achieve a clean socket is the responsibility of the Contractor. However, one option for installing pipe piles is to drill them in using a concentric drilling method such as the Symmetrix system by Atlas Copco. The Contractor's drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles, boulders or rock fill in the embankment fill and overburden soils. Care must be exercised while drilling into the bedrock. The drilling methodology used must be capable of advancing the pile without disturbing or fracturing the bedrock at the base of the pile.

Since the rock cutting shoe at the tip of a pipe pile will be slightly larger in diameter than the outside diameter of a pipe pile, there will be a small gap between the rock socket wall and the pipe pile shaft. It is recommended that the tip of casing be installed to 300 mm below the bedrock surface and a pilot bit be further advanced from inside the casing to achieve socket length required for structural fixity prior to installing reinforcement and pouring concrete.

During and subsequent to installation, the pipe pile will be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

An NSSP addressing the above issues is included in Appendix F.

## 8.4 Lateral Pile Resistance

### 8.4.1 Lateral Resistance in Soil

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 relative 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 (m)

The above equations and recommended parameters in Table 8.3 below may be used to analyse the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance.

**Table 8.3 – Soil Parameters for Lateral Pile Resistance**

Soil Unit	Elevation (m)		$\gamma'$ (kN/m³)	$n_h$ (kN/m³)	$K_p$	$S_u$ (kPa)
	Top	Bottom				
West Abutment (DCB-02, 13-3)						
Silty Clay	199.5 *	188.0	8	-	-	50
Silt (Compact)	188.0	186.0	10	2,500	3.0	-
Silt (Dense)	186.0	184.0 (Bedrock)	11	3,500	3.2	-



Soil Unit	Elevation (m)		$\gamma'$ (kN/m³)	$n_h$ (kN/m³)	$K_p$	$S_u$ (kPa)
	Top	Bottom				
West Pier (DCB-05, DCB-06)						
Fill (Loose)	198.5 **	196.5	19	2,000	2.9	-
Sand and Silt (Loose)	196.5	195.5	20	2,000	2.9	-
Silty Clay	195.5	190.0	8	-	-	50
Silt (Loose)	190.0	189.6 (Bedrock)	10	2,500	3.0	-
East Pier (DCB-07, DCB-08)						
Fill (Loose to Compact)	197.0 **	195.5	19	2,000	2.9	-
Silty Clay	195.5	185.5 (Bedrock)	8	-	-	40
East Abutment (DCB-03, 13-4)						
Sandy Silt (Loose)	200.5 *	196.5	9	2,000	2.9	-
Silty Clay	196.5	181.0	8	-	-	50
Sandy Clayey Silt Till	181.0	179.7 (Bedrock)	10	-	-	75

Note: \* Underside of pile cap at abutments.

\*\* Assumed finished ground surface at piers along bridge centreline.

The spring constant,  $K_s$ , for analysis may be obtained by the expression,  $K_s = k_s L D$  (kN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction (kN/m<sup>3</sup>),  $D$  is the pile width (m) and  $L$  is the 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.

The modulus of subgrade reaction and ultimate lateral resistance 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 8.4. Intermediate values may be obtained by linear interpolation.

**Table 8.4 – Subgrade Reaction Reduction Factors for Pile Spacing**

Condition	Pile Spacing (Centre to Centre)	Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> 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. Additional lateral resistance could also be provided by socketing the piles into bedrock. However, considering the depth to bedrock at this site, socketing of the piles is not expected to be an efficient means of developing lateral resistance.



#### 8.4.2 Lateral Resistance in Rock Socket

The ultimate passive resistance that can be mobilized by the embedded portion of a pile socketed in the bedrock may be assumed to be constant with depth and is given by:

$$P_p = 6 C D L$$

Where

$$C = 3,500 \text{ kPa (undrained shear strength of rock mass)}$$
$$D = \text{Socket diameter (m)}$$
$$L = \text{Depth of socket in rock (m)}$$

#### 8.5 Downdrag

The GA drawing indicates that the finished road grade will be approximately 0.3 m above the existing grade at the west abutment and 0.3 m below the existing grade at the east abutment. In addition, approximately 1 to 1.5 m platform widening will be required on each side of the approach embankments to accommodate the new wider bridge deck. However, the finished valley slopes will be flattened to a maximum inclination of 3H:1V and up to approximately 2 m and 3 m of existing fill will be removed between the existing abutment and the new abutment at the west and east abutments, respectively.

In view of the above, the foundation soils at the new abutments will likely be in a net unloading condition. Therefore, downdrag is not considered to be an issue for piles installed in bedrock.

#### 8.6 Integral Abutment Considerations

The soil conditions at this site are suitable for the design of an integral abutment structure. The pile flexibility requirements of this design must be checked by the structural designer, taking account of the lateral resistance of the soil surrounding the piles.

#### 8.7 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, drilled-in pipe piles socketed in bedrock and H-piles driven to bedrock are both suitable foundation options at this site. Driven H-piles to bedrock at the east abutment may require predrilling to install piles through the rock fill.

#### 8.8 Frost Cover

The depth of frost penetration at this site is 2.0 m. The base of pile caps must be provided with a minimum of 2.0 m of earth cover as protection against frost action.

### 9. EXCAVATION AND DEWATERING

The creek level was reported at Elev. 195.47 in December 2012. Where excavations penetrate below the water level, the Contractor must implement dewatering procedures.

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 may be classified as Type 3 soil above the water table and as Type 4 soil below the water table. Flatter slopes may be required at locations where water seepage affects surficial stability.

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

The selection of the method of excavation is the responsibility of the Contractor and must be based on his equipment, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable. Provision must be made for the handling of pavement materials, potential obstructions in the fill, cobbles and boulders, and rock fill.

## 10. ROADWAY PROTECTION

If roadway protection is required, the temporary excavation support system must be designed and constructed in accordance with OPSS.PROV 539. If roadway protection is required in the vicinity of the east abutment, the presence of rock fill may impede installation. In this case, further investigation of the extent of the rock fill may be required. The Contractor should select the wall type and design taking into account the soil conditions encountered in the boreholes.

The following parameters apply for design of the temporary shoring system:

$\gamma$	=	21 kN/m <sup>3</sup>	(bulk unit weight for fill)
	=	20 kN/m <sup>3</sup>	(bulk unit weight for native sand/silt)
$\gamma'$	=	11 kN/m <sup>3</sup>	(submerged unit weight for fill)
	=	10 kN/m <sup>3</sup>	(submerged unit weight for native sand/silt)
$K_a$	=	0.31	(active pressure coefficient for fill)
	=	0.35	(active earth pressure coefficient for native sand/silt)
$K_p$	=	3.3	(passive pressure coefficient for fill)
	=	2.9	(passive earth pressure coefficient for native sand/silt)

For free draining shoring systems, the short-term groundwater level should be assumed at the base of excavation both in front of and behind the wall.

The actual pressure distribution acting on the shoring system is a function of construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

The design of roadway protection should be the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such design.

## 11. APPROACH EMBANKMENTS

Based on the GA drawing, there will be an average grade raise of 0.5 m between Sta. 10+860

and 10+975 at the west approach and a grade lowering up to 0.3 m at the east approach. Additional fill placement is required to widen the approach embankments to accommodate the replacement bridge. Embankment settlements associated with the proposed grade raise and platform widening is estimated to not exceed 25 mm at both abutments and will be essentially complete at the end of construction.

The preliminary GA drawing indicates that the forward slopes on both sides of the creek will be flattened to no steeper than 3H:1V. The foundation soils governing stability of the approach embankments consist of loose to compact sand and silt and firm to stiff silty clay. Inclination of the side slopes of the widened approaches should be no steeper than 2H:1V.

Global stability analyses were carried out to assess the stability of approach slopes under the existing conditions and for the proposed 3H: 1V flattening which will unload the forward slopes. The stability analyses were carried out using commercially available slope stability program GEO-SLOPE with application of Morgenstern-Price method. The geotechnical model and results of the analyses are shown on Figures 1 to 6 in Appendix G. The computed factors of safety are summarized in Table 11.1. It should be noted that an effective cohesion ( $c'$ ) of 3 kPa and an effective friction angle ( $\phi'$ ) of  $27^\circ$  were used for the silty clay that governs long term approach embankment stability.

**Table 11.1 – Computed Factors of Safety for Approach Embankments**

Abutment	Condition	Factor of Safety	Figure No. (Appendix G)
East	Existing (drained)	1.35	1
	Short term (undrained)	1.30	3
	Long term (drained)	1.51	5
West	Existing (drained)	1.97	2
	Short term (undrained)	2.05	4
	Long term (drained)	2.06	6

Analyses for the west abutment indicated stable conditions for the existing approach embankment slope and proposed flattening. However, the analyses conducted for the existing east approach embankment indicated a factor of safety of 1.35 for drained conditions, which is representative of the current ground conditions and suggestive of the observed settlement and pile movement at the east abutment. The proposed slope flattening at the east abutment will increase the factors of safety to 1.30 for short term conditions and 1.51 for long term conditions, respectively. The above factors of safety for the flattened slopes are considered acceptable.

### 11.1 Reinforced Soil System (RSS) Walls

The General Arrangement drawing indicates that retained soil system (RSS) walls will be used on the south side of the approach embankments. The lengths of the RSS walls are shown to be 15 m and 11 m at the west approach and east approach, respectively.

In general, RSS walls used in conjunction with the new abutments must be “High Performance”

and “High Appearance”. The contract drawings should include information on the longitudinal alignment of each RSS wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

To provide an acceptable foundation performance, the RSS mass must be founded on competent native soils or engineered fill. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

The borehole information indicates that the soil conditions at the proposed wall base levels generally consist of loose sand and silt underlain by firm silty clay at the west approach and compact sand over loose sandy silt at the east approach. To provide a uniformly competent subgrade, it is recommended that all RSS walls be founded on a minimum 500 mm thick engineered fill pad. The top of the engineered fill pad should be a minimum 1 m below the final ground surface at the wall location. The engineered fill should consist of OPSS Granular “A” compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered fill pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.

Walls founded on engineered fill at the proposed wall locations should be designed for a factored geotechnical resistance at ULS of 180 kPa and a geotechnical resistance at SLS of 120 kPa. The geotechnical resistance at SLS corresponds to settlement up to 25 mm at the base of the RSS wall. If higher geotechnical resistances are required for the design of RSS walls, the design team should be consulted regarding possible solutions.

The geotechnical resistances provided above are for concentric, vertical loading and horizontal ground surface in front of the wall. The effects of load inclination and eccentricity need to be considered per the CHBDC. The resistance values assume that the RSS wall reinforcement will extend a distance behind the wall face of approximately 70% of the wall height. In the event of sloping ground in front of the wall, the resistance values need to be further reduced.

Any topsoil and soft/loose fill or native material should be stripped from the footprint of the RSS. All new embankment fill must be compacted in accordance with OPSS.PROV 501.

The RSS wall must also be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall on engineered fill may be estimated using an ultimate friction coefficient of 0.5. The internal stability of the RSS wall should be analysed by the supplier/designer of the proprietary product selected for this site.

## **12. SCOUR AND EROSION PROTECTION**

Erosion protection should be provided along any soil surfaces that may be in contact with the creek flow.

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

## **13. 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 (see Table 13.1)  
 $\gamma$  = unit weight of retained soil (see Table 13.1)  
 $h$  = 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 13.1.

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 active and passive earth pressure coefficients in Table 13.1 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).

**Table 13.1 – Coefficients of Lateral Earth Pressure (K)**

Loading Condition	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 Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active $K_A$ (Unrestrained Wall)	0.27	0.40*	0.31	0.48*
At-rest $K_0$ (Restrained Wall)	0.43	-	0.47	-
Passive $K_P$	3.7	-	3.3	-

\* For wing walls

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

## 14. SEISMIC CONSIDERATIONS

According to Clause 4.4.4 of the CHBDC, an earthquake with a 2475-year return period or 2%

probability of exceedance in 50 years should be used for seismic design. The peak ground acceleration (PGA) associated with the design earthquake is 0.154g for Site Class C.

Based on the encountered soil conditions, this site is assessed to be Site Class 'D' for seismic site response according to Table 4.1 of the CHBDC. The above PGA value should be modified by a site coefficient of 1.19 based on Table 4.8 of the CHBDC.

In accordance with Clause 4.6.5 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 14.1 may be used:

**Table 14.1 – Earth Pressure Coefficient for Earthquake Loading ( $K_E$ )**

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 $\phi = 32^\circ$ ; $\gamma = 21.2 \text{ kN/m}^3$
Active ( $K_{AE}$ )*	0.35	0.39
At Rest ( $K_{0E}$ )**	0.67	0.71
Passive ( $K_{PE}$ )	3.4	3.0

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

\*\* After Woods (1973).

The loose sandy silt layer underlying the fill and/or sand deposit may be susceptible to liquefaction under seismic loading. However, considering the normal water level in the creek and low seismic activity in the area, liquefaction of the foundation soils is not a concern.

## 15. CONSTRUCTION CONCERNS

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

- The Contractor's attention must be drawn to the presence of rock fill at the east abutment as this may impede the installation of piles unless it is first excavated. Alternatively, the Contractor may elect to drill through the rock fill.
- When selecting construction equipment and methodology, the Contractor must take account of the ground conditions. He must assess the ability of the soils on site to support his equipment and temporary structures or fill required to construct the project, e.g. crane pads. The placement of heavy equipment or fill on the valley slopes is a particular concern in light of the potential instability of the slopes. The design of the temporary works, including support of heavy equipment is the responsibility of the Contractor but he must be alerted to the requirement to consider slope stability. Suggested wording for an NSSP addressing this issue is included in Appendix F.

## 16. CLOSURE

Engineering analysis and preparation of the design report were carried out by Mr. Keli Shi, P.Eng. The report was reviewed by Mr. Alastair Gorman, P.Eng., and Dr. P. K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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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

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


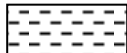



 Water Level  
 Shear Strength Determination by Pocket Penetrometer

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

# UNIFIED SOILS CLASSIFICATION

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

## EXPLANATION OF ROCK LOGGING TERMS

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

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Very thinly bedded	20 to 60mm				
Laminated	6 to 20mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Thinly Laminated	Less than 6mm				

<u>TERMS</u>					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

# RECORD OF BOREHOLE No DCB-01

1 OF 2

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 133.4 E 304 504.3 ORIGINATED BY PSM  
HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2015.12.14 - 2015.12.14 CHECKED BY DJP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE						
204.8	GROUND SURFACE													
0.0	ASPAHLT:(100mm)		1	GS										GR SA SI CL
0.1	Gravelly <b>SAND</b> to <b>SAND</b> , trace silt Dense to Compact Brown Moist (FILL)													
		2	SS	46										
		3	SS	22										
203.0			4	SS	13									
1.8	<b>SILT</b> and <b>SAND</b> , trace clay Compact to Loose Brown to Grey Wet													
		5	SS	7										
		6	SS	4										
200.7														
4.1	Silty <b>CLAY</b> , trace sand Firm to Very Stiff Grey													
		7	SS	4										
			8	SS	18									
			9	SS	4									

Continued Next Page

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

# RECORD OF BOREHOLE No DCB-01

2 OF 2

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 133.4 E 304 504.3 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2015.12.14 - 2015.12.14 CHECKED BY DJP

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																
	BOREHOLE OPEN TO 9.8m AND WATER LEVEL AT 8.0m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.1m AND ASPHALT COLD PATCH TO SURFACE.																

ONTMT4S 9485.GPJ 2015TEMPLATE(MTO).GDT 3/1/16

# RECORD OF BOREHOLE No DCB-02

1 OF 3

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 114.8 E 304 514.2 ORIGINATED BY PSM  
HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2015.12.15 - 2015.12.16 CHECKED BY DJP

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			20    40    60    80    100	W <sub>P</sub> W      W <sub>L</sub>	20    40    60	GR    SA    SI    CL			
204.4	GROUND SURFACE													
0.0	ASPHALT:(150mm)													
0.2	Gravelly <b>SAND</b> to <b>SAND</b> , trace silt Compact to Loose Brown Moist (FILL)		1	SS	27		204							30    63    7 (SI+CL)
			2	SS	18									
			3	SS	9									
202.3														
2.1	<b>SILT</b> and <b>SAND</b> , trace to some clay Compact to Loose Brown to Grey Wet		4	SS	20		202							0    35    53    12
			5	SS	17		201							
			6	SS	4									
			7	SS	23									
199.9														
4.5	Silty <b>CLAY</b> , trace sand Stiff to Very Stiff Grey						200							0    0    83    17
							199							
			8	SS	7		198							
			1	TW			197							
							196							
			9	SS	4		195							0    8    42    50

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

## METRIC

[illegible]


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

# RECORD OF BOREHOLE No DCB-02

3 OF 3

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 114.8 E 304 514.2 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2015.12.15 - 2015.12.16 CHECKED BY DJP

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		16	SS	37												
184.1 20.3	END OF BOREHOLE AT 20.3m UPON SPOON AND CASING REFUSAL ON PROBABLE BEDROCK. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m)  Dec16/2015      7.0      197.4 Dec18/2015      7.2      197.2																



## METRIC

[illegible]

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

# RECORD OF BOREHOLE No DCB-03

2 OF 3

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 055.7 E 304 575.3 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2015.12.16 - 2015.12.18 CHECKED BY DJP

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						
196.1	Continued From Previous Page													
10.2	Silty <b>CLAY</b> Very Stiff Grey		6	SS	27		196							0 0 51 49
							195							
	Beocming Firm to Stiff		7	SS	9		194							
							193	4.0						
			1	TW			192	3.0						
			8	SS	6		191							
							190							
			9	SS	4		189						0 0 47 53	
							188	4.0						
			2	TW			187	5.0						

Continued Next Page

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

20  
15  
10  
5  
0  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No DCB-03

3 OF 3

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 055.7 E 304 575.3 ORIGINATED BY PSM  
HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2015.12.16 - 2015.12.18 CHECKED BY DJP

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		WATER CONTENT (%)				
	Continued From Previous Page							20 40 60 80 100		W <sub>p</sub> W      W <sub>L</sub>			GR   SA   SI   CL	
186.2														
20.1	Becoming reddish brown, with silty sand interbeds		10	SS	7		186							
			11	SS	8		185							0   0   56   44
							184							
							183							
							182							
			12	SS	14		181							
180.8														
25.5	Clayey <b>SILT</b> , with sand, trace gravel Very Stiff Grey (TILL)		13	SS										6   35   39   20
180.1														
26.2	END OF BOREHOLE AT 26.2m UPON REFUSAL ON PROBABLE BEDROCK. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m)  Dec18/2015      4.0      202.3 Feb02/2016      4.5      201.8													



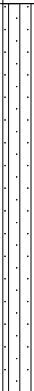
ONTMT4S 9485.GPJ 2015TEMPLATE(MTO).GDT 3/1/16

# RECORD OF BOREHOLE No DCB-04

1 OF 2

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 046.6 E 304 594.3 ORIGINATED BY PSM  
HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2016.02.02 - 2016.02.02 CHECKED BY DJP

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 (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
206.6	GROUND SURFACE							20	40	60	80	100				
0.0	ASPHALT: (150mm)															
0.2	SAND, trace to some gravel, trace to some silt Very Dense to Compact Brown Moist (FILL)		1	SS	77		206								17 72 11 (SI+CL)	
			2	SS	14											
205.0							205									
1.6	SAND, trace to some silt Compact Brown to Grey Moist to Wet		3	SS	15											
			4	SS	18		204									
			5	SS	23		203									
							202									
							201									
			7	SS	14		200								0 80 17 3	
199.4																
7.2	Sandy SILT, trace clay Loose Grey Wet		8	SS	7		199									
							198									
			9	SS	7		197								0 34 57 9	
196.8																
9.8	END OF BOREHOLE AT 9.8m.															

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No DCB-04

2 OF 2

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 046.6 E 304 594.3 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2016.02.02 - 2016.02.02 CHECKED BY DJP

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																
	BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.2m AND ASPHALT COLD PATCH TO SURFACE.																

# RECORD OF BOREHOLE No DCB-05

1 OF 1

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 108.4 E 304 540.7 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2015.12.20 - 2015.12.20 CHECKED BY DJP

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									
196.2	GROUND SURFACE																		
0.0	<b>SAND</b> , some organics Loose Brown to Dark Brown Moist (FILL)		1	SS	4														
			2	SS	4														
194.7																			
1.4	<b>SILT</b> and <b>SAND</b> to Silty <b>SAND</b> , trace clay Loose Brown to Grey Wet		3	SS	3											0 47 46 7			
	Wood mulch/organics at 2.3m (600mm)		4	SS	8											0 72 24 4			
	Trace wood mulch at 3.3m		5	SS	8														
			6	SS	4														
			7	SS	7														
188.9																			
7.3	END OF BOREHOLE AT 7.3m UPON AUGER REFUSAL ON PROBABLE BEDROCK. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m)  Dec20/2015    0.0            196.2 Feb02/2016    0.6            195.6																		

ONTMT4S 9485.GPJ 2015TEMPLATE(MTO).GDT 3/1/16

# RECORD OF BOREHOLE No DCB-06

1 OF 2

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 097.8 E 304 532.2 ORIGINATED BY PSM  
HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2015.12.19 - 2015.12.19 CHECKED BY DJP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
197.8	GROUND SURFACE							20   40   60   80   100		W <sub>p</sub> W      W <sub>L</sub>			GR   SA   SI   CL	
0.0	<b>SAND</b> , trace silt, trace gravel, trace organics Loose Brown to Dark Brown Moist to Wet (FILL)		1	SS	6									
			2	SS	7									
196.3														
1.5	<b>SILT</b> and <b>SAND</b> , trace clay Compact Grey Wet		3	SS	10									0   44   48   8
195.5														
2.3	Silty <b>CLAY</b> , trace sand Firm to Stiff Grey Wet		4	SS	12									0   7   57   36
			5	SS	6									
			6	SS	7									0   0   43   57
			1	TW										
191.1														
6.7	Becoming reddish brown, with silty sand interbeds													
189.9														
7.9	<b>SILT</b> , some clay		7	SS	6									0   0   85   15
189.6	Loose Grey Wet												FI	RUN #1 TCR=97% SCR=66% RQD=50%
8.2	<b>GRANITE</b> slightly weathered, strong to very strong, grey to pinkish grey		1	RUN									2	
			2	RUN									7	RUN #2 TCR=94% SCR=83% RQD=12%
			3	RUN									2	
													>10	RUN #3 TCR=100% SCR=33% RQD=0%
													5	
													2	

Continued Next Page

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

ONTMT4S 9485.GPJ 2015TEMPLATE(MTO).GDT 3/1/16

# RECORD OF BOREHOLE No DCB-06

2 OF 2

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 097.8 E 304 532.2 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2015.12.19 - 2015.12.19 CHECKED BY DJP

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									
	Continued From Previous Page		4	RUN													
			5	RUN													
186.1							187										
11.7	END OF BOREHOLE AT 11.7m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO GROUND SURFACE.																



# RECORD OF BOREHOLE No DCB-07

1 OF 3

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 081.3 E 304 568.7 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2015.12.21 - 2015.12.22 CHECKED BY DJP

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			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)				
200.7	GROUND SURFACE													
0.0	TOPSOIL: (50mm)													
	SAND to Gravelly SAND, some silt, trace gravel, occasional cobbles Loose to Compact Brown Moist (FILL)		1	SS	3									
			2	SS	14									
			3	SS	17									
198.6														
2.1	Sandy SILT, some clay Loose Grey Wet		4	SS	5									0 20 66 14
			5	SS	6									
196.6														
4.1	Silty CLAY, trace sand Stiff to Very Stiff Grey		6	SS	3									0 0 60 40
			7	SS	12									
			8	SS	4									
			1	TW										

Continued Next Page

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

# RECORD OF BOREHOLE No DCB-07

2 OF 3

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 081.3 E 304 568.7 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2015.12.21 - 2015.12.22 CHECKED BY DJP

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 (%)		
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL						×	LAB VANE	
	Continued From Previous Page							20	40	60	80	100	20	40	60	GR	SA	SI	CL
			9	SS	9		190									0	0	50	50
							189				5.0								
188.2			10	SS	11		188												
12.5	Becoming reddish brown, with silty sand interbeds						187												
			11	SS	11		186												
185.5							185												
15.2	GRANITE slightly weathered, strong to very strong, grey to pinkish grey		1	RUN			184												
			2	RUN			183												
			3	RUN															
			4	RUN															
182.0							182												
18.7	END OF BOREHOLE AT 18.7m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.																		
	WATER LEVEL READINGS: DATE      DEPTH (m)    ELEV. (m)																		

Continued Next Page

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

RECORD OF BOREHOLE No DCB-07

3 OF 3

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 081.3 E 304 568.7 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2015.12.21 - 2015.12.22 CHECKED BY DJP

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			20	40	60	80	100					
	Continued From Previous Page																
	Dec22/2015 0.3 200.4																
	Feb02/2016 1.6 199.1																

ONTMT4S 9485.GPJ 2015TEMPLATE(MTO).GDT 3/1/16

## 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			SHEAR STRENGTH kPa			w <sub>p</sub>	w	w <sub>L</sub>		
								○ UNCONFINED    + FIELD VANE							
								● QUICK TRIAXIAL    × LAB VANE							
197.7 0.0	GROUND SURFACE														
	Gravelly SAND to SAND, some gravel, trace silt, trace organics (roots and rootlets) Compact Brown Moist to Wet (FILL)		1	SS	10										
			2	SS	13										
			3	SS	10										
195.7 2.0	Silty CLAY Firm to Very Stiff Grey		4	SS	10										
			5	SS	5										
			6	SS	5										
			7	SS	5										
			8	SS	4										
189.0 8.7	Becoming reddish brown with sandy silt interbeds		9	SS	7										

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

ONTMT4S 9485.GPJ 2015TEMPLATE(MTO).GDT 3/1/16

# RECORD OF BOREHOLE No DCB-08

2 OF 2

METRIC

W.P. 5120-07-01 LOCATION Duchesnay Creek N 5 132 072.8 E 304 558.0 ORIGINATED BY PSM  
 HWY 17B BOREHOLE TYPE Tripod with BW Casing COMPILED BY AN  
 DATUM Geodetic DATE 2016.01.20 - 2016.01.20 CHECKED BY DJP

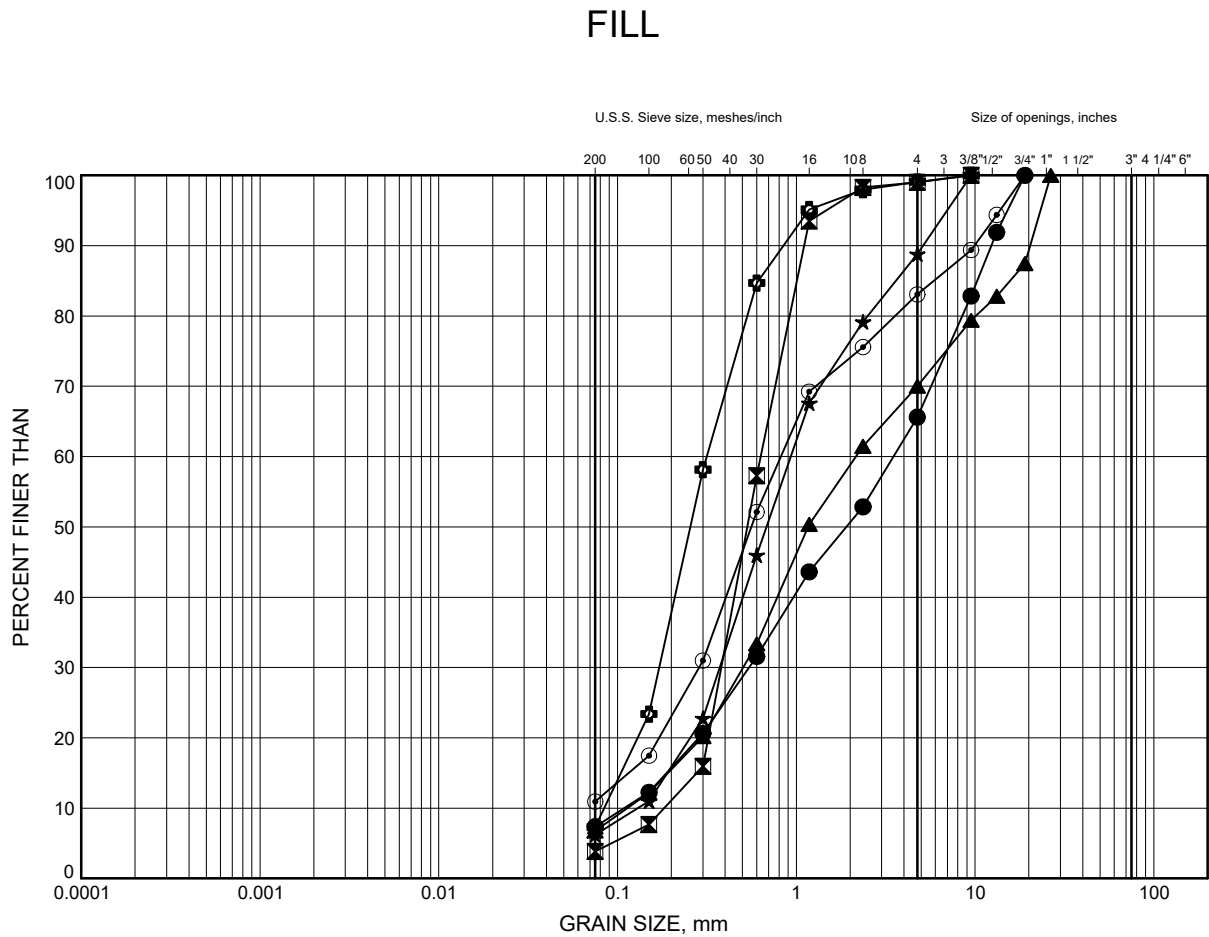
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page		10	SS	13		187										
			11	SS	12												
185.8			12	SS	100/		186										
11.9	END OF BOREHOLE AT 11.9m ON PROBABLE BEDROCK. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.				0.025												



**Appendix B**  
**Laboratory Test Results**

# Duchesnay Creek GRAIN SIZE DISTRIBUTION

FIGURE B1



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCB-01	0.15	204.65
⊠	DCB-01	1.45	203.35
▲	DCB-02	1.07	203.33
★	DCB-03	0.47	205.83
⊙	DCB-04	0.46	206.14
⊞	DCB-08	1.75	195.94

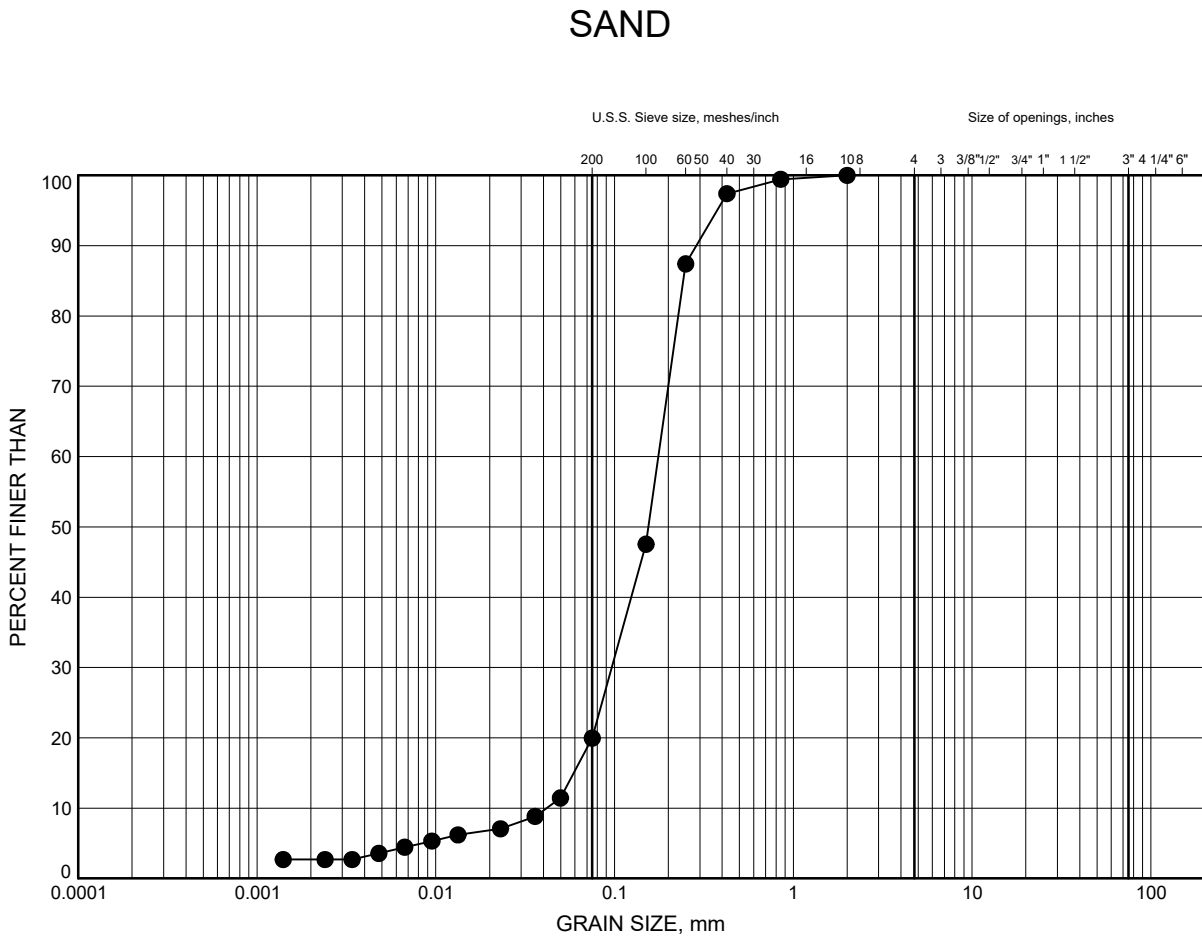
Date March 2016  
W.P. 5120-07-01



Prep'd AN  
Chkd. DJP

# Duchesnay Creek GRAIN SIZE DISTRIBUTION

FIGURE B2



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCB-04	6.40	200.20

Date March 2016  
W.P. 5120-07-01



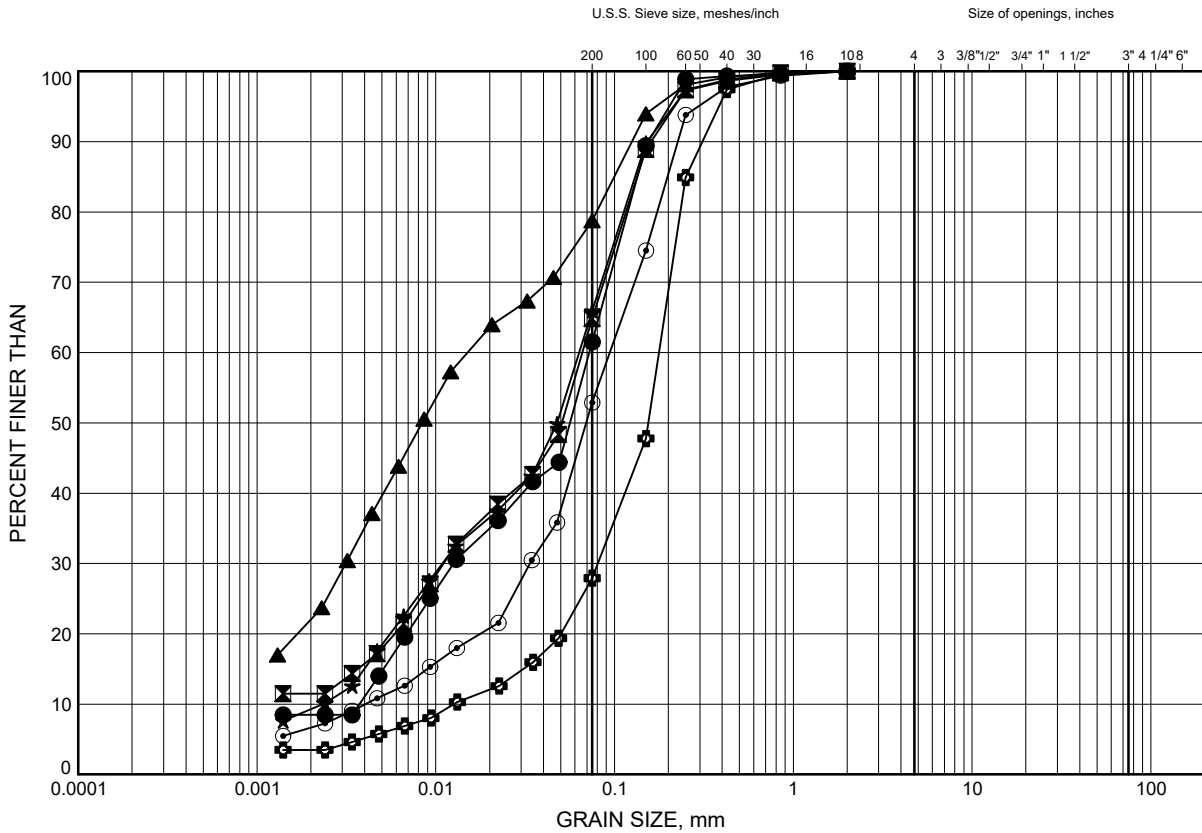
Prep'd AN  
Chkd. DJP



# Duchesnay Creek GRAIN SIZE DISTRIBUTION

FIGURE B3

## 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)
●	DCB-01	2.74	202.05
⊠	DCB-02	2.59	201.81
▲	DCB-03	8.03	198.27
★	DCB-04	9.43	197.16
⊙	DCB-05	1.83	194.37
⊕	DCB-05	2.41	193.78

Date March 2016  
W.P. 5120-07-01

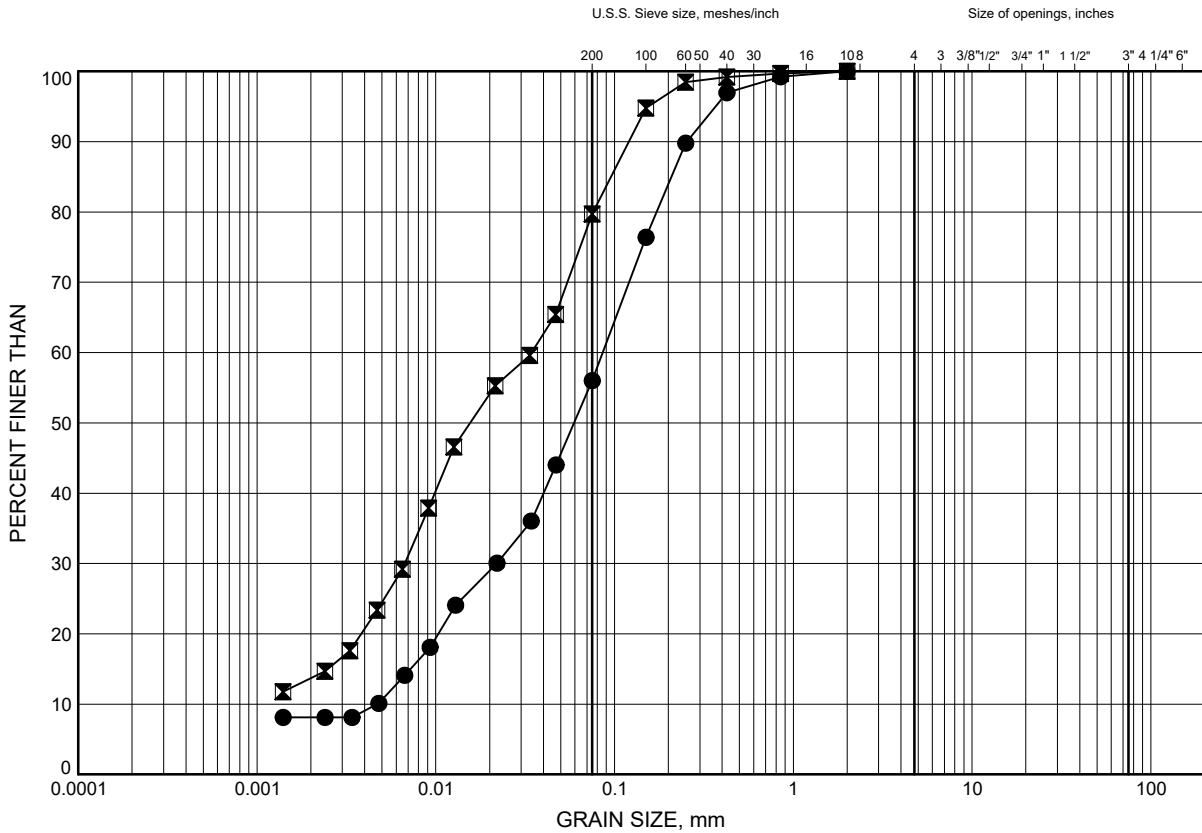


Prep'd AN  
Chkd. DJP

# Duchesnay Creek GRAIN SIZE DISTRIBUTION

FIGURE B4

## 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)
●	DCB-06	1.83	195.97
⊠	DCB-07	2.59	198.11

Date March 2016  
W.P. 5120-07-01

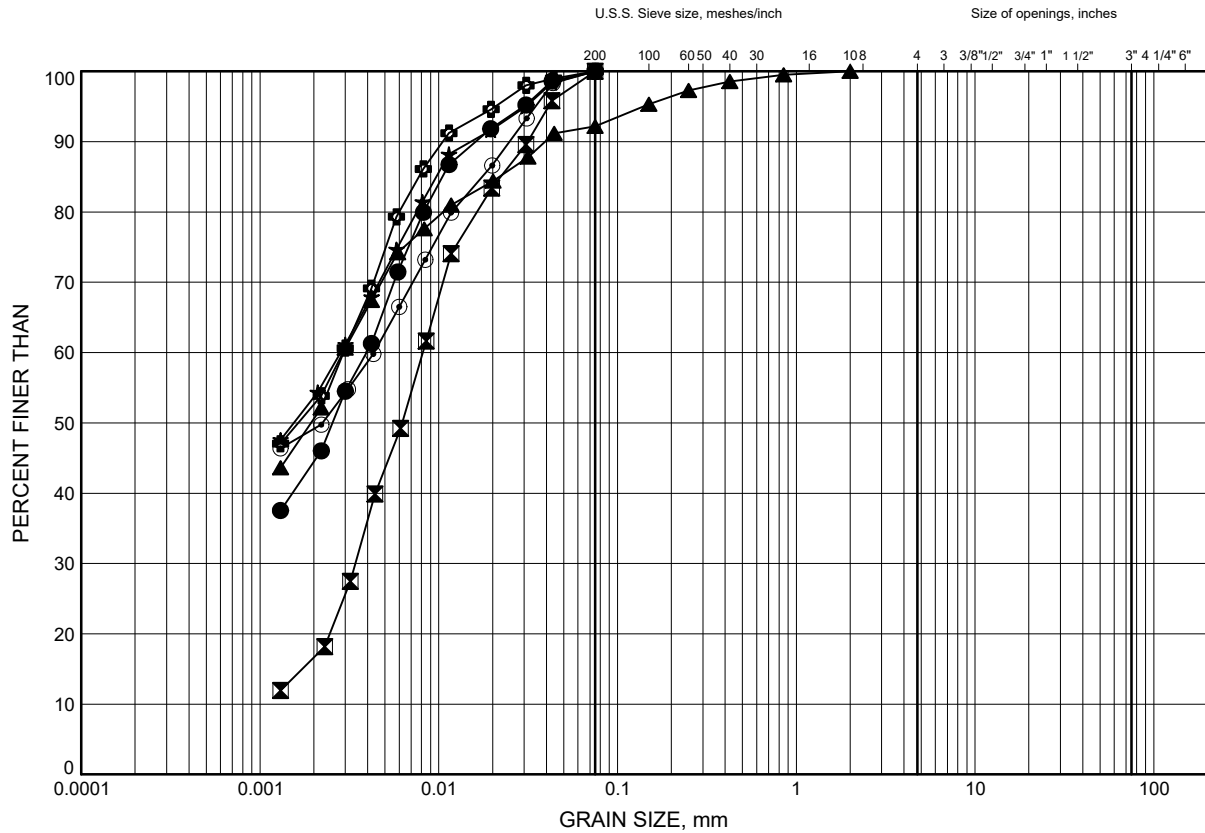


Prep'd AN  
Chkd. DJP

# Duchesnay 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)
●	DCB-01	4.88	199.92
⊠	DCB-02	4.88	199.52
▲	DCB-02	9.45	194.95
★	DCB-02	14.02	190.38
⊙	DCB-03	10.97	195.32
⊕	DCB-03	17.07	189.23

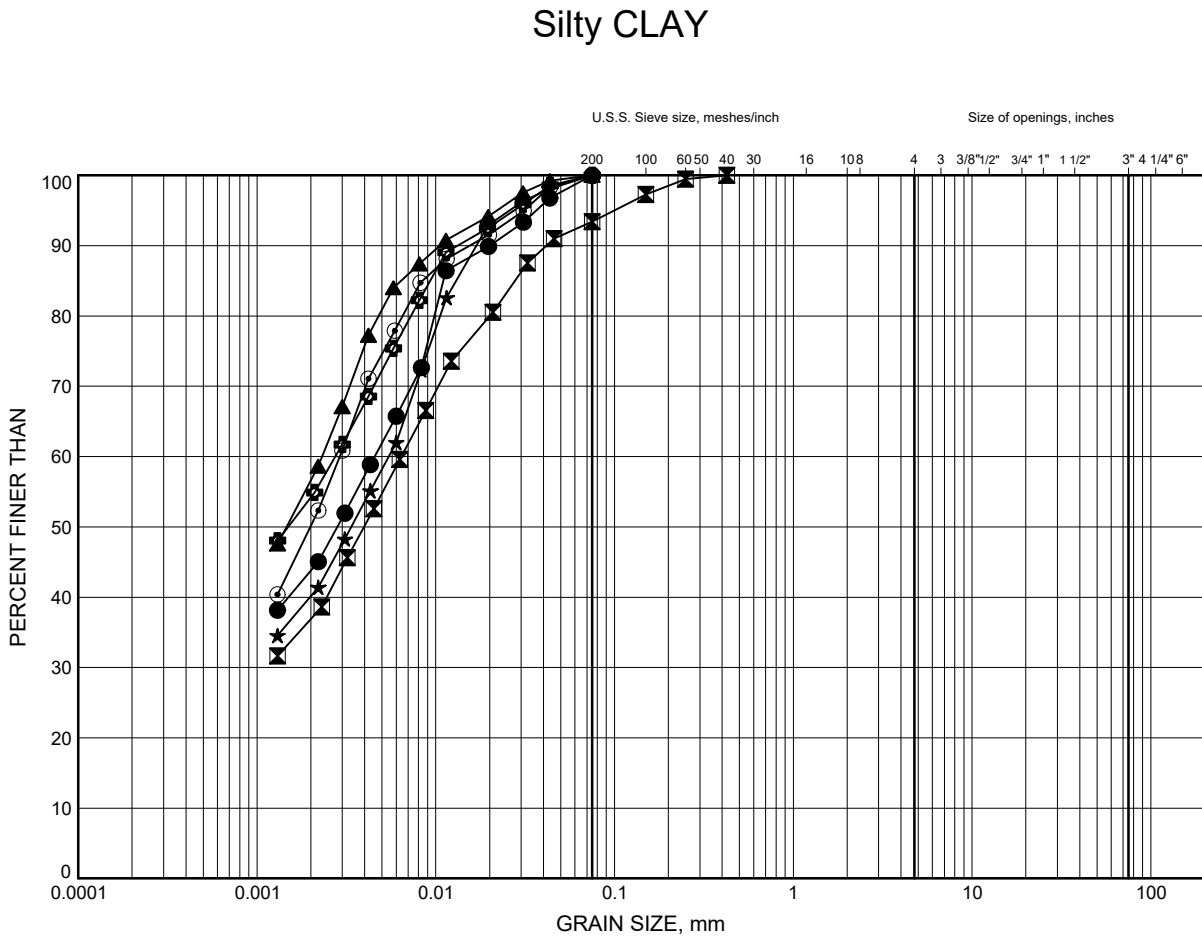
Date March 2016  
W.P. 5120-07-01



Prep'd AN  
Chkd. DJP

# Duchesnay 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)
●	DCB-03	21.64	184.66
⊠	DCB-06	3.17	194.62
▲	DCB-06	4.75	193.05
★	DCB-07	4.88	195.82
⊙	DCB-07	10.97	189.72
⊕	DCB-07	13.94	186.75

Date March 2016  
W.P. 5120-07-01

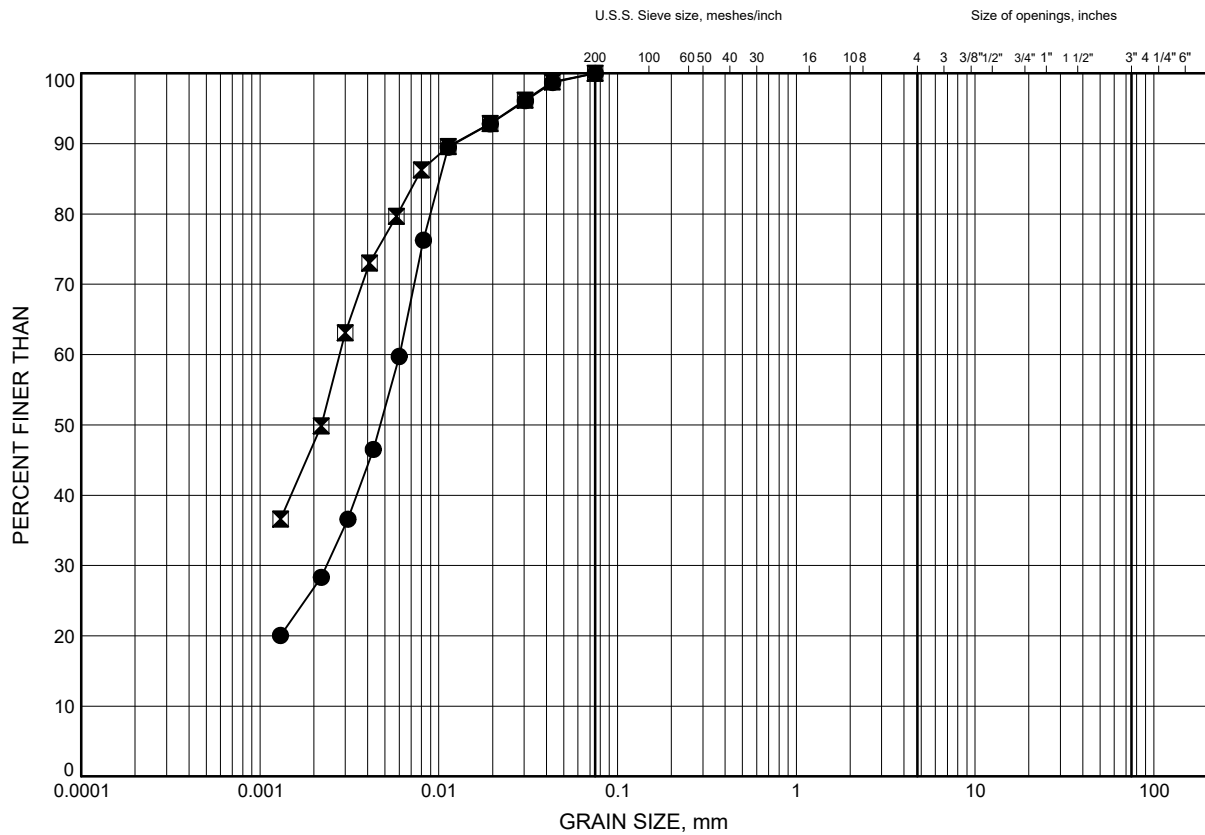


Prep'd AN  
Chkd. DJP

# Duchesnay Creek GRAIN SIZE DISTRIBUTION

FIGURE B7

## Silty CLAY



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCB-08	2.59	195.11
⊠	DCB-08	7.92	189.77

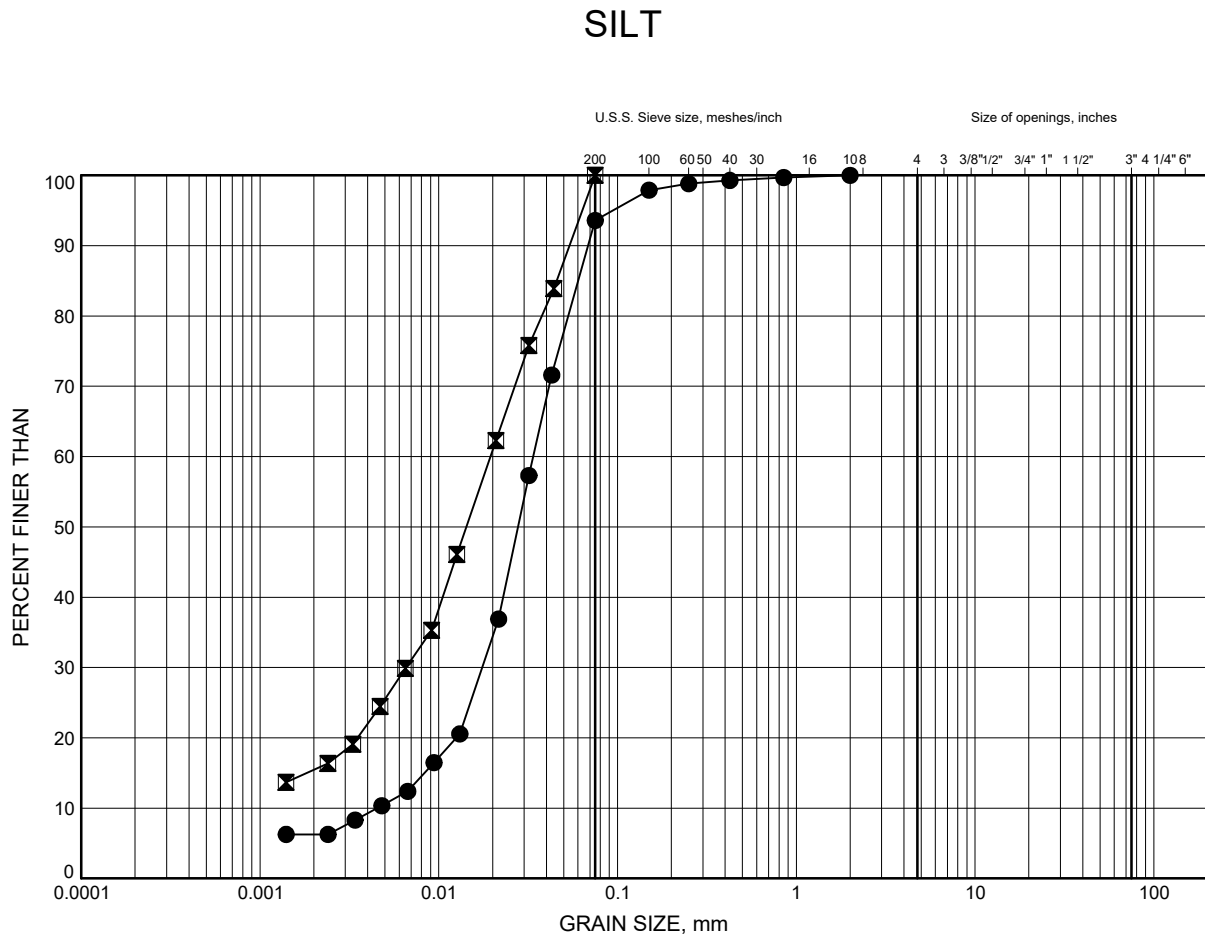
Date March 2016  
W.P. 5120-07-01



Prep'd AN  
Chkd. DJP

# Duchesnay Creek GRAIN SIZE DISTRIBUTION

FIGURE B8



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCB-02	18.59	185.80
⊠	DCB-06	7.92	189.87

Date March 2016  
W.P. 5120-07-01

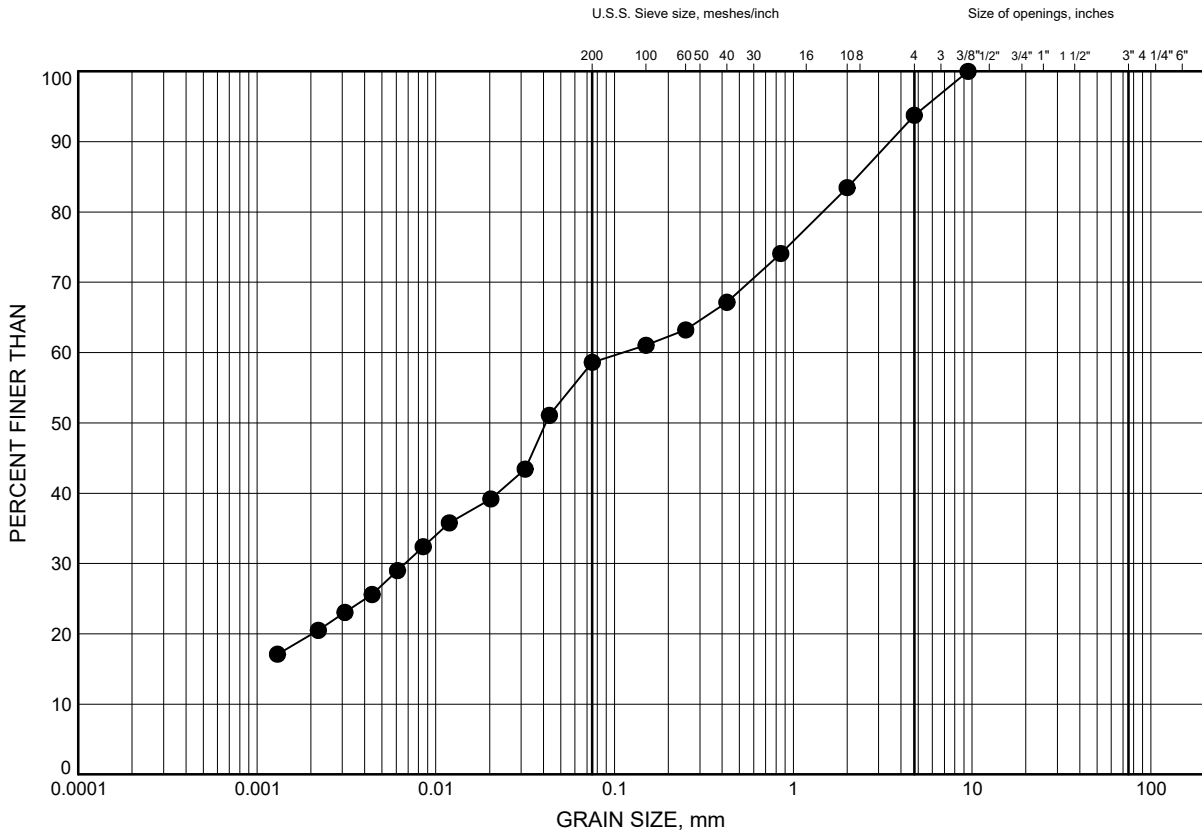


Prep'd AN  
Chkd. DJP

# Duchesnay Creek GRAIN SIZE DISTRIBUTION

FIGURE B9

## Clayey SILT With SAND TILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCB-03	25.91	180.39

Date March 2016  
W.P. 5120-07-01

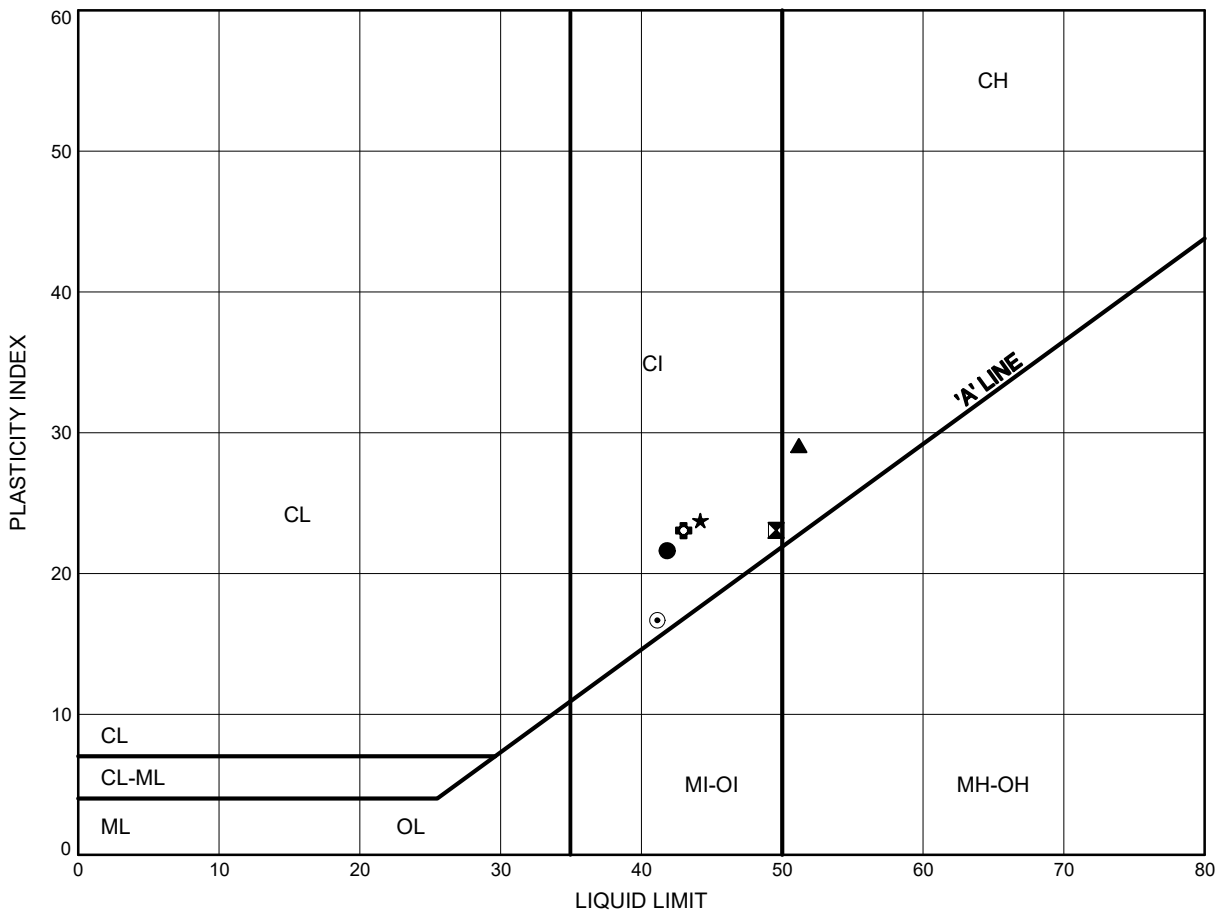


Prep'd AN  
Chkd. DJP

Duchesnay Creek  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B10

Silty CLAY



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCB-01	4.88	199.92
⊠	DCB-02	9.45	194.95
▲	DCB-02	14.02	190.38
★	DCB-03	17.07	189.23
⊙	DCB-03	21.64	184.66
⊕	DCB-06	4.75	193.05

Date March 2016  
 W.P. 5120-07-01



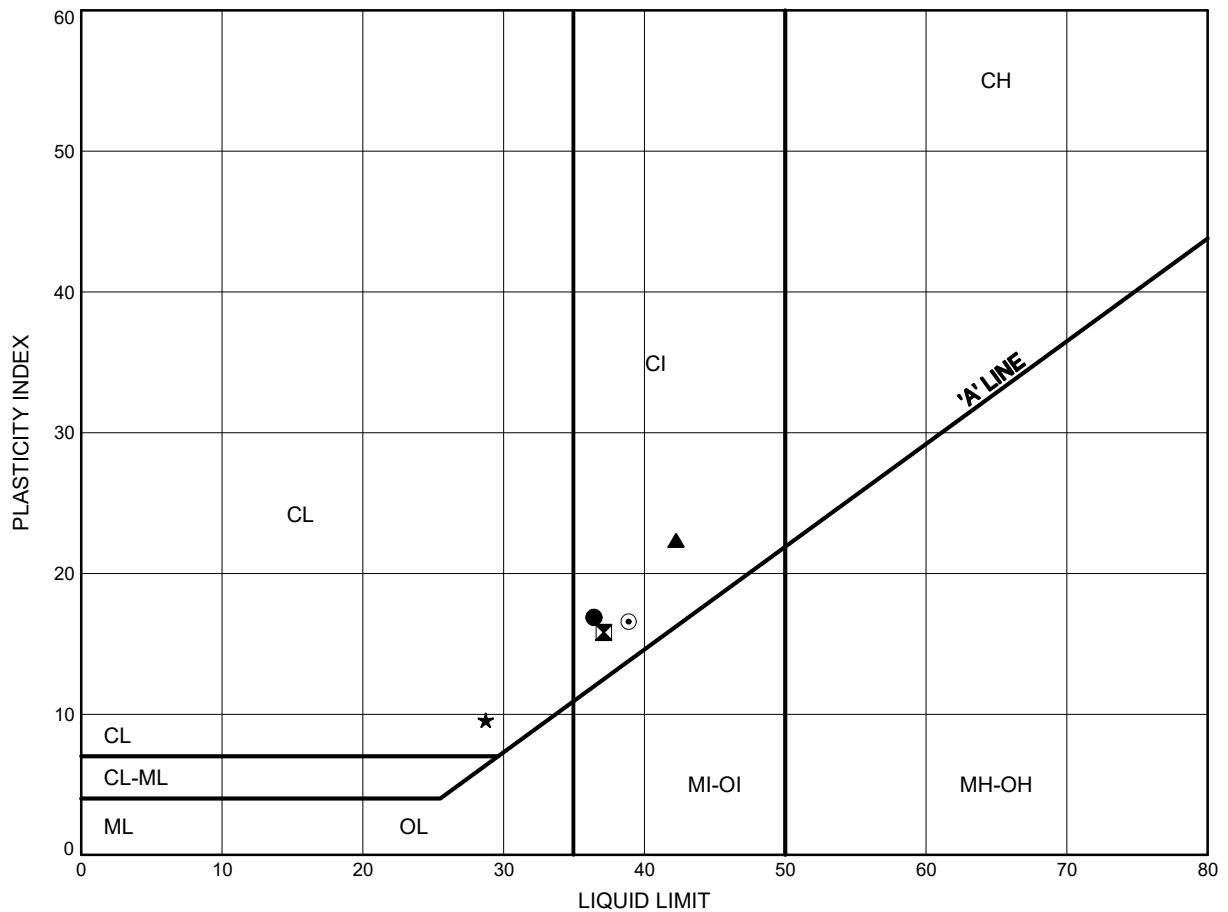
Prep'd AN  
 Chkd. DJP



Duchesnay Creek  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B11

Silty CLAY



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCB-07	4.88	195.82
⊠	DCB-07	10.97	189.72
▲	DCB-07	13.94	186.75
★	DCB-08	2.59	195.11
⊙	DCB-08	7.92	189.77

Date March 2016  
W.P. 5120-07-01



Prep'd AN  
Chkd. DJP

**PHOTOGRAPHS OF BEDROCK CORE SAMPLES  
DUCHESNAY CREEK BRIDGE REPLACEMENT  
HIGHWAY 17B, NORTH BAY, ONTARIO**

**DCB-06**



**RUN #1 (27' – 28'7"); RUN #2 (28'7" – 30'8"); RUN #3 (30'8" – 31'8")  
RUN #4 (31'8" – 34'10")  
RUN #5 (34'10" – 38'4")**

**DCB-07**



**RUN #1 (50' – 52'); RUN #2 (52' – 55'1");  
RUN #3 (55'1" – 56'10");  
RUN #4 (56'10" – 61'6")**



**THURBER** ENGINEERING LTD.

## POINT LOAD TEST SHEET

Job No : 19-3948-5 Client : LEA CONSULTING LTD  
 Date Drilled : 19-Dec-15  
 Project Name : DUCHESNAY CREEK DD Date Tested : 04-Jan-16  
 Core Size : NQ BH No : DCB-06 Tester : ISP

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	8.3	D	19.4	46.6	139.1	196.5	Granite	Very Strong
2	1	8.6	D	18.4	46.7	101.4	186.5	Granite	Very Strong
3	2	9.0	A	10.2	46.6	59.2	71.3	Granite	Strong
4	2	9.3	D	15.8	46.5	80.3	161.4	Granite	Very Strong
5	4	9.9	D	16.4	46.6	75.2	166.5	Granite	Very Strong
6	4	10.3	A	15.5	46.7	66.2	99.0	Granite	Strong
7	4	10.6	D	13.6	46.7	151.2	137.7	Granite	Very Strong
8	5	10.7	D	15.2	46.7	151.2	153.5	Granite	Very Strong
9	5	10.9	D	10.5	46.7	151.2	105.7	Granite	Very Strong
10	5	11.5	D	13.1	46.7	151.2	132.8	Granite	Very Strong
11	5	11.6	A	3.2	46.7	70.1	19.7	Granite	Weak
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									
31									
32									
33									
34									
35									

\* 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.

**THURBER ENGINEERING LTD.****POINT LOAD TEST SHEET**

**Job No :** 19-3948-5 **Client :** LEA CONSULTING LTD  
**Date Drilled :** 21-Dec-15  
**Project Name :** DUCHESNAY CREEK DD **Date Tested :** 05-Jan-16  
**Core Size :** NQ **BH No :** DCB-07 **Tester :** ISP

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	15.4	D	19.8	46.9	101.0	199.5	Granite	Very Strong
2	1	15.8	A	15.3	46.8	70.7	92.9	Granite	Strong
3	2	16.1	D	12.9	46.8	133.6	130.0	Granite	Very Strong
4	3	17.0	D	12.9	46.8	151.2	130.3	Granite	Very Strong
5	4	17.4	A	10.1	46.9	66.4	64.2	Granite	Strong
6	4	17.7	D	10.3	46.9	92.4	103.2	Granite	Very Strong
7	4	18.4	A	10.8	46.8	63.5	71.1	Granite	Strong
8	4	18.7	D	9.0	46.9	99.0	90.4	Granite	Strong
9									
10									
11									
12									
13									
14									
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32									
33									
34									
35									

\* 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 C**

### **Borehole Logs from Previous Investigation**

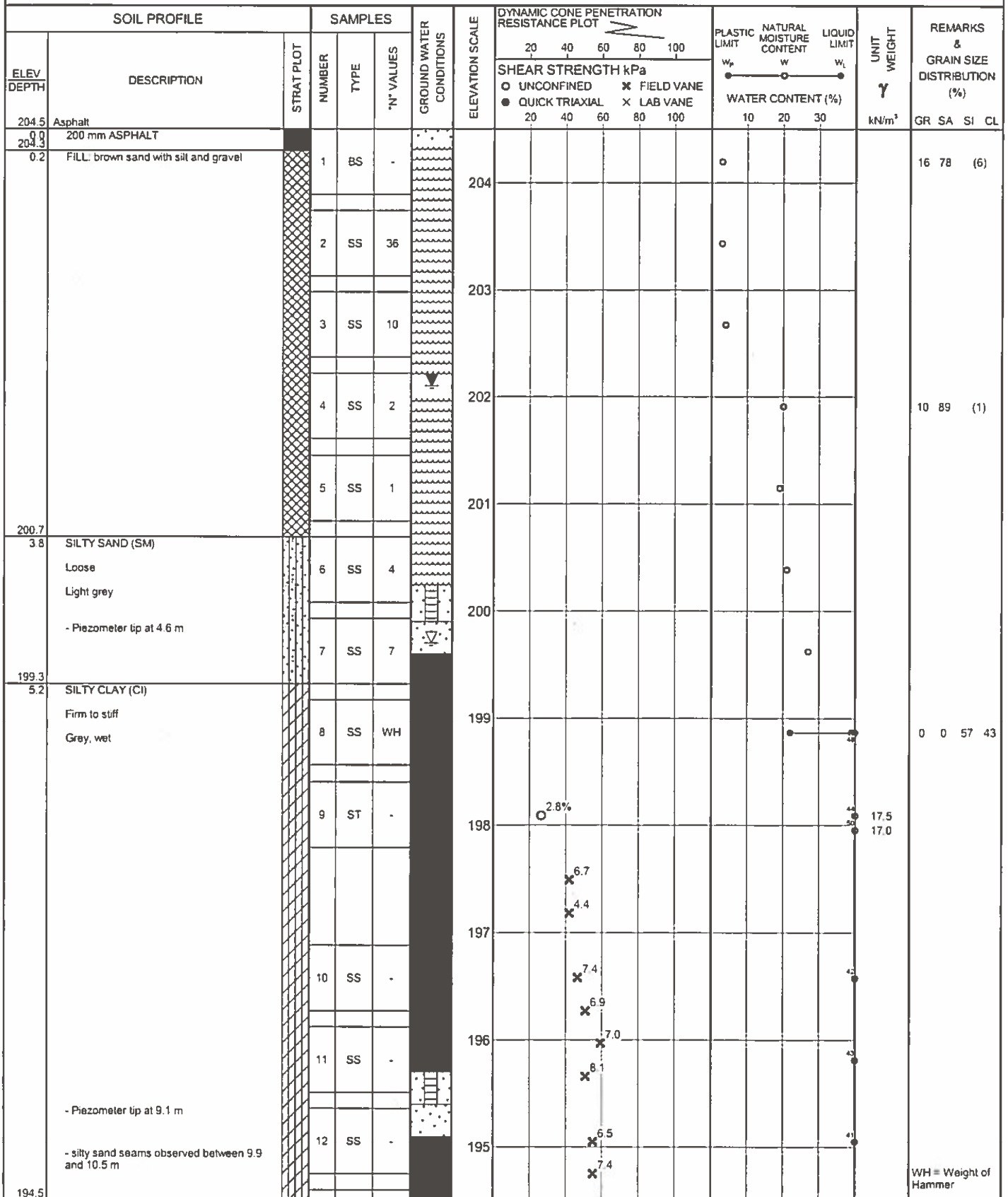


## RECORD OF BOREHOLE No BH13-3

1 OF 3

METRIC

W.P. GWP 5120-07-00 LOCATION Hwy 17B Duchesnay Creek Bridge, North Bay, ON N: 5 132 120 E: 304 519 ORIGINATED BY AB  
DIST HWY 17B BOREHOLE TYPE 8" Augers, Split Spoon Sampler, NQ Rock Core COMPILED BY BB  
DATUM Geodetic DATE 2013 09 30 - 2013 10 01 CHECKED BY CM/SG



STN13-ONTARIO MTO STANTEC 165000835 HWY 17B NORTH BAY.GPJ ONTARIO MOT.GDT 4/23/14

Continued Next Page

× 3 × 3 Numbers refer to Sensitivity  
○ 3% STRAIN AT FAILURE



## RECORD OF BOREHOLE No BH13-3

2 OF 3

METRIC

W.P. GWP 5120-07-00 LOCATION Hwy 17B Duchesnay Creek Bridge, North Bay, ON N: 5 132 120 E: 304 519 ORIGINATED BY AB  
DIST HWY 17B BOREHOLE TYPE 8" Augers, Split Spoon Sampler, NO Rock Core COMPILED BY BB  
DATUM Geodetic DATE 2013 09 30 - 2013 10 01 CHECKED BY CMSG

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 <sub>p</sub>	w	w <sub>L</sub>			
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE						
								20   40   60   80   100	WATER CONTENT (%)						
10.0	(continued) SILTY CLAY (CI)  Firm to very stiff  Gray, wet		13	SS	-									0   2   53   45	
192.3 12.2	SILTY CLAY (CI)  Firm to very stiff  Grayish red, wet		14	ST	-			10.0%					18.4 19.5	s <sub>u</sub> > 106 kPa @ 10.1 m	
			15	SS	-									s <sub>u</sub> > 106 kPa @ 14 m	
			16	SS	-										
188.0 16.5	SILTY SAND (SM)  Compact  Gray		17	SS	11										
186.2 18.3	Silty sand (SM), some gravel TILL  Dense to very dense  Gray		18	SS	38									7   59   (34)	
184.9 19.6	Granite BEDROCK		19	SS	100/ 130mm										
184.5															

Continued Next Page

✕ 3 ✕ 3

Numbers refer to  
Sensitivity

○ 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY GPJ ONTARIO MOT.GDT 4/23/14



Stantec

# RECORD OF BOREHOLE No BH13-3

3 OF 3

METRIC

W.P. GWP 5120-07-00 LOCATION Hwy 17B Duchesnay Creek Bridge, North Bay, ON N: 5 132 120 E: 304 519 ORIGINATED BY AB

DIST HWY 17B BOREHOLE TYPE 8" Augers, Splitspoon Sampler, NQ Rock Core COMPILED BY BB

DATUM Geodetic DATE 2013 09 30 - 2013 10 01 CHECKED BY CM/SG

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 (%)		
							20	40	60	80	100								
20.0	(continued) Granite BEDROCK  - good to excellent quality - gray to pinkish grey - slightly weathered - close to medium joint set spacing  (Refer to Field Bedrock Core Log)		20	NQ	-		184										TCR = 100% ROD = 96%		
			21	NQ	-		183											TCR = 100% ROD = 92%	
			22	NQ	-		182											TCR = 72% ROD = 50%	
181.7 22.8	End of Borehole  Vibrating wire piezometers installed with tips at 4.6 and 9.1 m below ground surface  Inferred water level time of drilling = 4.8 m deep (elevation 199.6 m)  Measured water level in shallower piezometer = 2.4 m deep (elevation 202.1 m)  Measured water level in deeper piezometer = 3.1 m deep (201.4 m)																		

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY GPJ ONTARIO MOT GDT 4/23/14

× 3 × 3

Numbers refer to Sensitivity

○ 3%

STRAIN AT FAILURE





# RECORD OF BOREHOLE No BH13-4

1 OF 4

METRIC

W.P. GWP 5120-07-00 LOCATION Hwy 17B Duchesnay Creek Bridge, North Bay, ON N: 5 132 061 E: 304 580 ORIGINATED BY AB  
 DIST HWY 17B BOREHOLE TYPE 8" Augers, Split Spoon Sampler, NQ Rock Core COMPILED BY BB  
 DATUM Geodetic DATE 2013 09 25 - 2013 09 26 CHECKED BY CM/SG

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								
								○ UNCONFINED	×	FIELD VANE						
								● QUICK TRIAXIAL	×	LAB VANE						
206.4	Asphalt						20	40	60	80	100					
0.0	250 mm ASPHALT															
206.2																
0.3	FILL: brown sand with silt and gravel		1	BS	-		206								6	81 (13)
			2	SS	5											
205.0							205									
1.4	FILL: Inferred rockfill, cobbles and boulders  - coring carried out to advance through rockfill (sample #3)															
			3	NQ	-		204									
							203									
							202									
201.8																
4.6	Poorly graded SAND (SP) with gravel, trace silt  Very loose  Brown, moist		4	SS	4		201									
			5	SS	1											
200.6																
5.8	SILTY SAND (SM), trace gravel  Loose  Grey, wet						200									
			6	SS	8											
			7	SS	8		199								0	66 (34)
			8	SS	4											
198.0							198									
8.4	SANDY SILT (ML)  Loose  Grey, wet		9	SS	3											
			10	SS	6		197									
196.5																

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY.GPJ ONTARIO MOT.GDT 4/23/14

Continued Next Page

× 3, × 3 Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

W.P.	GWP 5120-07-00	LOCATION	Hwy 17B Duchesnay Creek Bridge, North Bay, ON	N: 5 132 061 E: 304 580	ORIGINATED BY	AB
DIST	HWY 17B	BOREHOLE TYPE	8" Augers, Spitspoon Sampler, NQ Rock Core		COMPILED BY	BB
DATUM	Geodetic	DATE	2013 09 25 - 2013 09 26		CHECKED BY	CM/SG

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/m³	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						
9.9	SILTY CLAY (CI), trace sand  Firm  Grey, wet (continued)		11	SS	9										
			12	ST	-										
192.7 13.7	CLAYEY SILT (CL) to SILTY CLAY (CI)  Firm to very stiff  Grey, wet		13	SS	1										
			14	SS	WH										
			15	SS	1										
			16	ST	-										

Continued Next Page

$\times^3, \times^3$  Numbers refer to Sensitivity
 
 $\bigcirc^{3\%}$  STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY.GPJ ONTARIO MOT.GDT 4/23/14



## RECORD OF BOREHOLE No BH13-4

3 OF 4

METRIC

W.P. GWP 5120-07-00 LOCATION Hwy 17B Duchesnay Creek Bridge, North Bay, ON N: 5 132 061 E: 304 580 ORIGINATED BY AB  
DIST HWY 17B BOREHOLE TYPE 8" Augers, Spitspoon Sampler, NQ Rock Core COMPILED BY BB  
DATUM Geodetic DATE 2013 09 25 - 2013 09 26 CHECKED BY CM/SG

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		WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE	w <sub>p</sub>	w	w <sub>L</sub>		
								20 40 60 80 100						
	CLAYEY SILT (CL) to SILTY CLAY (CI)		17	SS	3		2.4						GR SA SI CL	
	Firm to very stiff												0 0 54 46	
	Grey, wet (continued)													
				18	SS	-							s <sub>u</sub> > 106 kPa @ 21.6 m	
													s <sub>u</sub> > 106 kPa @ 21.9 m	
							185							
							184							
			19	SS	-								s <sub>u</sub> > 106 kPa @ 23.1 m	
							183						s <sub>u</sub> > 106 kPa @ 23.4 m	
							182							
			20	SS	-								s <sub>u</sub> > 106 kPa @ 24.7 m	
181.1							181							
25.3	Sandy clayey silt (CL), some gravel TILL													
	Very stiff													
	Grey, wet		21	SS	-								1 48 31 20 s <sub>u</sub> > 106 kPa @ 26.2 m	
179.7							180							
26.7	Granite BEDROCK												TCR = 51% RQD = 21%	
	- poor to good quality			22	NQ	-								
	- grey to pinkish grey													
	- slightly weathered													
	- close joint set spacing						179							
	(Refer to Field Bedrock Core Log)													
							178						TCR = 93% RQD = 78%	
			23	NQ	-									
							177						TCR = 77% RQD = 57%	
			24	NQ	-									
176.7														
29.7	End of Borehole													

Continued Next Page

x 3 x 3

Numbers refer to  
Sensitivity

O 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY, GPJ ONTARIO MTO GDT 4/23/14



# RECORD OF BOREHOLE No BH13-4

4 OF 4

METRIC

W.P. GWP 5120-07-00 LOCATION Hwy 17B Duchesnay Creek Bridge, North Bay, ON N: 5 132 061 E: 304 580 ORIGINATED BY AB  
 DIST HWY 17B BOREHOLE TYPE 8" Augers, Splitspoon Sampler, NQ Rock Core COMPILED BY BB  
 DATUM Geodetic DATE 2013 09 25 - 2013 09 26 CHECKED BY CW/SG

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 $\gamma$ 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 (%)					
	Vibrating wire piezometer installed with its tip at 15.2 m below ground surface Inferred water level at time of drilling = 6.8 m (elevation 199.6 m) Measured water level = 6.4 m (elevation 200.0 m)													

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY GPJ ONTARIO MOT GDT 4/23/14

$\times^3 \cdot \times^3$

Numbers refer to Sensitivity

$\bigcirc^{3\%}$

STRAIN AT FAILURE



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



**Looking West towards Highway 17**



**South Elevation – Looking West from East Abutment**





**South Elevation – Looking East from West Abutment**



**North Elevation – Looking East from West Abutment**



## **Appendix E**

### **Foundation Comparison**





### COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil or Engineered Fill	H-Piles Driven to Bedrock	Drilled-in Pipe Pile	Caissons/Drilled Shafts
<b>Advantages:</b> i. Ease of construction. ii. Lower cost than deep foundations.	<b>Advantages:</b> i. Piles will develop high geotechnical resistance on bedrock. ii. Installation of piles could continue in freezing weather. iii. Allows integral abutment design. iv. Requires less excavation than footings.	<b>Advantages:</b> i. High axial resistance is available for pipe pile socketed in bedrock. ii. Higher lateral resistance is available due to larger pile diameter and in the rock socket. iii. Below-grade installation of drilled-in pipe piles could continue in freezing weather. iv. Liner is not required to support excavation sidewalls. v. Cleaning and inspection of the socket base is not required.	<b>Advantages:</b> i. High resistance is available for caissons founded on bedrock. ii. Below-grade construction of caissons could continue in freezing weather. iii. Minimum requirements for excavation and dewatering.
<b>Disadvantages:</b> i. Low geotechnical resistance is available in native soils at abutments. ii. Potential for consolidation settlement in silty clay. iii. Dewatering may be required, depending on depth of excavation.	<b>Disadvantages:</b> i. Higher unit cost than footings. ii. Possibility that cobbles and/or boulders may be encountered in the fill and native deposits. iii. Care must be taken when setting piles on sloping bedrock.	<b>Disadvantages:</b> i. Specialized installation. ii. Piles must be socketed into very strong bedrock. iii. Difficulty in grouting the annular space between the pile and the socket.	<b>Disadvantages:</b> i. Temporary liners will be required to install caissons in cohesionless soils below the creek and groundwater levels. ii. Difficulty in sealing liners at bedrock surface. iii. Possibility of cobbles and boulders being encountered during augering and liner installation. iv. Difficulty in cleaning and inspecting bases.
<b>NOT RECOMMENDED</b>	<b>RECOMMENDED</b>	<b>RECOMMENDED</b>	<b>NOT RECOMMENDED</b>



## **Appendix F**

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

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

OPSS.PROV 539  
OPSS.PROV 804  
OPSS 902  
OPSS.PROV 903

2) Suggested text for NSSP on “Construction of Driven H-piles”

Installation of H-piles shall be in accordance with OPSS 903 and the following.

Cobbles, boulders and rock fill are present within the existing embankment fill on site. The cobbles, boulders and rock fill will interfere with pile installation and some piles may meet refusal on boulders or rock fill. The Contractor must be prepared to remove, predrill or otherwise penetrate these obstructions to advance the piles to bedrock while meeting the specified deflection tolerances. The predrilling may involve setting a casing through the rock fill through which piles may be driven.

Piles driven to refusal at the abutments and piers may encounter a sloping bedrock surface. Care must be taken during driving of the piles to seat the pile on bedrock and minimize the potential for sliding of the pile tip along the sloping bedrock surface. All H-piles must be provided with the Titus H bearing pile point, rock injector model, or approved equivalent.

3) Suggested text for NSSP on “Construction of Drilled-in Pipe Piles”

Installation of drilled-in pipe piles shall be in accordance with OPSS 903 and the following.

Drilled-in pipe pile installation at this site will require excavation through granular embankment fill with cobbles, boulders and rock fill, as well as cohesionless soils below the groundwater table. The piles must also be advanced into the underlying bedrock for construction of sockets. The Contractor is advised of the following:

- The cohesionless soil above the bedrock is susceptible to disturbance under conditions of unbalanced hydrostatic head, and measures must be taken to maintain base and sidewall stability during installation and prevent collapse/washing of cohesionless soils into the rock sockets. Selection of the drilling methods and equipment to this effect is the responsibility of the Contractor.
- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles, boulders and rock fill in the embankment fill and native soils.
- The bedrock consists of very strong granitic rock. The strength and hardness of the bedrock must be taken into account when selecting equipment to advance the pile into rock. Equipment supplied to advance the pile into rock must be capable of penetrating the bedrock to create a clean socket without disturbing or fracturing the bedrock adjacent to the pile.

- The rock socket must be formed entirely within the sound bedrock. Any length of pile above the bedrock surface will not be considered part of the specified length of rock embedment.
- The annular space between the rock socket wall and pile shaft shall be filled with 30 MPa concrete or grout to the top of the bedrock surface. The verticality and alignment of the pile shall be maintained during concreting.
- During and subsequent to installation, the pipe pile may be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

4) Suggested text for “Use of Heavy Construction Equipment”

The use of heavy construction equipment and in particular heavy lifting cranes may be required during removal of the existing and erection of the new bridge. The impact of the heavy equipment loads on the underlying soils, river 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 bridge abutments and existing foundations;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment, especially at the west approach; 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.



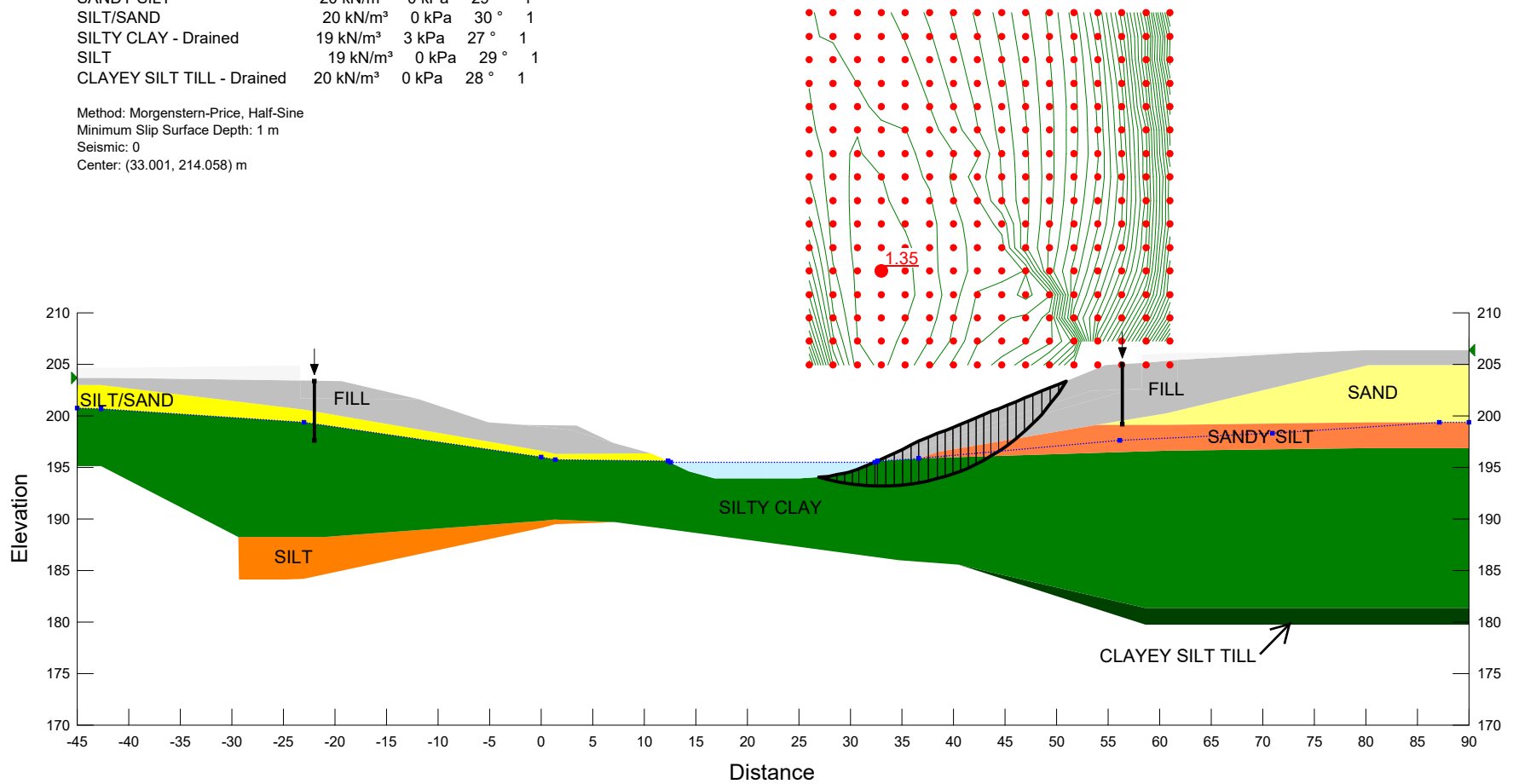
**Appendix G**  
**Results of Global Stability Analyses**

# **DUCHESNAY CREEK EAST ABUTMENT (EXISTING)**

**FIGURE 1**

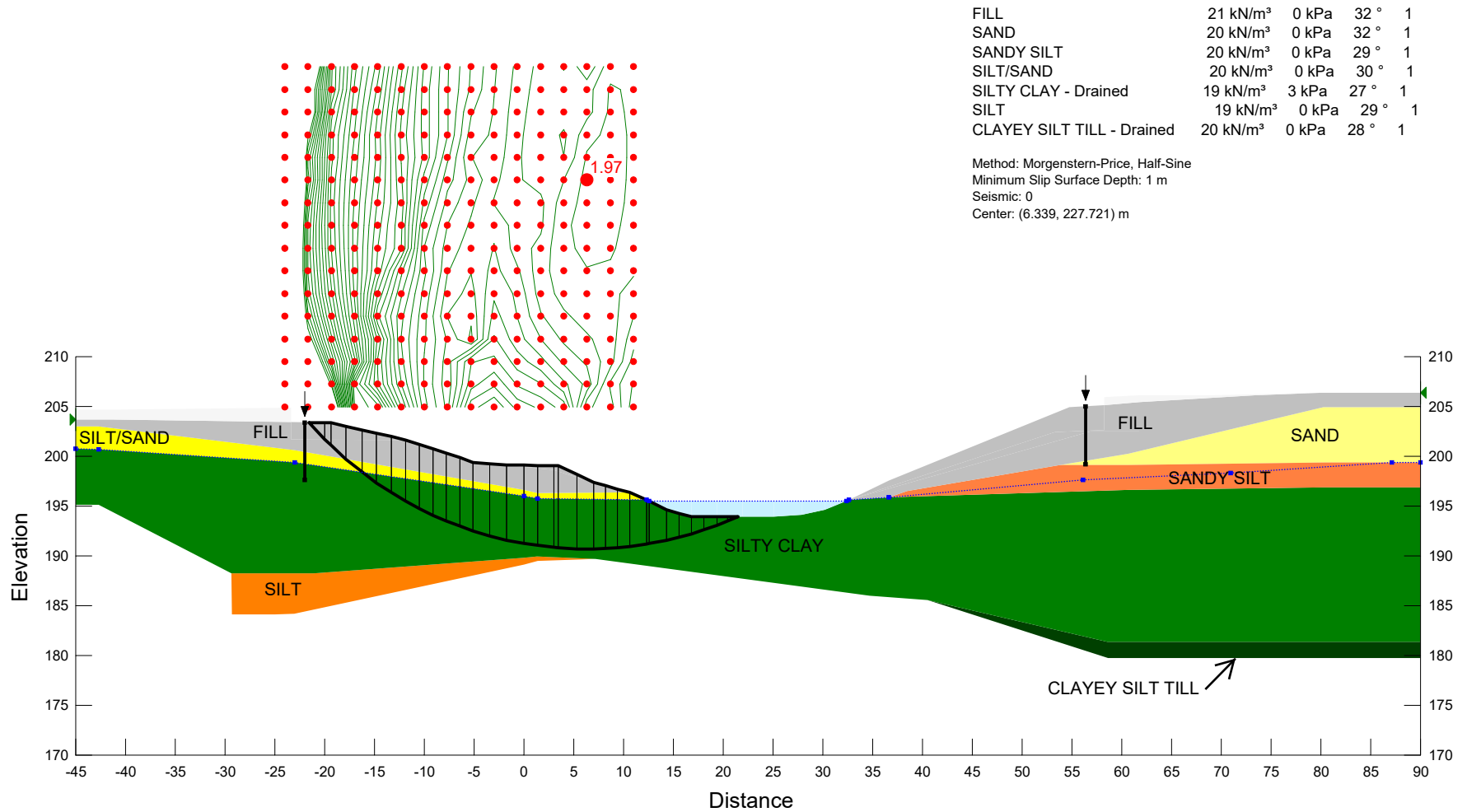
FILL	21 kN/m <sup>3</sup>	0 kPa	32 °	1
SAND	20 kN/m <sup>3</sup>	0 kPa	32 °	1
SANDY SILT	20 kN/m <sup>3</sup>	0 kPa	29 °	1
SILT/SAND	20 kN/m <sup>3</sup>	0 kPa	30 °	1
SILTY CLAY - Drained	19 kN/m <sup>3</sup>	3 kPa	27 °	1
SILT	19 kN/m <sup>3</sup>	0 kPa	29 °	1
CLAYEY SILT TILL - Drained	20 kN/m <sup>3</sup>	0 kPa	28 °	1

Method: Morgenstern-Price, Half-Sine  
Minimum Slip Surface Depth: 1 m  
Seismic: 0  
Center: (33.001, 214.058) m



# **DUCHESNAY CREEK WEST ABUTMENT (EXISTING)**

**FIGURE 2**

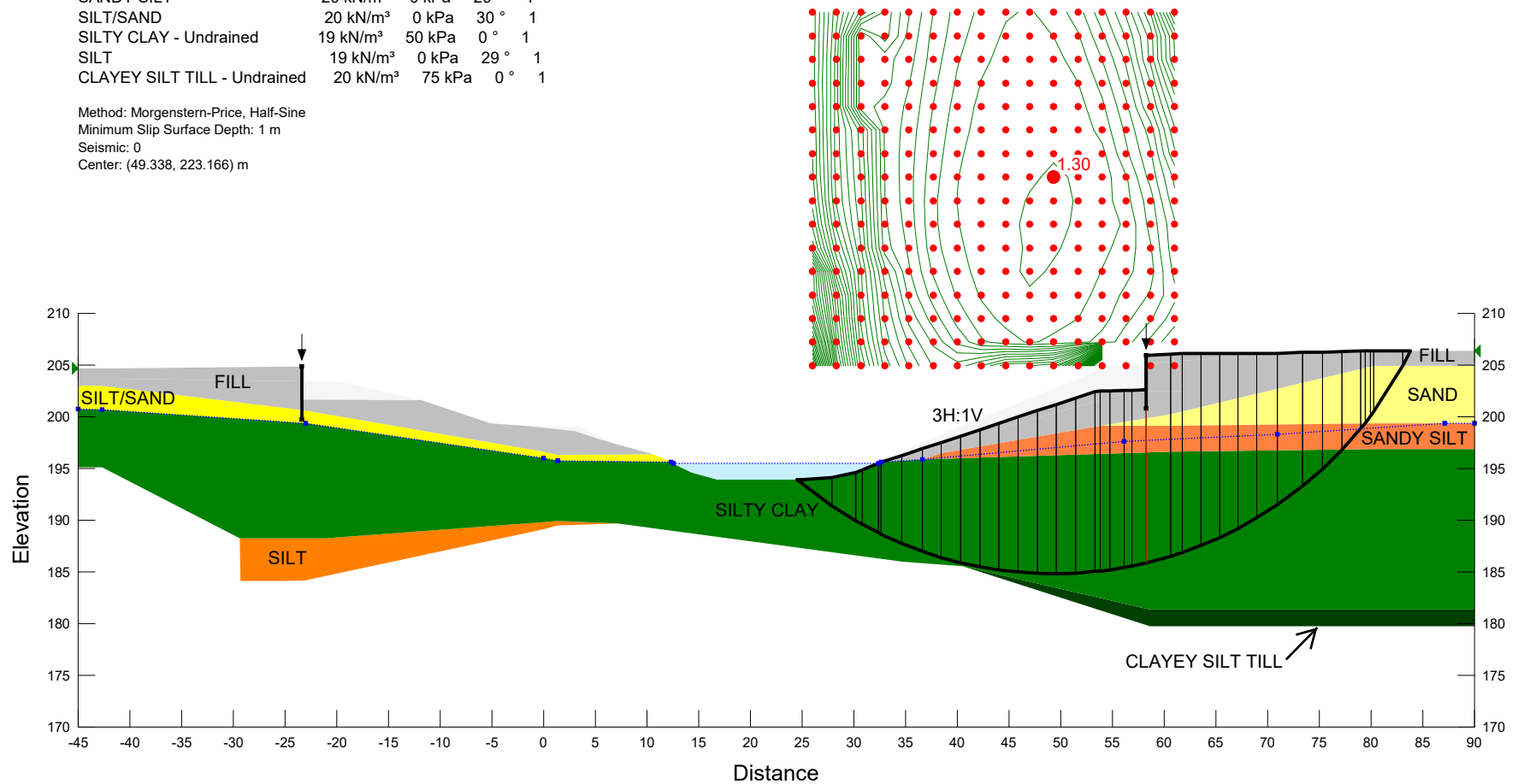


# **DUCHESNAY CREEK EAST ABUTMENT (UNDRAINED)**

**FIGURE 3**

FILL	21 kN/m <sup>3</sup>	0 kPa	32 °	1
SAND	20 kN/m <sup>3</sup>	0 kPa	32 °	1
SANDY SILT	20 kN/m <sup>3</sup>	0 kPa	29 °	1
SILT/SAND	20 kN/m <sup>3</sup>	0 kPa	30 °	1
SILTY CLAY - Undrained	19 kN/m <sup>3</sup>	50 kPa	0 °	1
SILT	19 kN/m <sup>3</sup>	0 kPa	29 °	1
CLAYEY SILT TILL - Undrained	20 kN/m <sup>3</sup>	75 kPa	0 °	1

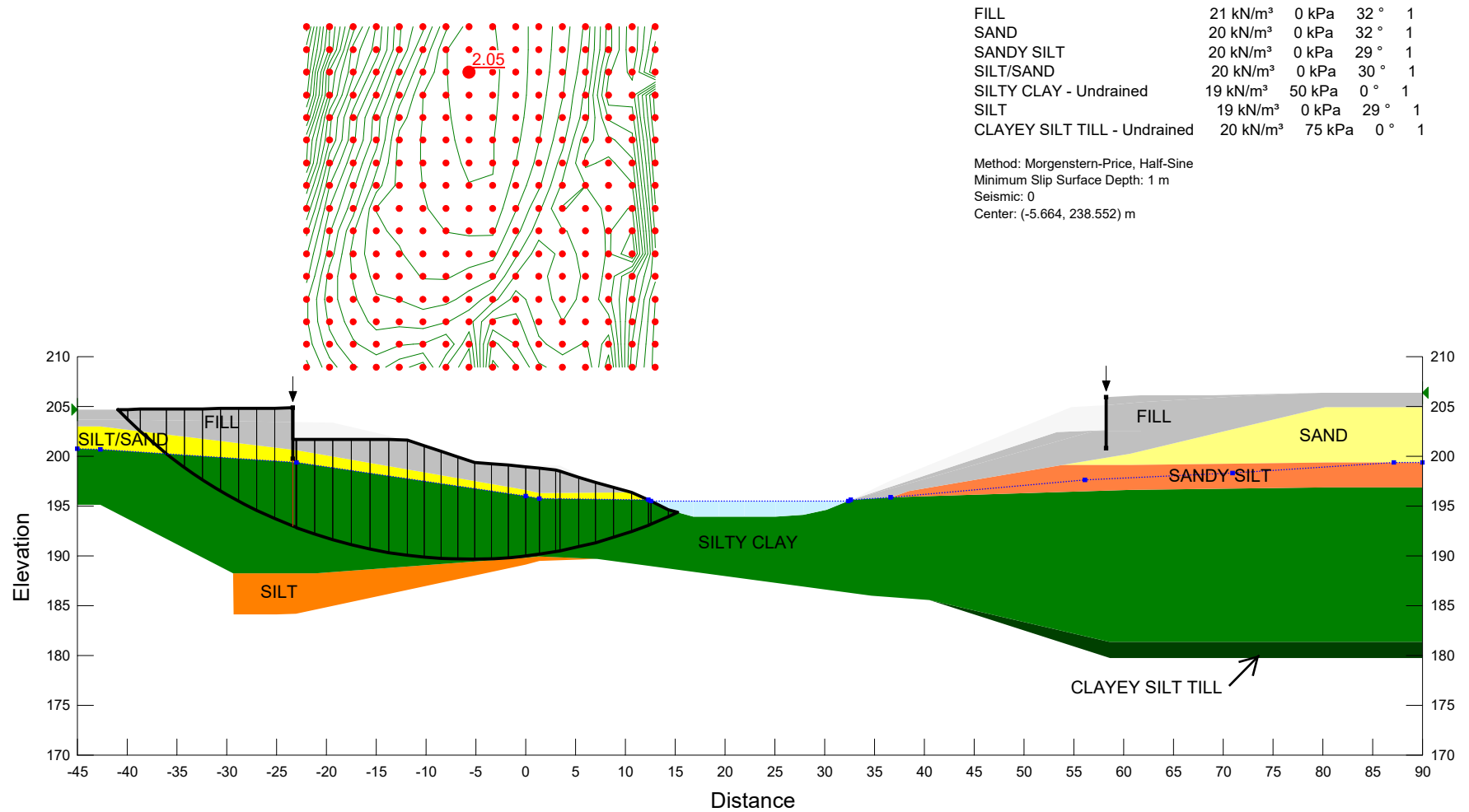
Method: Morgenstern-Price, Half-Sine  
Minimum Slip Surface Depth: 1 m  
Seismic: 0  
Center: (49.338, 223.166) m





# **DUCHESNAY CREEK WEST ABUTMENT (UNDRAINED)**

**FIGURE 4**

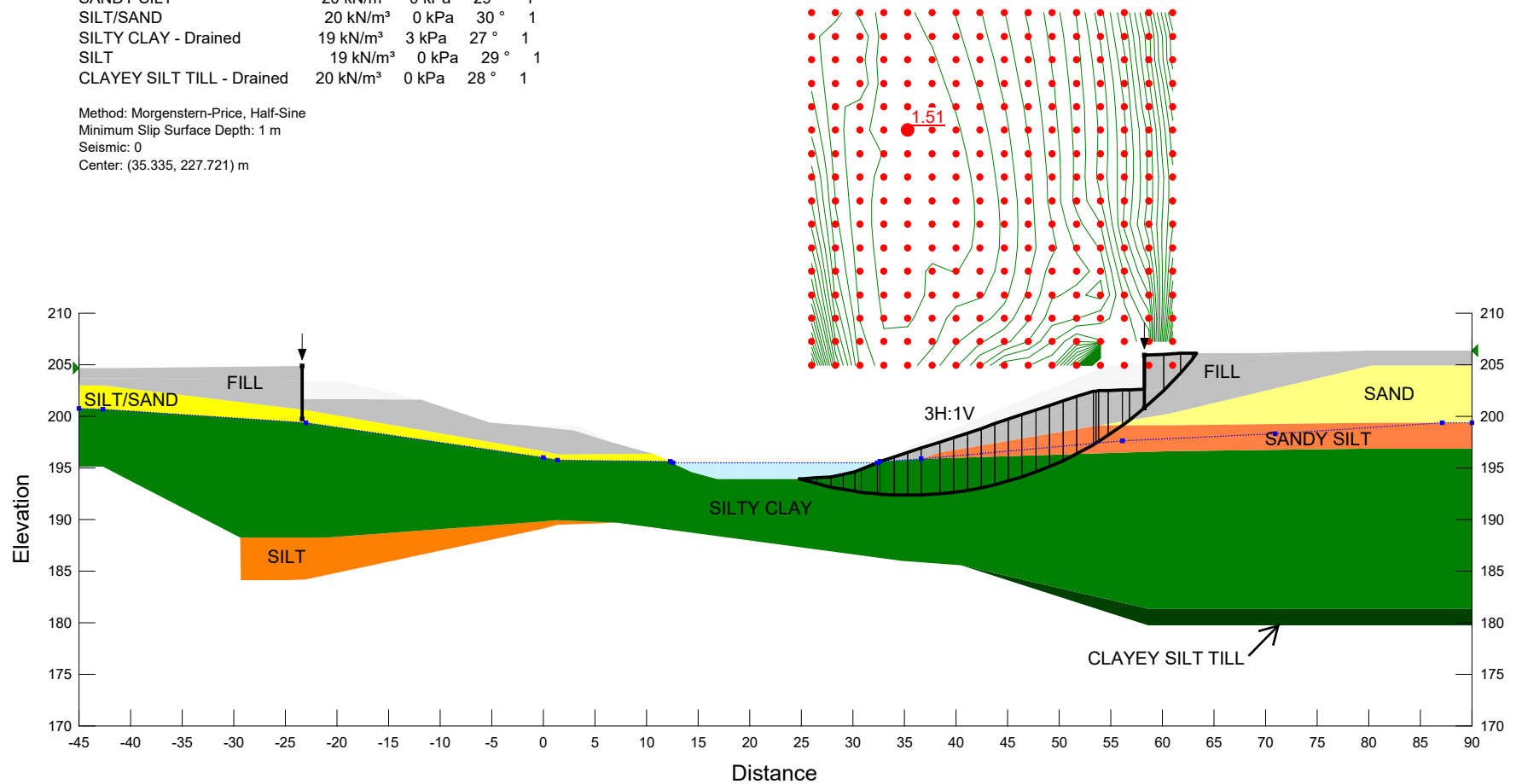


# **DUCHESNAY CREEK EAST ABUTMENT (DRAINED)**

**FIGURE 5**

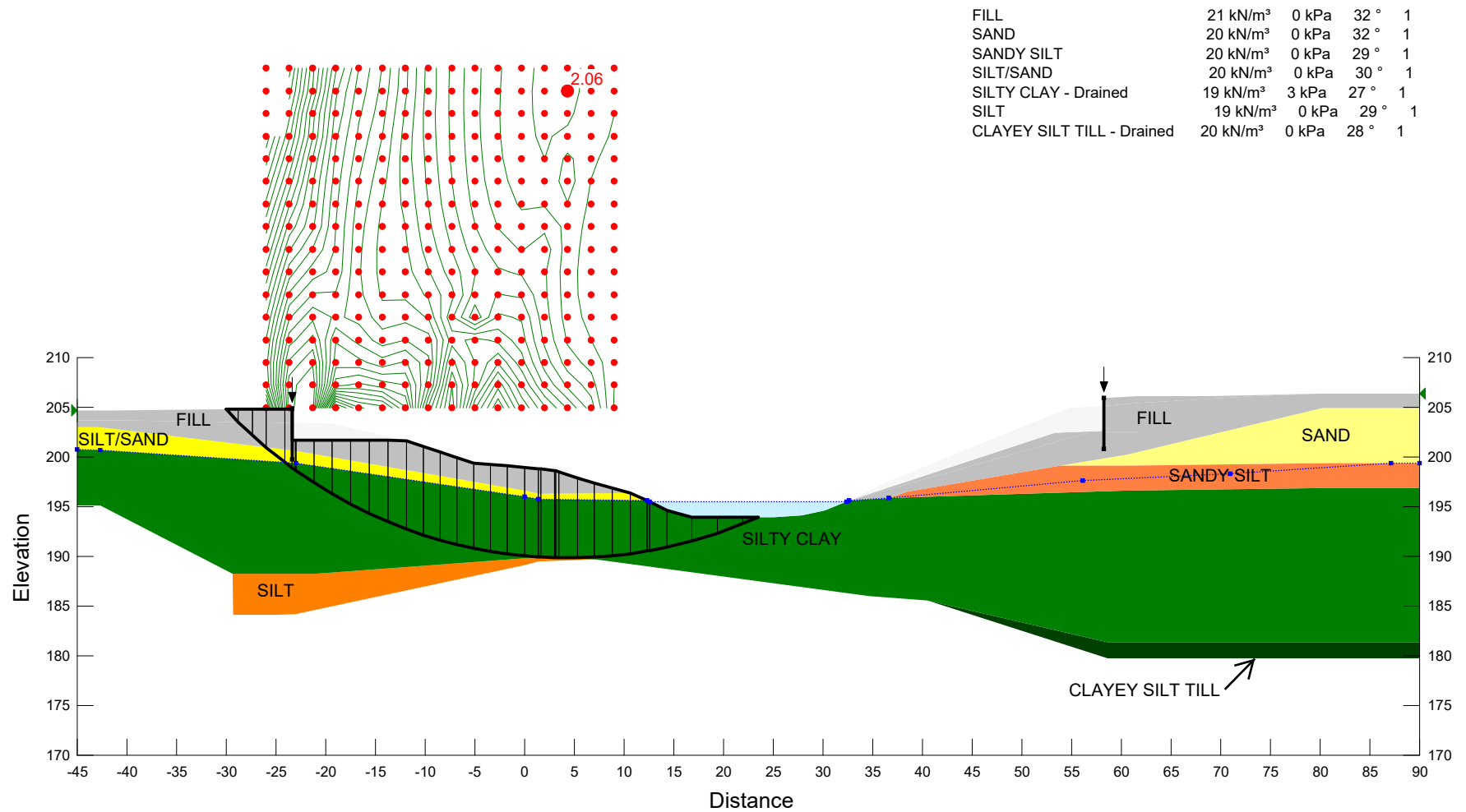
FILL	21 kN/m <sup>3</sup>	0 kPa	32 °	1
SAND	20 kN/m <sup>3</sup>	0 kPa	32 °	1
SANDY SILT	20 kN/m <sup>3</sup>	0 kPa	29 °	1
SILT/SAND	20 kN/m <sup>3</sup>	0 kPa	30 °	1
SILTY CLAY - Drained	19 kN/m <sup>3</sup>	3 kPa	27 °	1
SILT	19 kN/m <sup>3</sup>	0 kPa	29 °	1
CLAYEY SILT TILL - Drained	20 kN/m <sup>3</sup>	0 kPa	28 °	1

Method: Morgenstern-Price, Half-Sine  
Minimum Slip Surface Depth: 1 m  
Seismic: 0  
Center: (35.335, 227.721) m



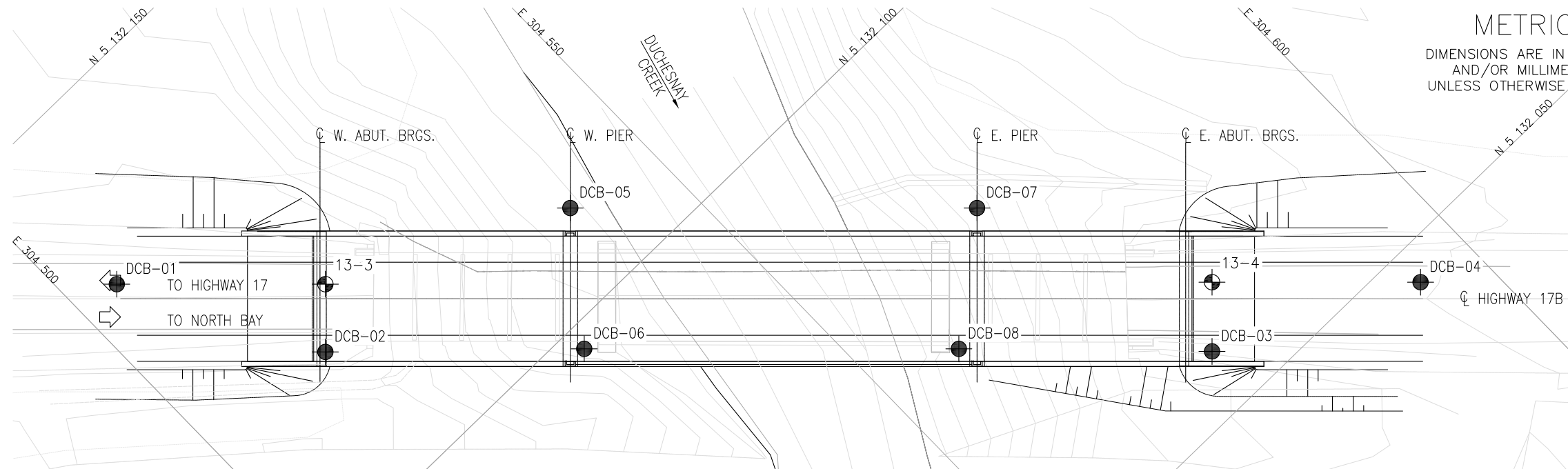
# **DUCHESNAY CREEK WEST ABUTMENT (DRAINED)**

**FIGURE 6**





**Appendix H**  
**Borehole Locations and Soil Strata Drawings**



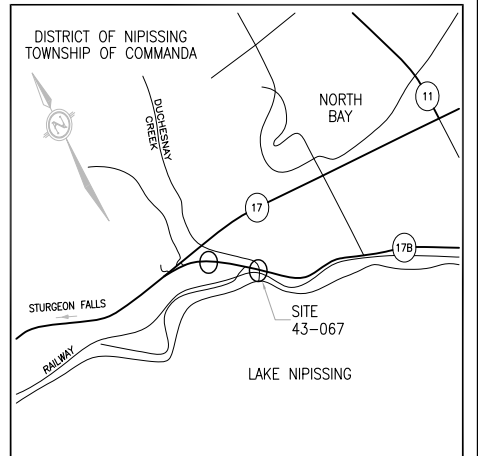
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 5120-07-00



HIGHWAY 17B  
DUCHESNAY CREEK  
BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



### KEYPLAN

### LEGEND

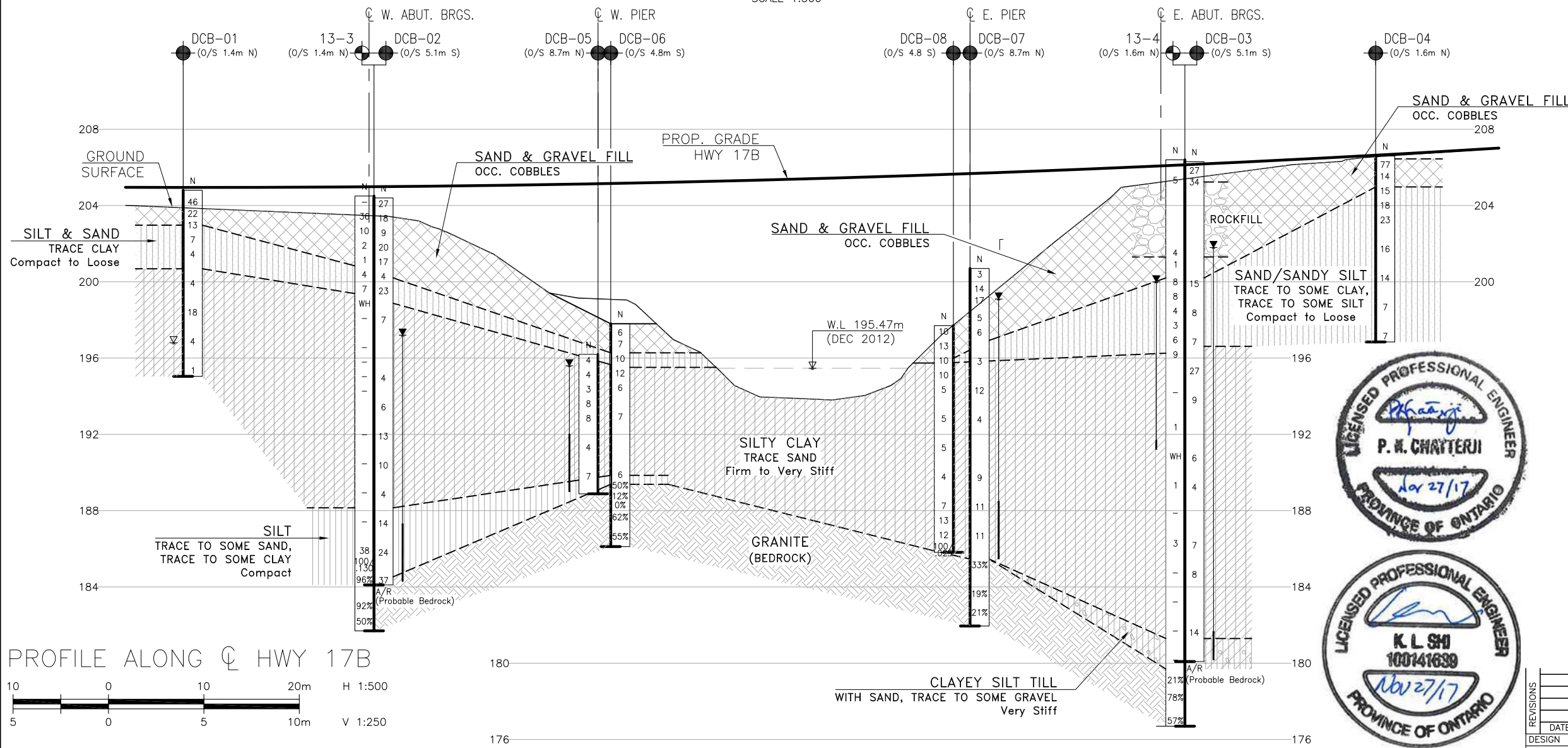
●	Borehole (By Thurber)
⊙	Borehole (By Stantec)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
⬇	Head Artesian Water
⬆	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
13-3	204.5	5 132 119.5	304 518.7
13-4	206.4	5 132 060.5	304 579.9
DCB-01	204.8	5 132 133.4	304 504.3
DCB-02	204.4	5 132 114.8	304 514.2
DCB-03	206.3	5 132 055.7	304 575.3
DCB-04	206.6	5 132 046.6	304 594.3
DCB-05	196.2	5 132 108.4	304 540.7
DCB-06	197.8	5 132 097.8	304 532.2
DCB-07	200.7	5 132 081.3	304 568.7
DCB-08	197.7	5 132 072.8	304 558.0

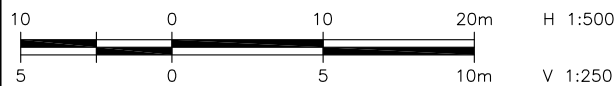
### NOTES

- 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.

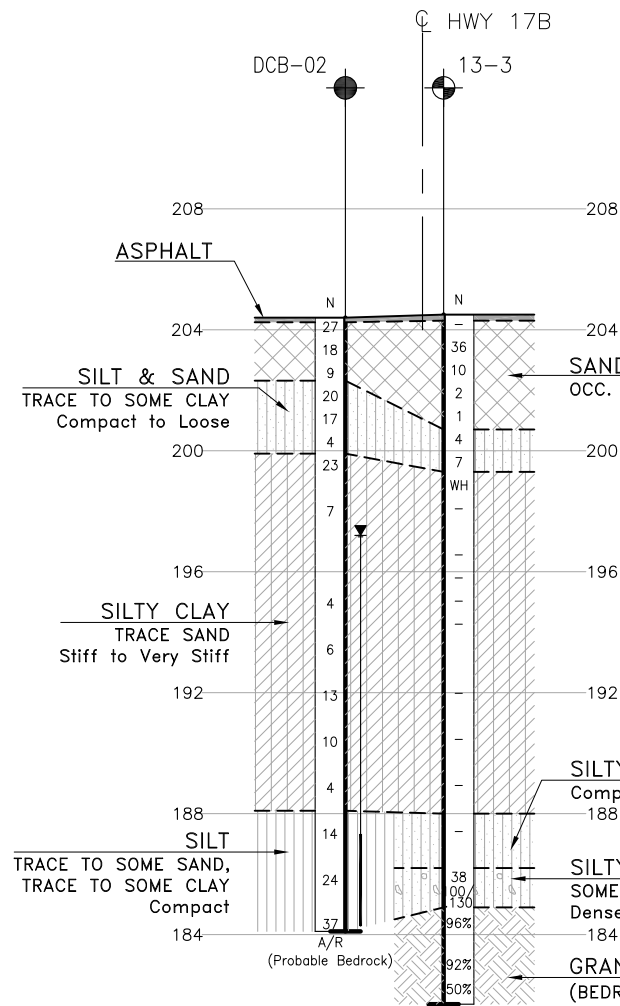
GEOCRES No. 31L-197



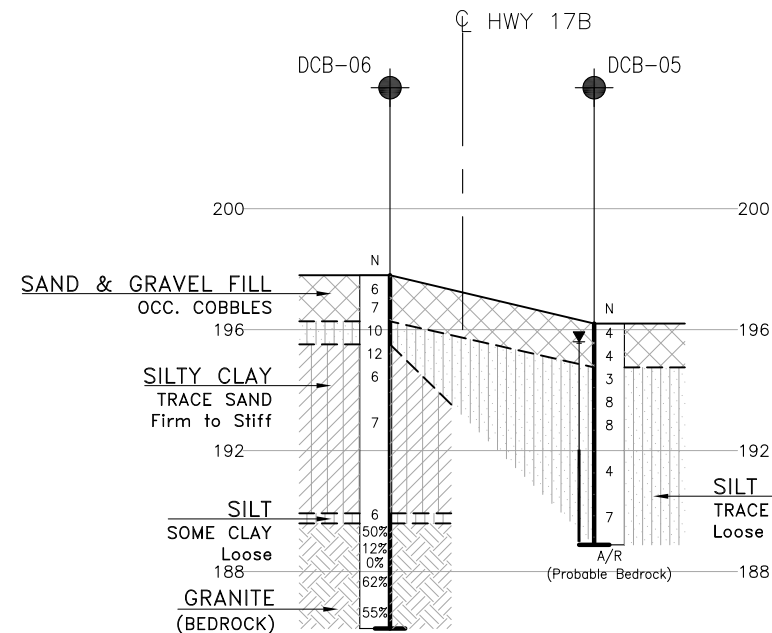
### PROFILE ALONG CL HWY 17B



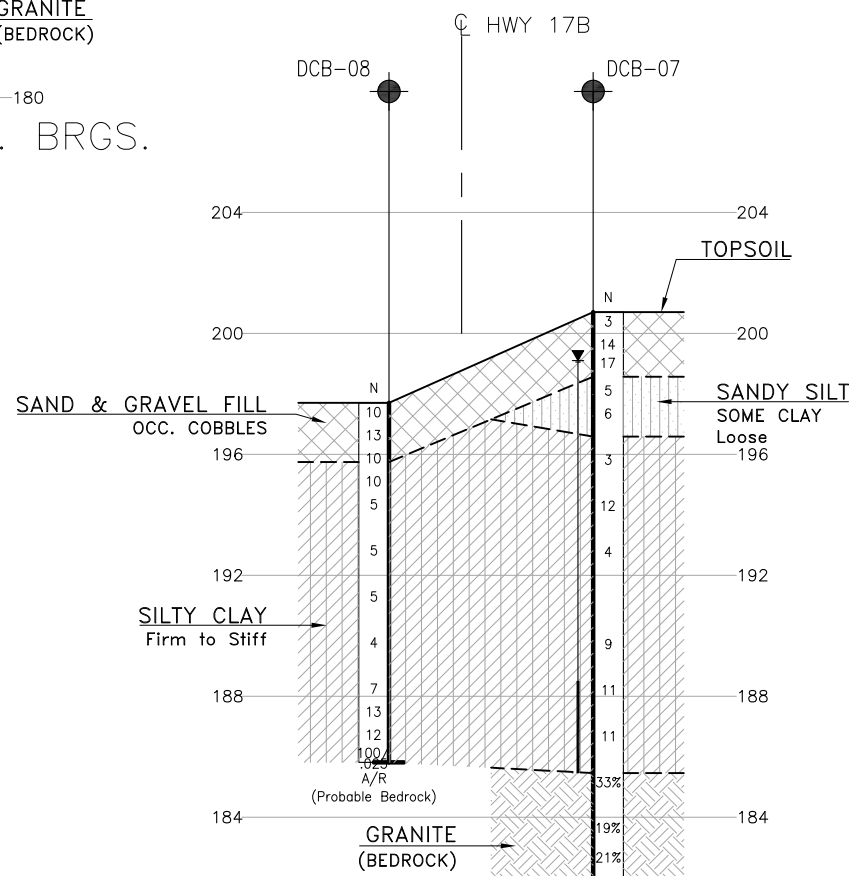
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DESIGN	DJP	CHK AEG	CODE
DRAWN	AN	CHK DJP	SITE
LOAD	DATE	NOV 2017	
STRUCT	DWG	1	



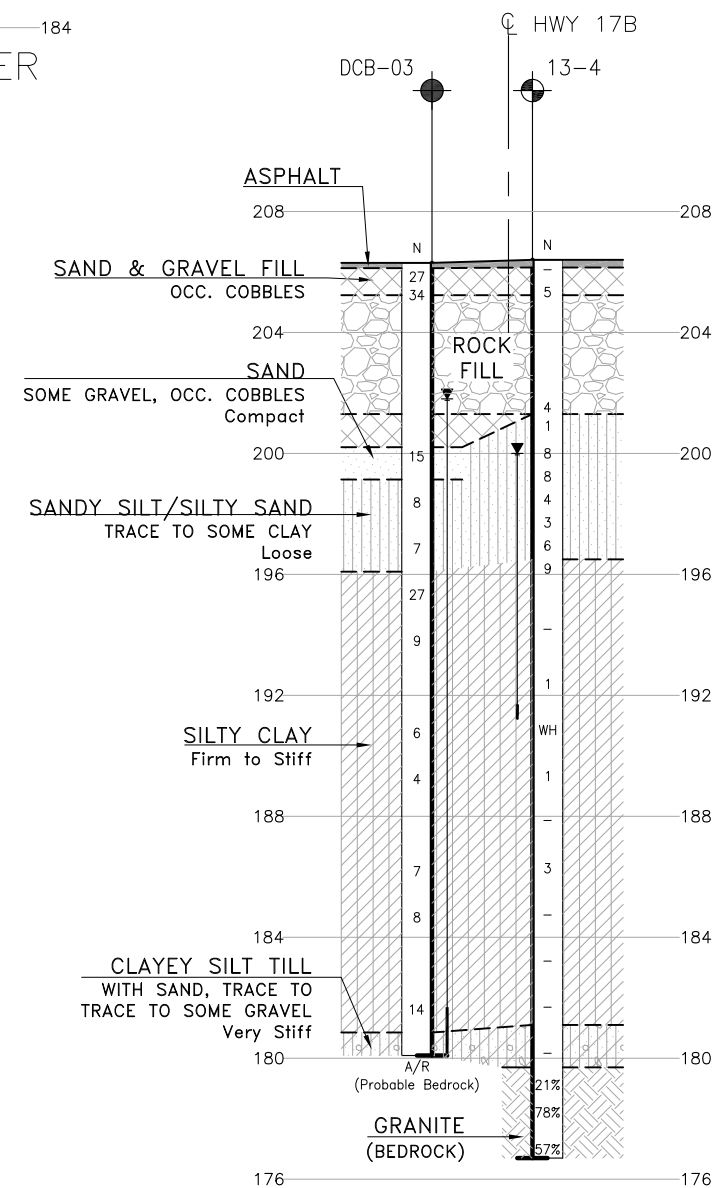
SECTION ALONG W. ABUT. BRGS.



SECTION ALONG W. PIER



SECTION ALONG E. PIER



SECTION ALONG E. ABUT. BRGS.

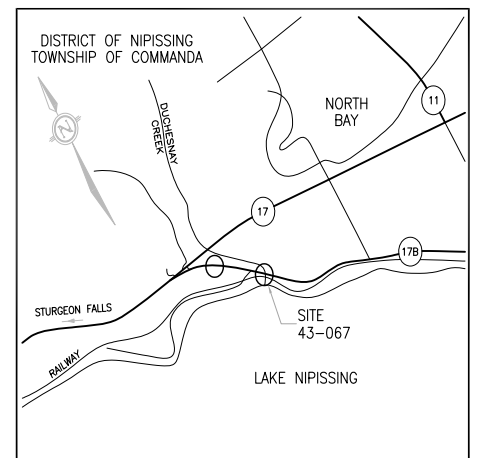
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN



CONT No  
GWP No 5120-07-00

HIGHWAY 17B  
DUCHESNAY CREEK  
BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEYPLAN

## LEGEND

	Borehole (By Thurber)
	Borehole (By Stantec)
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
13-3	204.5	5 132 119.5	304 518.7
13-4	206.4	5 132 060.5	304 579.9
DCB-01	204.8	5 132 133.4	304 504.3
DCB-02	204.4	5 132 114.8	304 514.2
DCB-03	206.3	5 132 055.7	304 575.3
DCB-04	206.6	5 132 046.6	304 594.3
DCB-05	196.2	5 132 108.4	304 540.7
DCB-06	197.8	5 132 097.8	304 532.2
DCB-07	200.7	5 132 081.3	304 568.7
DCB-08	197.7	5 132 072.8	304 558.0

## 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. 41H-166

REVISIONS	DATE	BY	DESCRIPTION
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DRAWN	AN	CHK DJP	SITE
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STRUCT	DWG	2	